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SEISMIC VULNERABILITY INDEX FOR LOW-RISE COMPOSITE REINFORCED CONCRETE AND MASONRY BUILDINGS IN NEPAL

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Abstract

Three- to five-storey high reinforced concrete (RC) frames with brick masonry infill walls have been predominantly used for housing construction in urban areas of Nepal since the late 1980s. These buildings are typically constructed without detailed engineering input during design and construction. Several buildings of this type were damaged and/or collapsed in the April 25, 2015 Gorkha earthquake (M 7.8), even in areas characterized with moderate and low shaking intensity such as Kathmandu Valley (MMI intensity of VI and VII). Due to inadequate column size, amount of reinforcement, and an absence of seismic detailing, RC components were not effective in resisting seismic loads. As a result, these buildings behaved essentially like shear wall structures and their lateral load resistance depended mostly on the shear capacity of unreinforced brick masonry walls. This building typology will be referred to as "composite RC and masonry buildings" in the paper. The paper describes the results of a study of 98 buildings of this typology at three different sites that were affected by the Gorkha (Nepal) earthquake. The main objective of the study was to correlate the observed damage grades for individual buildings with the corresponding wall density index, that is, a ratio of the sum of cross-sectional areas of all walls in each major direction of the building plan and the total floor plan area. The damage classification was based on the European Macroseismic Scale (EMS-98) with some modifications. The data collection was performed using electronic survey tools developed for the Global Earthquake Model (GEM). Each building was characterized by its location (latitude and longitude), and 13 attributes from the GEM Building Taxonomy describing details of the lateral load-resisting system, prevalent construction materials, building height, shape of the building plan, type of floor/roof system etc. In addition, earthquake damage photographs and floor plans with relevant dimensions were mapped for each building. This study builds on the approach taken in previous studies from Chile which used wall density as a seismic vulnerability index for masonry buildings. The results presented in this paper may be relevant for assessing seismic vulnerability of existing RC buildings.

Keywords: seismic vulnerability; reinforced concrete frames; wall density; earthquake damage classification

1. Background

On April 25, 2015, Nepal was affected by a devastating earthquake of magnitude (M_w) 7.8 with the epicenter at Barpak, Gorkha District, a remote hilly area of the country. The earthquake and subsequent aftershocks caused more than 8,700 fatalities and damage or total collapse of more than 700,000 buildings, including several UNESCO World Heritage sites. The earthquake had a significant impact on housing, institutional facilities, heritage buildings, schools, hospitals, and lifelines. Most deaths were caused by the collapse of vulnerable unreinforced masonry dwellings (adobe and stone masonry). However, the damage and collapse of several reinforced concrete (RC) buildings revealed the negative effect of building irregularities and inadequate design and construction practices on the seismic performance of these buildings. RC frame construction is the most prominent building typology in urban and suburban areas of Nepal. This practice originally started in the late 1970s, however the rate of construction increased after the 1988 Udaipur earthquake (M 6.6), mostly due to poor performance of unreinforced masonry buildings. RC construction did not experience significant damage, but relatively few RC buildings were exposed to the earthquake (since the construction practice was not common at that time). Increasing use of RC construction is also associated with economic development in urban areas and social factors (aspirations of residents to live in such construction). Surveys of building construction in the Kathmandu Valley showed that about 49% of buildings constructed in the 1990s were of RC construction, while only 11% of buildings of the 1970s vintage were of that construction type [1]. Most RC buildings in urban and suburban areas of Nepal are of low-rise construction. They are used as residential buildings for extended families, which is a common housing pattern in Nepal. The space at ground floor level in these buildings is often used for commercial purposes (small retail stores). Also, many buildings of this type in the Kathmandu region are used as hostels for workers from rural areas who have migrated to the capital region.

Due to excessively small RC column size, as well as an inadequate amount and detailing of reinforcement, RC components in many buildings were not able to effectively resist earthquake-induced forces in the 2015 earthquake. These RC frame buildings behaved essentially like shear wall structures, hence their lateral load resistance depended mostly on the masonry walls. This paper describes the results of a study of 98 RC buildings at 3 different locations in Nepal that were exposed to the April 2015 earthquake and its aftershocks. The main objective of the study was to correlate the observed damage in individual buildings with the wall density as an indicator of earthquake damage for low-rise RC buildings with masonry infill walls.

2. Design and Construction Practice

Most low-rise RC buildings are three- to five-storey high residential buildings, but some of these buildings are also used as hotels/hostels or commercial buildings. Many buildings have mixed functions, with the ground floor used for commercial purposes and upper floors used for residential purposes. These buildings are known as open storefront buildings and have one or two open sides in plan, as shown in Fig. 1a). Open storefront buildings have a rectangular plan shape with variable plan dimensions. Typically, stores at the ground floor level are 3 m wide rooms separated by brick masonry walls. Fully residential buildings of this type usually have smaller plan dimensions, with 9 to 12 m length and 6 to 8 m width.

Most buildings of this type have one or more structural irregularities. For example, RC buildings with an open storefront are characterized by a torsional irregularity in plan (due to the absence of walls on one or two sides). These buildings are also characterized by a weak storey irregularity, since the shear capacity of the bottom storey is less than the upper storeys. Very often, the top floor in these buildings has a setback with significantly smaller plan area than the lower floors, which is considered as a half-floor (see Fig.1b).

RC frames enclosed by unreinforced brick masonry infill walls are considered to be the main lateral force-resisting system in these buildings. RC floor and roof structures typically have 100 mm thick slabs. A post-earthquake survey of buildings damaged in the 2015 earthquake [2] has shown that the typical column size was 227 mm (9 in) square, but rectangular columns with cross-sectional dimensions of 227 by 305 mm (9 by 12 in) were also found in some buildings. Beams in these buildings were 227 mm (9 in) wide, while the depth ranged from 305 mm (1 ft) to 425 mm (1 ft 5 in). RC columns and beams typically have 4 or more longitudinal deformed steel bars (variable sizes), while the transverse reinforcement (ties) was usually in the form of 7 mm diameter bars at 200 mm (8 in) spacing (in some cases 5 mm wires were also observed). In the majority of the buildings where ties were exposed, anchorage was provided by means of 90 degree hooks (as opposed to 135 degree hooks that are required for ductile seismic performance). Masonry infill walls were built using burnt clay bricks in cement mortar. It was observed that exterior walls are thicker (230 mm) than interior walls (115 mm). Typical brick compressive strength is 7 to 10 MPa, and the mortar mix proportion ranges from 1:4 cement:sand for exterior walls to 1:6 cement:sand for interior walls [3].

Most low-rise RC residential buildings were owner built and were not designed by engineers. Even when engineers were involved, prescriptive design provisions, known as Mandatory Rules of Thumb (MRT), have been followed [4]. MRT are intended for pre-engineered design, where the sizes of key structural components, reinforcement details, and standard design drawings are included. Rigorous seismic analysis and design are not required for construction of low-rise RC buildings up to three-storeys high with a built-up plinth area less than 92.9 m^2 (1000 sq.ft.). These rules should be applicable only to regular buildings, however in practice they have been used for the design of buildings with various irregularities. In some cases, buildings taller than three storeys were constructed following the same rules – without a detailed engineering design. Even when RC frame buildings were designed by engineers, it is likely that the effect of masonry infills was neglected in the design.

a) b)

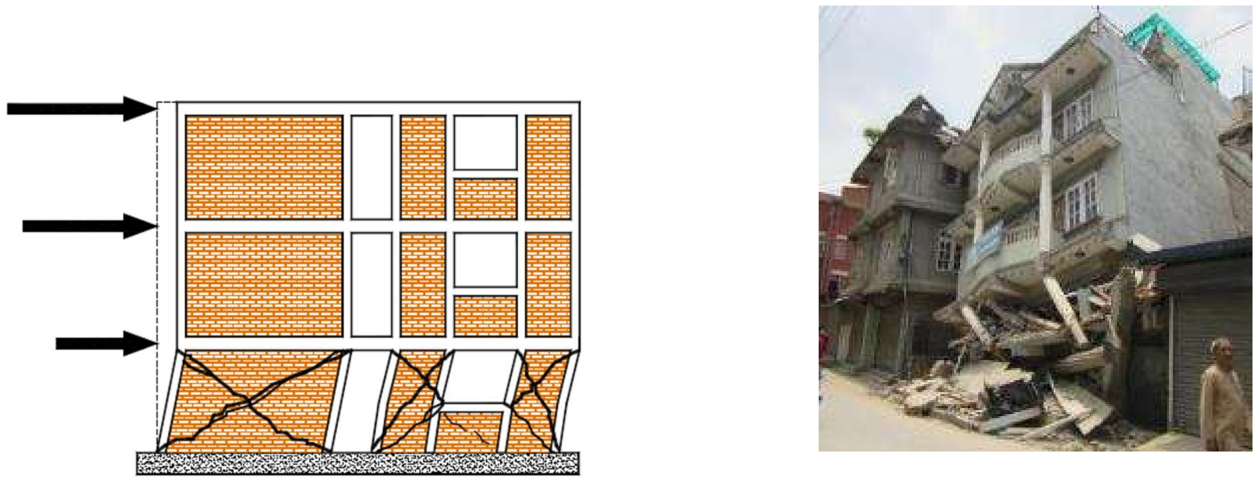
Fig. 1 - RC buildings with irregularities: a) open storefront buildings, and b) a setback at the top floor level (Photos: Svetlana Brzev)

3. Observed Damage and Failure Mechanisms

Many low-rise RC buildings were exposed to the 2015 earthquake and its aftershocks. Fortunately, most buildings, especially those located in the Kathmandu area, remained undamaged. This could be expected, based on the available acceleration records, which showed that the Peak Ground Acceleration (PGA) in Kathmandu was on the order of 0.15g, that is, significantly less than the design PGA of 0.32g, corresponding to the 300 year return period earthquake [5]. However, several RC buildings were affected by the earthquake, with the damage extent ranging from minor damage (cracks in the masonry walls and RC columns) to complete building collapse, particularly in Kathmandu and smaller communities located closer to the epicentre (e.g. Dolakha and Sindupalchok districts). It should be noted that severely damaged RC buildings in Kathmandu were found in a few localized areas (pockets).

RC frame buildings subjected to severe earthquake ground shaking can experience either a flexural or a shear failure. A flexural failure mechanism is characterized by the development of flexural hinges in RC columns and/or beams. Alternatively, RC frames with masonry infills can experience a shear failure, which is characterized by diagonal shear cracking of masonry infill walls and adjacent RC columns. Most extensive damage usually occurs at the ground floor level of a building where the seismic demand is largest and it may lead to the collapse at that level once the base shear capacity has been exhausted (Fig. 2). The capacity of an RC frame with a shear failure mechanism is largely governed by the shear capacity of masonry walls. Essentially, the behaviour is similar to confined masonry walls which are enclosed by RC confining elements (tie-columns and tie-beams), and the lateral seismic loads are resisted by composite action of masonry walls and RC confining elements.

A conceptual force-deformation curve (backbone curve) for a confined masonry wall shown in Figure 3 illustrates a shear-dominant behaviour of a confined masonry wall subjected to a lateral seismic load [6]. There are two critical stages in the behaviour of a confined masonry wall with a shear-dominant behaviour: i) an onset of cracking in the masonry (point 1 on the diagram), and ii) the maximum load-resisting capacity (point 2), which is characterized by extensive diagonal cracking in the masonry wall and the adjacent RC tie-columns. It is expected that a drop in the lateral load-resisting capacity will occur after point 2. This is accompanied by increasing lateral drift and damage, however the structure will still be able to sustain lateral and gravity loads.



a) b)

Fig. 2 - Shear failure of a RC frame with masonry infill walls: a) an illustration of the failure mechanism [6], and b) a shear failure of a RC frame building with masonry infills in the 2015 Nepal earthquake (photo: D.K. Maharjan)

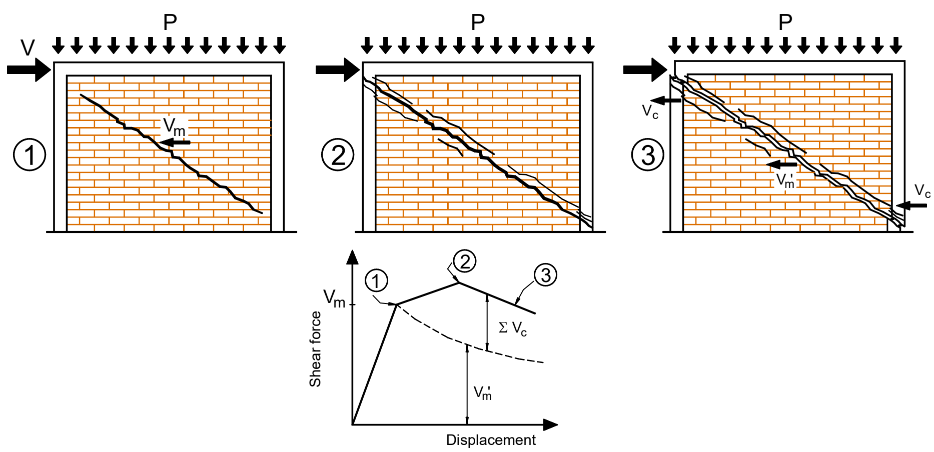


Fig. 3 - Failure mechanism in composite RC and masonry wall system with shear-dominant behavior is similar to confined masonry [6]

4. Damage Classification

Various approaches for post-earthquake building assessment have been proposed to determine the severity of damage in structural and non-structural components and verify structural integrity after a damaging earthquake [7, 8, 9, 10, 11]. The findings of post-earthquake damage assessments influence important decisions, such as whether the building can remain occupied or if it should be vacated after a damaging earthquake. Also, it is important to determine whether a damaged building should be repaired and retrofitted or demolished. Damage classification, which characterizes the type and severity of damage, is a critical aspect of post-earthquake damage assessment. Some publications outline general damage patterns for each damage grade [8, 9, 11], while others offer comprehensive recommendations regarding the extent of damage, e.g. size of crack widths in structural components [7]. The evidence from research studies has also been used to characterize the severity of damage in structural components of masonry and RC structures [13].

a) b)

Fig.4 – Examples of Damage Grade 3 (DG3) from Nepal: a) cracks in a masonry wall, and b) shear cracking in a RC column (photos: S. Brzev)

a) b)

Fig.5 – Examples of Damage Grade 4 (DG4) from Nepal: a) shear failure of a wall at the ground floor level of a severely damaged building (S. Brzev), and b) a vertical separation crack between the wall and the RC column and a major diagonal shear crack extended from the wall into the column (B. Pandey)

Most damage classifications have identified 3 to 5 Damage Grades (DG), ranging from minor damage to total destruction (collapse). These classifications are applicable to various lateral load-resisting systems (e.g. loadbearing masonry walls or RC frames with masonry infills). Damage classification for masonry buildings is associated with an increasing extent of cracking in masonry walls, however the damage in RC buildings with masonry infills is characterized by the damage both in the RC components and masonry infill walls. Very few publications recognize the difference between flexural and shear failure mechanisms for RC frame buildings [7]. Damage classifications for composite masonry and RC buildings are presented in Table 1. It can be seen from the table that the existing damage scales for RC frame structures, e.g. EMS-92 and EMS-98 scale [8, 9], describe damage in RC columns mostly due to flexural behavior (e.g. buckling of reinforcing rods – DG3). The authors of this paper have proposed a revised damage classification for RC buildings with a shear-dominant failure mechanism, which is based on the EMS-98 scale. It is assumed that RC columns experience predominantly shear damage while RC beams do not experience any significant damage due to the nature of this failure mechanism. Examples of damage grades are illustrated in Figures 4 and 5. The authors have found it challenging to make a distinction between the damage grades 1 and 2 (DG1 and DG2) as defined by the EMS-98 scale. The suggested solution is to combine these damage grades, in a manner similar to the Chilean damage classification [12].

Table 1. Damage Classifications for RC Frame Buildings with Masonry Infills: Shear- and Flexure-Dominant Mechanisms

	Shear-dominant behaviour (proposed damage classification)		Flexure-dominant behaviour (EMS-98) [8]	
Damage Grade	Masonry walls	RC columns[1]	Masonry infills	RC columns and beams
Damage Grade 1 (DG1): Negligible to slight damage	Hairline cracks in very few walls (plaster cracks only).		Fine cracks in partitions and infills.	Fine cracks in plaster over frame members at the base.
Damage Grade 2 (DG2): Moderate damage	Hairline cracks in many walls (mostly plaster cracks); cracks along the wall-to-frame interface; crushing in the corners of masonry walls.	Plaster cracks in a few RC columns.	Cracks in partitions and infill walls; fall of plaster; fall of brittle cladding and plaster; falling mortar from the joints of wall panels.	Cracks in columns and beams.
Damage Grade 3 (DG3): Substantial to heavy damage	Diagonal cracks in most walls, but they are not severe and there is no sign of fallen bricks/blocks and wall segments.	Visible shear cracks in RC columns.	Large cracks in partitions and infill walls; failure of individual infill panels.	Cracks in columns and beam-column joints of frames at the base; spalling of concrete cover, buckling of reinforced rods
Damage Grade 4 (DG4): Very heavy damage	Wide diagonal shear cracks in walls; severe damage at the wall intersections; partial structural failure of roof and floor structures.	Wide shear cracks and/or tilting of RC columns.	Not described[2]	Large cracks in structural elements with compression failure of concrete and fracture of rebars; bond failure of beam reinforced bars; tilting of columns; collapse of a few columns or a single upper floor.
Damage Grade 5 (DG5): Destruction	Severe damage and possible collapse of several walls, usually at the ground floor level.			Collapse of ground floor or parts (e.g. wings) of buildings.

Notes: 1) No damage in RC beams; 2) Many infill walls will have failed at this stage

5. Building Survey

The survey was performed in July 2015 (less than 3 months after the earthquake), before the post-earthquake rehabilitation activities started. The following three sites were selected: Sitapaila and Balaju in Kathmandu, and Batar in Nuwakot District. Sitapaila and Balaju are localized areas where significant damage of RC buildings was reported within the Kathmandu Valley. Batar is a semi-urban area in Nuwakot District, where 3- to 5-storey RC buildings experienced damage. A map showing the epicenter of the April 25, 2015 and the May 12, 2015 earthquakes, and the locations where the building survey was conducted is shown in Fig. 6. Note that the Kathmandu sites (Sitapaila and Balaju) are about 80 km away (aerial distance) from the epicentre of the April 2015 earthquake, while the Nuwakot site (Batar) is about 55 km away from the epicentre. Buildings at all 3 sites were primarily affected by the April 2015 earthquake.

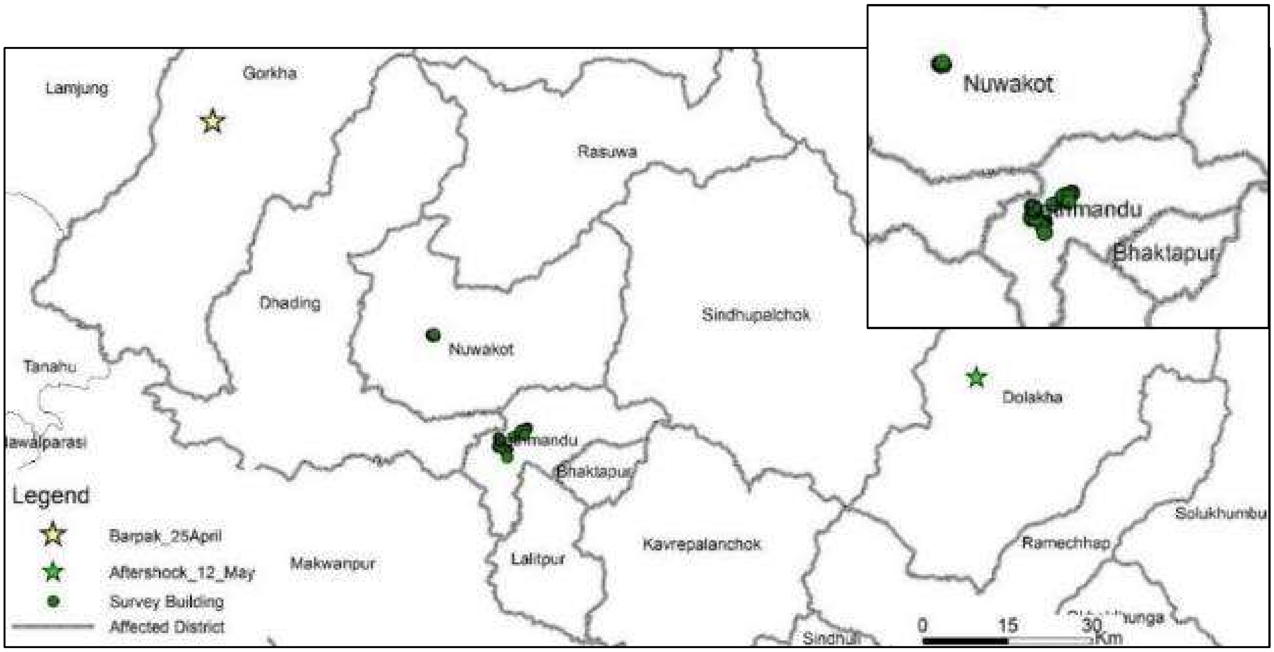


Fig.6 - A map showing the damage survey sites and the epicentre locations for the April and May 2015 earthquakes

In total, 98 buildings were surveyed. Out of these, 46 buildings were surveyed at Sitapaila, 21 at Balaju (both in Kathmandu), and 31 at Batar (Nuwakot District); these numbers correspond to 47%, 21%, and 32% of all surveyed buildings. Therefore, most surveyed buildings (68%) were located in Kathmandu Valley. The number of storeys varied from 2 to 5, however 59 buildings (60%) were 3 storeys high, while an additional 37 buildings (38%) were 2-storey high. Only 2 buildings were taller than 3 storeys (a 4- and a 5-storey building). Most buildings were of recent vintage, with an average age of 11 years (as of July 2015). The oldest building was built in 1989 while the most recent one was built in 2014.

All surveyed buildings were RC buildings with brick masonry infill walls. Most buildings were characterized by a regular (usually rectangular) plan shape. The south façade of a typical building surveyed in Sitapaila, Kathmandu is shown in Fig. 7a, and its ground floor plan is shown in Fig. 7b (note open storefront at the ground floor level). Plan dimensions are typical for the surveyed buildings: 12 m x 8 m (length x width). It was found that the average area of the ground floor plan for the surveyed buildings was 70.3 m.sq., with the standard deviation of 22.04 m.sq. The plan area of the ground floor ranged from 22.23 to 175.6 m.sq. Fig. 7 shows an example of an open storefront building. The building had several window openings at the perimeter. Windows were 1220 mm high with variable width (either 600 or 1220 mm). Doors were 2130 mm high by 780 mm wide. All exterior walls were 230 mm (one-brick) thick brick masonry while all interior walls were 115 mm (half-brick) thick. Typical RC columns were 227 mm (9 inch) square. This building was irregular in elevation. The top floor had a terrace which covers approximately 25% of the plan area, and the south wall at the same level was offset with regards to the lower floors (see Fig. 7c). The calculated wall density index for this building is 1.12 and 0.54% for x- and y-direction respectively. Note that x-direction coincides with the N-S direction shown on the plan in Fig. 7. The damage was in the form of diagonal cracks at the two bottom storeys (the top floor experienced less damage) and it was classified as DG 2 by the survey team. The cracks in mortar joints along the wall-to-frame interface were also observed. The damage mostly occurred in the North-South walls.

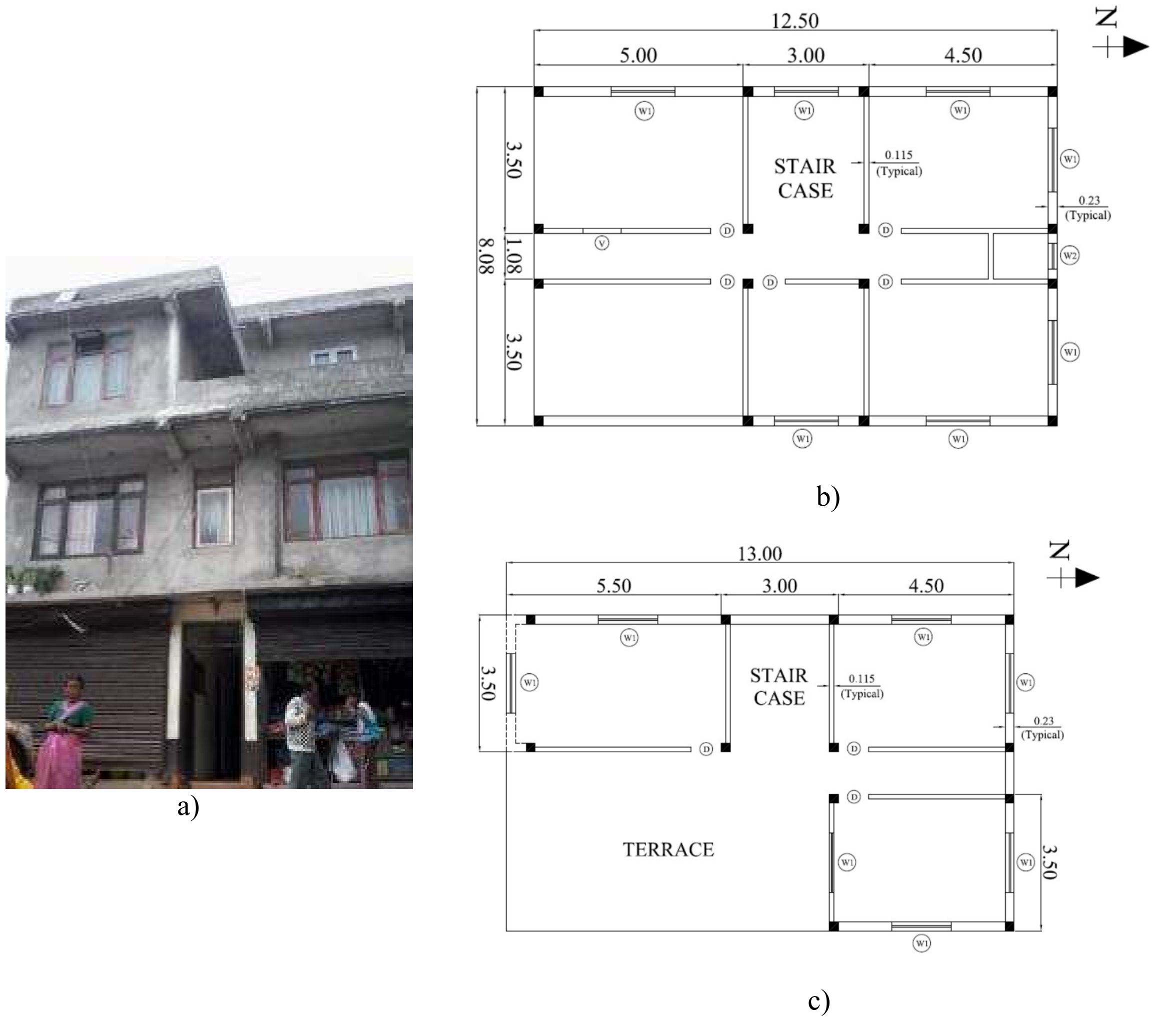


Fig.7 - An example of a surveyed building in Kathmandu: a) a photo showing the south façade (entrance); b) ground floor plan, and c) top floor plan (note that x-direction is horizontal – it coincides with N-S direction)

The data collection was performed using an electronic survey form IDCT DO Survey on a Samsung Galaxy Tab 3 Lite tablet. The survey form was developed in the framework of the Global Earthquake Model (GEM) for use with the OpenQuake platform [14]. Each building is characterized by its location (latitude and longitude), and 13 attributes describing the details of the lateral load-resisting system, materials, height, shape of the building plan, type of floor/roof etc. according to the GEM Building Taxonomy V 2.0 [15]. The research team also took physical measurements of building plan dimensions and wall and column dimensions. Multiple earthquake damage photographs were taken for each building.

6. Wall Density as a Seismic Vulnerability Index

Wall density index, d, is a measure of the amount of walls which provide shear resistance in the specific building direction, and can be determined as a ratio of the sum of cross-sectional areas for all walls along the direction of lateral seismic force under consideration, A_w, and the total floor plan area, A_{ptotal}. It should be noted that only solid walls (without major openings) have been taken into account in wall density calculations. For example, walls with openings on the north façade (along y-direction) shown in Fig. 7b) were disregarded in the wall density calculations. Wall density is determined at the base level of the building (ground floor level), thus A_{ptotal} denotes the sum of floor plan areas above the base of the building. Wall density can be used to assess the seismic vulnerability of masonry and RC buildings with shear wall lateral load-resisting systems.

Several research studies have confirmed a relationship between the wall density and the extent of earthquake damage in masonry and RC shear wall buildings for countries like Mexico and Chile [12, 16]. Chilean researchers have correlated the actual wall density and the observed damage in the 1985 Llolleo, Chile earthquake (M 7.8). It was concluded that a minimum wall density of 1.15% or higher was required to avoid earthquake damage. Buildings with a wall density in the range of 0.50 to 1.15% suffered moderate to severe damage, while buildings with a wall density of less than 0.50% suffered heavy damage [12]. A study of confined masonry buildings affected by the 2010 Maule, Chile earthquake (M 8.8) showed that, in general, buildings with a wall density of 0.9% and higher remained undamaged, while buildings with a wall density of 0.75% experienced severe damage for MSK intensity of VII or higher [16]. The concept of wall density (termed as the wall index) was also used to develop a procedure for assessing seismic vulnerability of RC frame buildings with masonry infills in Turkey [17, 18]. Gulkan and Sozen [18] established a relationship between the wall and column indices and the drift demand in RC buildings. The column index was determined as a ratio of effective column area at the base of the building and the total floor area.

The required wall density for a particular building can be determined based on the given seismic hazard level, type of soil at the building site, masonry shear strength, expected seismic performance (ductility), average storey weight, and the number of storeys [6]. This procedure can be applied to buildings with regular plan shapes and wall layout, that is, without significant torsional effects. It is assumed that all walls at specific storey level have shear-dominant behaviour and that they reach shear capacity simultaneously. The calculation procedure can be adapted for application in countries with different seismic design codes. For Nepal, the required wall density for a single-storey masonry building in seismic zone 1 is 1.1%; this is based on the following seismic design parameters: C=0.08, I=1, and K=4 [5]. It was assumed that the masonry shear strength is 0.33 MPa (corresponding to the brick compressive strength of 7 MPa and 1:4 cement:sand mortar). This corresponds to a required wall density of 3.3% for a 3-storey building.

7. Results and Discussion

Plan measurements and wall dimensions were recorded for all surveyed buildings and it was possible to determine the wall density for the principal horizontal directions for each building. The results show a relation between the wall density and the extent of damage sustained in the 2015 Gorkha earthquake. An average wall density for all surveyed buildings (98 in total) was 1.38%, with a standard deviation (STD) of 1.01% and a coefficient of variation (COV) of 0.703. The wall density in the surveyed buildings ranged from 0.19 to 5.65%. Figure 8 illustrates how surveyed buildings are clustered in Damage Grades (DG) 1 to 4 depending on their wall densities; note that wall densities are shown in both horizontal directions for each building. The chart shows that the cluster with DG1 is spread over a wide range of wall densities whereas buildings with other damage grades are clustered within smaller wall density ranges. Figure 9 shows a relationship between the cumulative number of damaged buildings characterized by different damage grades and the corresponding average wall densities for each cluster. It can be seen that only 3% of all surveyed buildings experienced DG4, and their average wall density was 0.97%. About 20% of all surveyed buildings experienced DG3 and DG4 and the corresponding average wall density was 1.3%. Most buildings (about 80% of all surveyed buildings) experienced damage grades 1 and 2 (DG1 and DG2), and the corresponding average wall density was around 1.5%. The trend line indicates a strong relationship between wall density and the damage grade.

Figure 10 shows the damage grade versus an average and minimum wall density for each building (based on the two horizontal directions). The buildings are clustered based on the damage category. Weighted averages of wall densities for each damage category were used to determine the trend line that establishes a relationship between the wall density and damage grade. The chart shows a clear trend indicating a higher damage grade for buildings with a lower average wall density.

A significant fraction of the surveyed buildings (46 out of 98) were located in Sitapaila, a neighbourhood in the capital Kathmandu. Wall density versus the damage grade for the surveyed buildings at Sitapaila are shown in Fig. 11. The trend line shows a lower average wall density than the overall building sample (see the trend line in Fig. 10). The buildings in Sitapaila experienced significant damage compared to other localities in

Kathmandu. This can be explained by a lower wall density and also poor quality of RC construction, which was observed in Sitapaila.

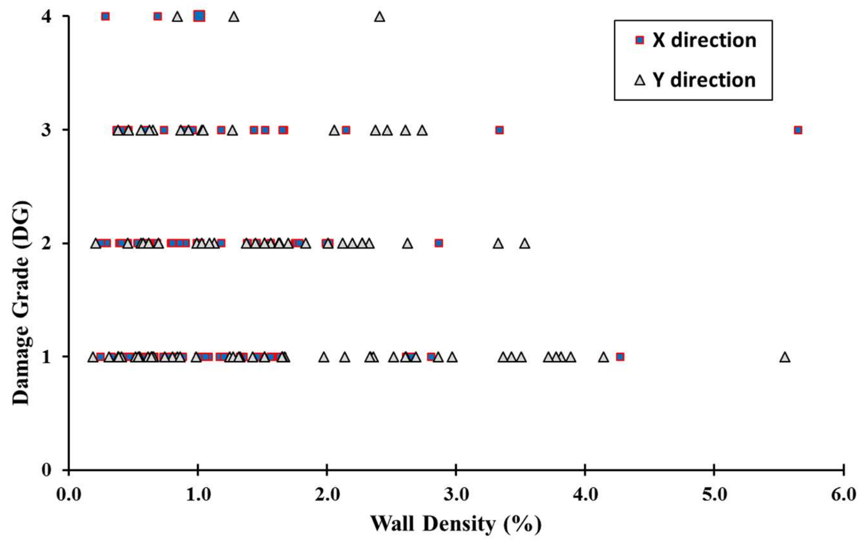


Fig.8 - Damage grade (DG) versus wall density in two horizontal directions for the surveyed buildings (sample: 98 buildings and 2 wall density values per building)

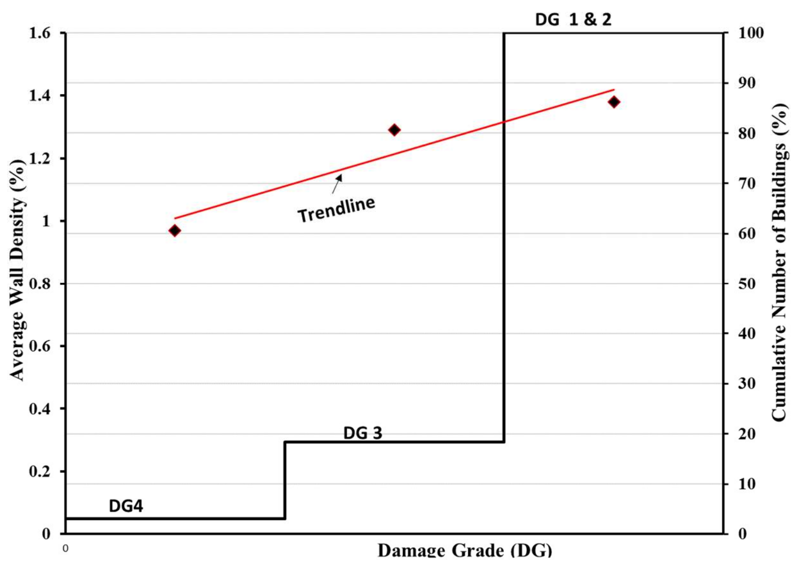


Fig.9 - A cumulative ratio of buildings with the given damage grade (DG) versus the average wall density index

The data was also analysed to understand the effect of the number of RC columns on the extent of damage in the surveyed buildings. The main indicator is Column Index (CI), which was determined as the sum of cross-sectional areas for all columns at the base of the building and the total floor plan area A_{ptotal} (which was also used to find the wall density ratio). An average CI value for all surveyed buildings was 0.37%, with the standard deviation of 0.169% and the coefficient of variation of 0.461. The minimum reported CI value is 0.13% and the maximum value is 1.20%. It was observed that there is no significant relation between the CI and the corresponding damage grade for the surveyed buildings. This can be explained by the fact that for most of these buildings masonry walls are the main lateral load-resisting elements, thus the overall seismic response is governed by the wall shear capacity. It should be noted that RC column areas were also taken into account in the wall density calculations (by considering RC columns as ends of masonry walls).

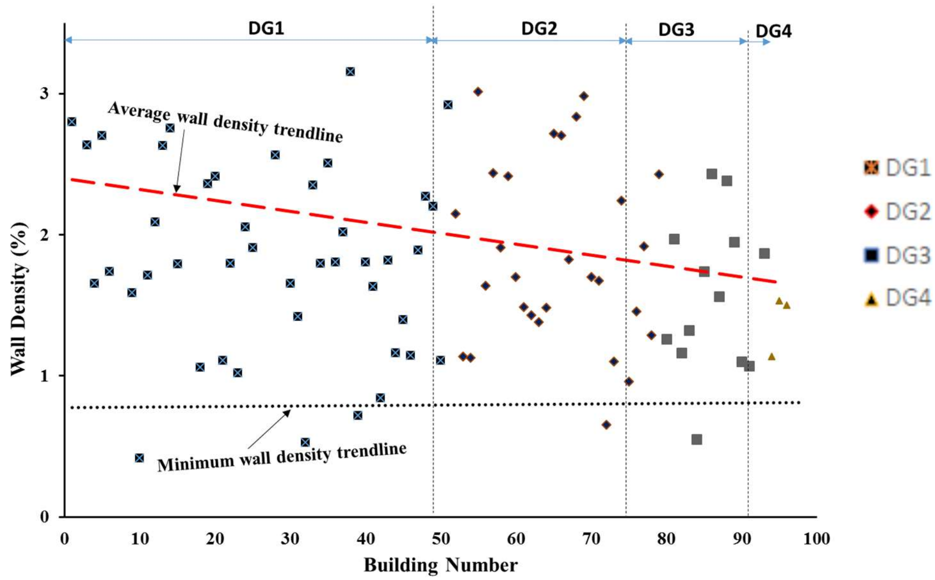


Fig.10 - Average and minimum wall density indices for different damage grades (DG1 to DG4) (sample: 98 buildings)

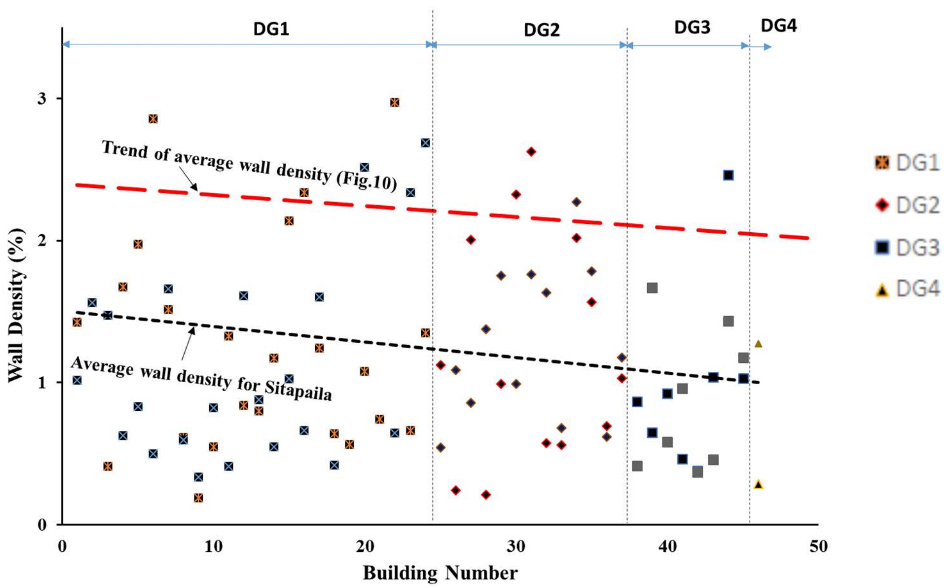


Fig.11 - Average wall density index for the surveyed buildings in Sitapaila, Kathmandu (sample: 46 buildings)

8. Conclusions

The paper presents the findings of a survey of 98 low-rise buildings which experienced damage in the 2015 Gorkha, Nepal earthquake. All buildings were composite RC and masonry buildings. The results have shown a strong relationship between the extent of damage and the wall density for these buildings: the buildings with lower wall density suffered more extensive damage. The buildings demonstrated shear-dominant behaviour which was governed by the masonry wall shear capacity, as opposed to the flexural capacity of RC frames. The relationship between the column index and the extent of damage in these buildings was found to be weak. Based on the limited data considered in this study it can be concluded that wall density may be used as an indicator of seismic vulnerability for RC buildings with masonry walls which are characterized by the predominant shear behaviour.

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