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Evaluation of Structural Performance of Low-rise Reinforced Concrete Buildings to the Gorkha Earthquake and NBC-2020 Demands

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ABSTRACT

There were significant damages to or collapses of low-rise reinforced concrete (RC) buildings in the 2015 Gorkha (Nepal) Earthquake even in areas of moderate to low shaking. These RC buildings with unreinforced masonry brick infill walls constitute the largest portion of individual residential construction since the late 1980s in urban areas of the country. This study looked at twenty-two RC buildings that were surveyed in detail in the field after the 2015 earthquake in Kathmandu and Nuwakot subjected to low to moderate shaking. These concrete frame buildings with masonry infill walls were sampled from the area to include typical residential buildings of height 1 to 5 story with plinth area ranging from 30 to 160 sqm. Nonlinear static analyses of these individual buildings are carried out to investigate the capacity of typical building stock in Kathmandu in relation to the demand of the 2015 Gorkha earthquake, demand of then-active building code, NBC 1994 code and newly revised seismic building code Nepal, NBC105-2020. The study showed that available system ductility and peak resistance of those buildings were significantly low and that they sustained damage even in the low-demand earthquake of 2015. We analyzed the strength and ductility capacity of these buildings to measure those against code demands for a type of building categorized as either 'Reinforced concrete moment frame buildings' or 'Confined masonry buildings' in the new code. The study shows that ductility and overstrength factors stipulated in the code do not apply to almost all sampled buildings. The paper discusses results of field observations including damage conditions in Gorkha earthquake, nonlinear analysis output relating to those damages as well as code demand for strength and ductility in relation to their capacities. It raises issues on how to deal with current building stock and construction practice and the code provisions.

Keywords: Nonlinear Analysis, Assessment, Ductility, Non-ductile RC, Building stock

INTRODUCTION

The 2015 Gorkha (Nepal) Earthquake inflicted heavy damage to rural houses made of stones and adobe bricks, but it also caused significant damages to buildings of low rise reinforced concrete (RC) frame with masonry infill [1]. The damage was also prevalent in areas characterized with low to moderate shaking intensity such as Kathmandu Valley. The spectral acceleration at period of low-rise buildings in Kathmandu was only about 1/3rd of the design earthquake [2]. While it was not widespread as in stone and mud buildings and those in town centers close to epicenters, the observation of those damages revealed the inherent vulnerability of large building stock in Kathmandu and other urban centers. The RC construction with masonry infill became a typical construction for residential building since 1990s [3]. They are mostly low rise with heights ranging from one to five story.

Before National Building Code (NBC) was updated in 2020, these RC buildings were designed and built as either under Mandatory Rule of Thumb (MRT) with prescriptive design provisions in the 1994 NBC code [4] or as engineered design that had some reference to Indian Standards [5] for detailing. The seismic hazard in the code was based on 300 years return period. MRT are pre-engineered design intended for regular low-rise RC construction up to 3 story height with maximum built-up plinth area of 93 sq.m (1000 sq. ft.) . The design basis of these MRT specifications were based on analysis prescribed in the seismic provisions of the NBC code [6,7]. The NBC 1994 code provisions and commentary indicated that these RC construction should have overall ductility, $\mu = 4$. This is, however, stipulated through a structural performance factor, K . The base shear demand as per the 1994 NBC-105 was stipulated as:

$$V_d = C_d.W \tag{1}$$

Where C_d denotes seismic coefficient and W denotes seismic weight. The values of C_d can be determined as follows:

$$C_d = C.Z.I.K \tag{2}$$

where C is the basic seismic coefficient, Z is seismic zoning factor, I is building importance factor and K is structural performance factor, which depends on the structural system and expected ductility potential ranging from 1.0 for ductile moment-resisting frames to 4.0 for structures with no ductility. The RC structure with masonry infill had the performance factor K =2. Since K has multiplicative effect in the base shear demand and it bears value of 4 for structure with no ductility and 1 for RC moment resistant frames, it can derived that MRT and engineered designed RC construction has force reduction owing to a ductility $\mu = 2$ or $\mu = 4$.

The National Building Code of Nepal was revised in 2020 [8]. The revision does not only include new seismic hazard based on return period of 475 years but also revised the base shear demand. The base shear demand for the equivalent static method for ultimate limit state is given by

$$V_d = \frac{C_h(T).Z.I}{R_\mu.\Omega_u}.W \tag{3}$$

Where $C_h(T)$ is period dependent spectral shape factor with value of 2.5 for all soil site class except for very soft soil in the short period range up to 0.5sec. In the equation, Z is seismic zoning factor, I is building importance factor, R_μ is ductility factor and Ω_u is the overstrength factor. The value of R_μ ranges from 2.0 for unreinforced masonry with seismic bands to 4.0 for RC moment resisting frame. The ductility factor, R_μ is 2.5 for reinforced masonry wall or confined masonry with RC tie-columns. Similarly, the value of Ω_u is 1.5 for RC moment resisting frame and 1.2 for reinforced masonry wall or confined masonry. It is comparable to other international code provisions. It can be derived that the effect of R_μ is similar to that of R_d and that of Ω_u is similar to R_o in Canadian seismic code NBCC2020 [9]. The combined effect of R_μ Ω_u is similar to elastic force reduction by R_dR_o.

These provisions in both versions of Nepalese seismic code imply for specific minimum ductility that is associated with force reduction from elastic demand to nonlinear system. While research literatures on the performance of prevalent RC construction in Nepal is limited, the 2015 Gorkha Earthquake provided a first field test of these structures. Although the shaking intensity of the earthquake in Kathmandu and nearby town was not strong enough to mobilize the expected full capacity of code-based structure, the widespread damage in RC frame buildings with masonry infills in the earthquake provided an opportunity to assess the performance of the building stock against the demand of the earthquake. This would further help to get an insight into these buildings in terms of available ductility and strength in comparison to the demand of the new code.

This study investigates the performance of residential RC construction through a sample study of 22 RC building blocks subjected to the Gorkha Earthquake in Kathmandu and nearby town in Nuwakot. These buildings are subset of larger survey of buildings reported in the paper by Brzev et.al[1] and constitute only residential buildings of minimal irregularity, ranging height of 1 to 5 story, size of small to medium with plinth area range of 30 to 160 square meter and damage range of minimal to moderate level. These buildings were surveyed within 2 months of the earthquakes for any damage and for the as-built details of structural members including dimensions, building configuration as well as material properties wherever possible. The buildings were analyzed for static pushover load and comparative study is carried out for available ductility and strength to the demand of code as well as that of the Gorkha earthquake.

BUILDINGS AND THE GORKHA EARTHQUAKE DAMAGE

A team of engineers surveyed damage and other details of 98 low-rise RC buildings in July 2015 in the area of Sitapaila and Balaju in the Kathmandu and Batar in Nuwakot where shaking intensity was light to moderate. This study looks into only 22 buildings selected out of those 98 buildings for further analytical study. While details of the field survey methodology were reported in the paper by Brzev et.al.[1], details including observed damage in the selected building are presented in Table 1.

A visual survey of the damage conducted at the field was documented by the survey team for each building. Most of that damage was found in first floor infill walls. These walls had horizontal and diagonal cracks in the infill panel as well as separation from adjoining frame elements. In some buildings, mostly of those with 3 story and higher in height, cracks in walls were extended to adjoining columns. Separation of walls from the frame was observed even in single story buildings. Significant damage to the RC frame elements (slabs, beams and columns) were not observed except for few cases (NUW-BAT-02 and NUW-BAT-16). Representative damage photographs were presented in Annex 1.

The main seismic-force-resisting system in these buildings is RC frame with brick masonry infill walls. RC floor and roof structures typically have 100–120 mm thick slabs. While RC column size ranges from $230mm \times 230mm$ to $230mm \times 300mm$, beams are larger with dimension of $230mm \times 350mm$ that include monolithic slab depth. Transverse reinforcements (stirrups) in beams and columns usually constitute 7 to 10 mm diameter bars with 90^0 anchorages spaced typically at 200 mm. The compressive strength of concrete used in slabs beams and columns ranges from 15 to 20 MPa. Masonry infills are built using burnt clay brick in cement mortar and have thickness of 230mm. These infills are provided as exterior walls whereas 115 mm thick single-wyth clay brick walls are used as partition walls in building interiors.

Table 1. Damage observed in the study building in the 2015 Gorkha Earthquake.

SN	Building ID	Plinth Area (m^2)	Storey	Number of Columns	Beam Size (mm X mm)	Column Size (mm X mm)	Damage observed
1	KAT-SIT-02	93.6	2	13	230X350	230X230	Cracks in door and window corners in infill walls, separation of wall with columns
2	KAT-SIT-41	71.186	3	8	230X350	230X300	Cracks in partition walls, horizontal, and vertical cracks along frame line
3	KAT-SIT-46	71.5	1	13	230X300	230X300	Cracks in walls separating walls from columns
4	KAT-BAL-01	30	2	9	230*350	230X230	Horizontal cracks in walls in 2nd story
5	KAT-Bal-06	71.55	3	9	230*350	230X300	Minor cracks in top floor
6	KAT-Bal-07	55.1687	4	12	230X350	230X300	No significant damage
7	KAT-BAL-09	51.2616	3	11	230X350	230X230	Cracks in the corners of openings in walls and other minor cracks in walls
8	KAT-BAL-11	39.7735	5	8	230X350	230X350	Significant damage in 5th story, water tanks failed at the roof
9	KAT-BAL-18	53.622	3	9	230X350	230X350	Several cracks in columns and walls in all floors, more severe damage in first floor, horizontal and vertical major cracks in walls
10	KAT-BAL-43	85.8452	2	12	230X350	230X350	No significant damage
11	NUW-BAT-02	77.996	3	14	230*350	230*300	Several damage in 3rd story, significant cracks in most of the partition walls and infills. Cracks include diagonal cracks, first storey column in the verge of collapse
12	NUW-BAT-05	158.976	1	15	230*350	230*300	Minor cracks on first story walls
13	NUW-BAT-07	80.71	3	16	230*350	230X230	Cracks between infill walls and frame at top floor
14	NUW-BAT-08	59.9048	3	12	230X300	230X230	Wide-spread cracks in walls
15	NUW-BAT-09	53.198	3	9	230*350	230*300	Minor horizontal cracks in walls
16	NUW-BAT-10	46.8975	3	9	230*350	230*300	Cracks in infill walls
17	NUW-BAT-15	66.7542	3	12	230*350	230*300	Damage in walls, cracks extended to columns
18	NUW-BAT-16	71.1809	2	11	230*350	230X230	Horizontal and vertical cracks in walls. Horizontal cracks in infill extended to columns
19	NUW-BAT-17	62.33	2	12	230*350	230X300	Minor cracks in walls in first story
20	NUW-BAT-19	58.44	3	15	230*350	230X300	Horizonal cracks in walls
21	NUW-BAT-26	53.2335	2	10	230X350	230X350	Cracks in infill walls, severe cracks in opening corners
22	NUW-BAT-27	49.5672	2	11	230X350	230X350	Damage in first story walls

NONLINEAR STATIC ANALYSIS

Nonlinear static analysis (pushover analysis) was performed to estimate the capacity of all selected buildings with brick masonry infill in each X and Y directions separately using triangular load pattern. SAP2000[10]. Each building is modelled with beams and columns as frame elements, slabs as area elements acting as rigid diaphragms and masonry infill as equivalent struts. The force deformation curve for the beam was used from the model by Mehmet et al.[11]. The concrete with characteristic strength f'_c = 15MPa and 415 MPa steel bar are used for beams and columns. Bar detailing of beam and column are selected based on field record of prevalent construction practice at the time of survey. The relevant strut parameters for infill are determined as per ASCE41-19[12]. These parameters account for geometric and material properties of the URM infills taken as weak infill relative to frame. As strut widths are a function of the frame dimensions, they are calculated for each bay along the exterior walls where infill panels are present. The masonry compressive strength was taken as 7.5 MPa [1]. The elastic modulus of masonry is obtained based on IS 1893:2016[13]. Deformation controlled axial P hinges are assigned at the centre of the compression strut. Rigid diaphragm condition was imposed reflecting the presence of thick concrete slab at each floor. Figure 1 shows a sample SAP model with equivalent strut for the building NUW-BAT-05 loaded in X-direction (E-W).

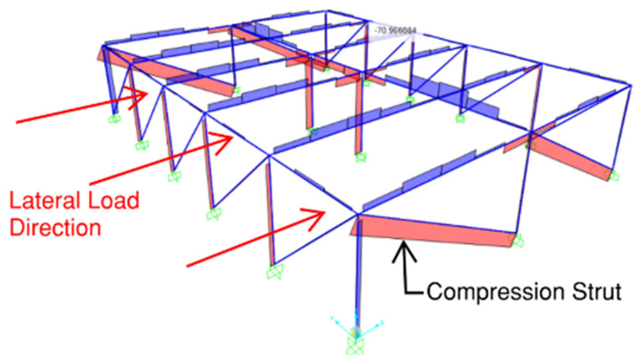


Figure 1. SAP model of RC frame with infill for the building NUW-BAT-05 in E-W direction

To account the hinge formation associated with different performance level under pushover load, RC frame elements are set such that Immediate Occupancy (IO), Lafe safety (LS) and Collapse Prevention (CP) points are set to be 10%, 60% and 90% of total plastic rotation up to collapse point. For masonry infill, IO is set to the yield strain Δ_y, LS is at $0.75\Delta_{res}$ and CP at Δ_{res} following ASCE41-19 reflecting a flexible infill in stiff frame.

The structural building models are pushed all the way up to the major drop of resistance in the force displacement curve. This allowed us to obtain the maximum available ductility of the system. A joint at the top story (roof) is taken as monitor point for displacement control. Details of hinges are recorded for each building under both X and Y direction up to the target displacement, T_d, following FEMA 440 method [14]. In calculation of T_d in equation 4 below, the spectral demand, S_a is obtained from NBC105-2020 for Kathmandu; a triangular load pattern was selected to determine C_0; ratio of inelastic to elastic displacement is assumed to be 1 as C_1 whereas C_2 is taken as 1.1 reflecting life safety criteria as the code performance objective.

$$T_d = C_0 C_1 C_2 \frac{S_a T_e^2}{4\pi^2} \tag{4}$$

For this study, the Capacity Spectrum Method (CSM) of ATC 40 [15]was used to determine the performance point of the structure in each direction under the Gorkha Earthquake loading. In the absence of the other ground motion record, all the buildings were subjected to NS and EW record of Kantipath, Kathmandu (USGS KATNP station] in the respective directions of the buildings. The earthquake demand on strength and displacement were obtained from intersection of ADRS curve with the capacity curve from pushover analysis.

The capacity curves of study buildings are shown in Figure 2. The base shear capacity of the building ranges from 0.07W to 0.42W with a mean value of 0.155W, where W is seismic weight of the building. The ratio of the base shear capacity (strength) is plotted for normalized displacement with target displacement based on the NBC105-2020 demand. It was determined that while most of the buildings exhibit nonlinear response to reach the target displacement, some of them (KAT-BAL-07, NUW-BAT-16, NUW-BAT-26 and NUW-BAT-05) are still in elastic range up to that point of loading. It can be concluded that the ductility is not available as expected in the code in these buildings. It was also observed that some buildings are significantly weak in lateral capacity with less than 0.08W. This is the minimum required strength of the 1994 NBC code (associated with 300 years return period) for the most ductile RC frame building in Kathmandu for the short period structures. For RC buildings with masonry infill walls, the minimum required strength was 0.16W. In the sample study buildings, the mean capacity is less than required strength even assuming they exhibit the required system ductility.

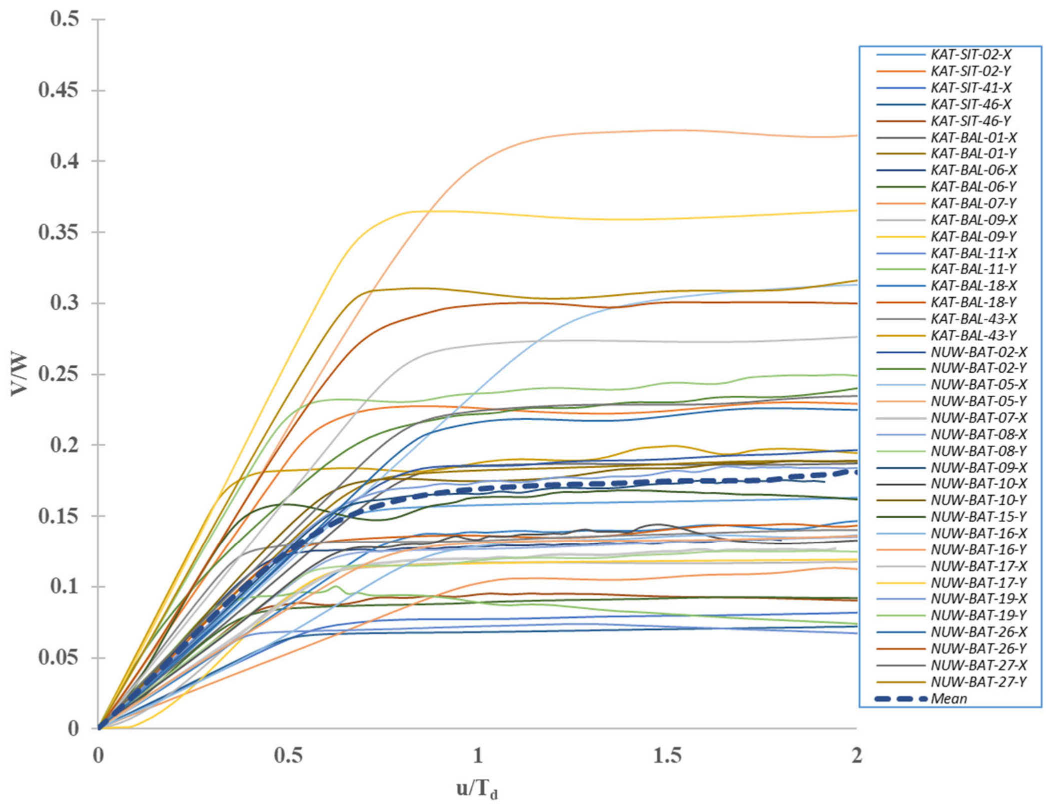


Figure 2. Capacity curve of study buildings under lateral loading.

THE 2015 GORKHA EARTHQUAKE DEMAND

The acceleration displacement response spectrum (ADRS) of the 2015 Gorkha Earthquake indicates the seismic demand in terms of strength and ductility when looked in conjunction with pushover capacity curve. In the capacity spectrum analysis (CSA), it was found that all study buildings do have a performance point which was also verified in the field with the fact that none of these buildings were collapsed or on the verge of collapse. The performance point of these buildings in the earthquake shaking (E-W and N-S records of USGS KATNP station (www.strongmotioncenter.org) in push over curve are the seismic demand values (V_{EQ}, Δ_{EQ}). These values are normalized with seismic weight, W and yield displacement of the building, Δ_y. Figure 3 shows these target points along with code demands on strength.

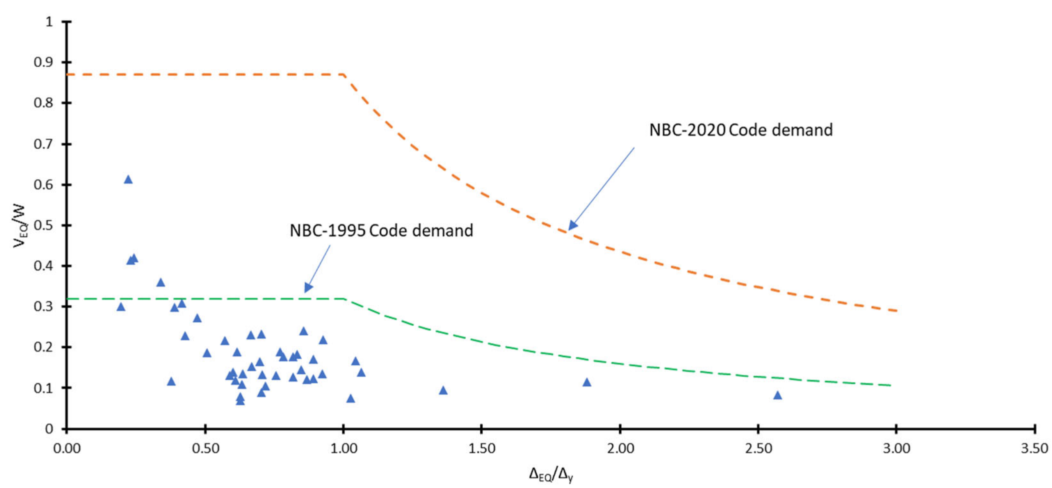


Figure 3. Gorkha Earthquake demand on study buildings.

NBC-1994 provision for base shear expressed in Eqs 1 and 2 above, basic seismic coefficient C was 0.08 for with low fundamental period, seismic zoning factor Z was 1.0 for Kathmandu region, building importance factor I is 1.0 for residential buildings and performance factor K is 2.0 for RC buildings with masonry infill. Among them, all factors have constant value except for K, which is related to ductility level. For any elastic design range (for the displacement range up to first yielding), value of K remains 4,0 for system with no-ductility and reduces in inelastic range by the ductility which can be expressed as Δ_{EQ}/Δ_y. This results the 1994 NBC-code demand curve as shown above figure.

Similarly, NBC-2020 provision for base shear expressed in Eq 3 above, spectral shape factor $C_h(T)$ for short period building is 2.5 and seismic zoning factor Z is 0.35 for Kathmandu region. The building importance factor I is same as 1.0 for residential buildings. The ductility factor R_μ and overstrength factor, Ω_u are related to force reduction. For the peak resistance that includes overstrength, the elastic force is reduced simply by R_μ which is simply equivalent to available ductility. Using all these parameters to calculate code demand, NBC-2020 curve was established in figure 3.

The figure shows that the demand of the Gorkha Earthquake to the buildings in Kathmandu was significantly low compared to what code was asking for although the magnitude of earthquake was significant (Mw7.8) there was widespread damage in in non-engineered buildings. The notion that the earthquake was big but the fact that it did not create significant damage to RC structure in Kathmandu led many to believe that prevalent construction system was not actually bad and that can be continued [2]. This analysis results correctly put the context of demand and explain why even damage was actually not expected in those buildings.

Figure 4 shows how most of the buildings were almost to their yield value. It explains the situation that many similar looking buildings were undamaged whereas their counterparts were significantly damaged. With low level of system ductility in place, buildings look all enact with no damage before yield point but once the buildings surpass this threshold, they may sustain significant damage or even collapse. With small variation of detailing and construction material, some buildings might have just crossed their yield capacity and went through severe damage while others remained elastic.

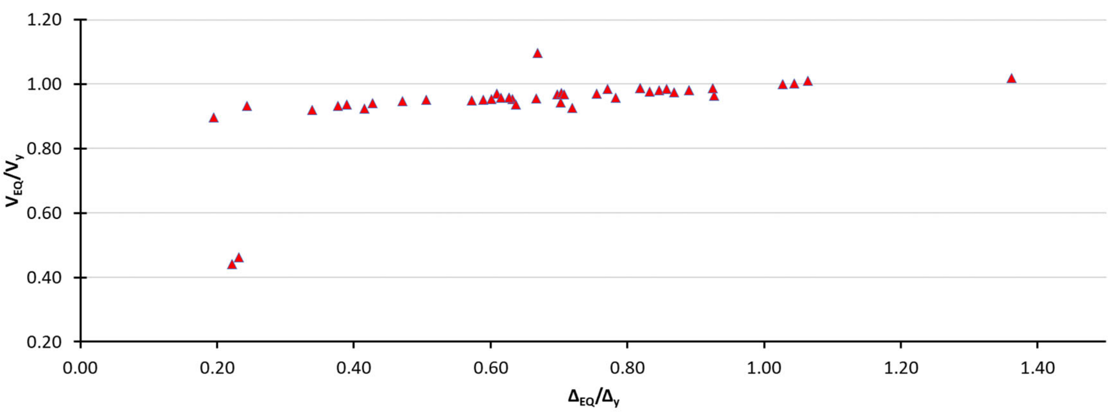
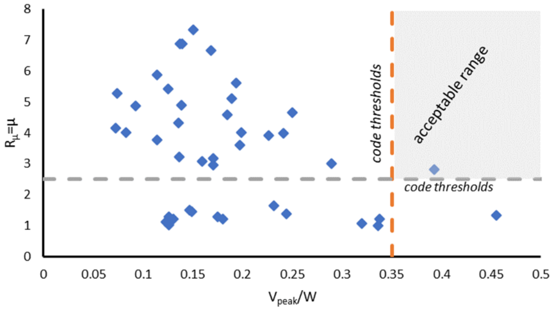


Figure 4. Gorkha Earthquake demand to buildings in relation to yielding.

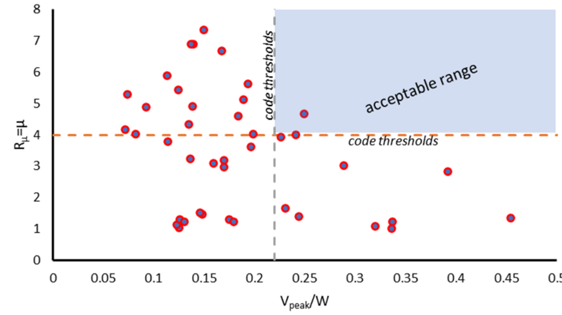
PERFORMANCE OF BUILDINGS IN COMPARISON TO NBC- 2020 DEMAND

A comparative study was carried out to these study buildings based on available ductility to ductility band and available strength to strength demand as per new code provisions of NBC-2020. In the new code, RC construction with infill can be either categorized as ductile moment resisting frame or confined masonry. There is no separate category of "RC frame with infill" as in NBC-1994. In the new code, not only there is significant ductility requirements for RC moment resisting frame but also there is requirement for reserve strength beyond deign (yield) strength. The ductility factor is 4.0 and overstrength factor is 1.5 for this typology. Requirements for confined masonry is less with ductility factor of 2.5 and overstrength factor of 1.2. Combining both factors, the elastic demand in RC moment resisting frame can be reduced by a factor of 6.0 where as the elastic demand can be reduced by a factor of 3.0 in confined masonry.

The elastic demand for a regular building of period shorter than 0.5 sec in Kathmandu region in hard to soft soil is 0.87W. If we analyze and design a building as moment resisting frame, there should be available ductility of μ=4. The corresponding peak resistance of the building should be at least 0.22W. However, if we consider the building as confined masonry with RC ties and columns, the available ductility should be minimum of 2.5 and corresponding peak resistance should be 0.35W, significantly higher than moment resisting frame. In Figure 5, we plot the study buildings tatus against these requirements. In either case, there is only one building that meets the code seismic demand. Other buildings either don't have enough strength or don't have inherent ductility to meet the code expectations.



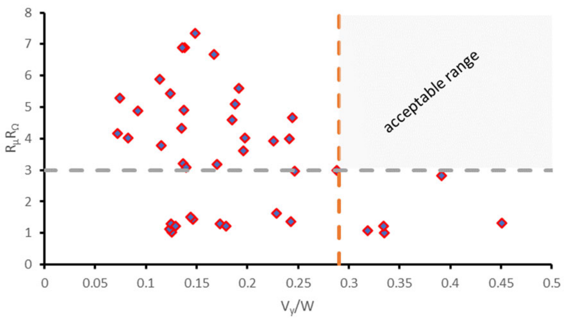
a. Building as a confined masonry



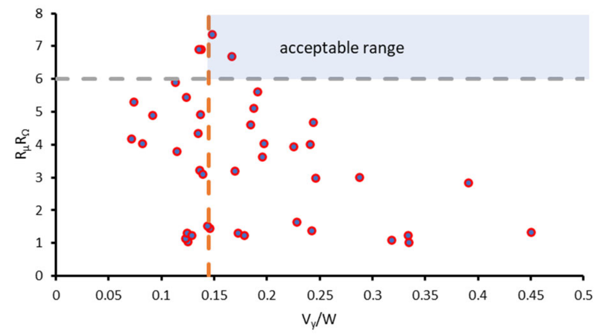
b. Building as reinforced concrete frame

Figure 5. Performance of study buildings in ductility and strength demand as per the code.

Similarly, we can also look at the status of these buildings in relation to yield strength. Figure 6 shows building data in relation to overall force modification though both ductility and overstrength factors. They are plotted against the yield (design) strength of buildings. From this perspective, none of the study buildings qualify to be considered as confined masonry as per the new code. Only two buildings surpass the minim thresholds for overall force modification as a moment resisting frame.



a. Building as a confined masonry



b. Building as reinforced concrete frame

Figure 6. Performance of study buildings in ductility and force modification requirement of code.

BUILDING PERFORMANCE CRITERIA

Figure 7 shows building cases of study samples with plastic hinges in structural components when buildings are subjected to displacement-controlled pushover up to the target displacement. While it does not indicate whether a particular building is undergone global collapse when pushed to this displacement, it shows number of severe local plastic hinges likely to be unacceptable based on safety criteria. In the figure, life safety (LS) criterion was chosen to be performance objective.

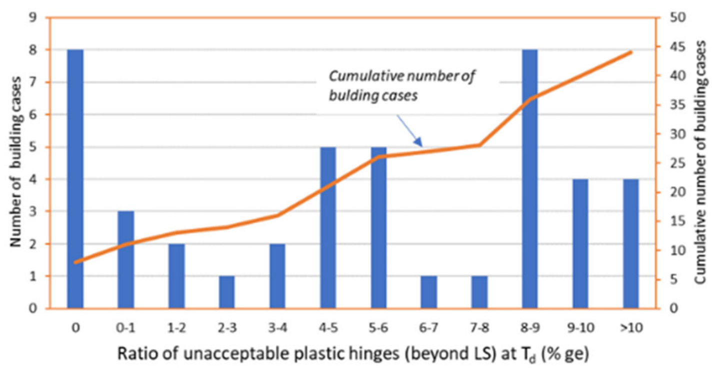


Figure 7. Hinges beyond life safety at target displacement.

Among a total of 44 building cases (22 buildings in two directions), only 8 cases have no hinges beyond the LS criteria. The majority of the buildings analysis cases showed unacceptable hinges (beyond LS) more than more than 5% when building are pushed to code based target displacements .

FUTURE WORKS

Since these buildings largely represent the current building stock in Kathmandu and other urban centers, the study suggests further investigation of larger samples and develop a mitigation strategy to reduce the vulnerabilities to avoid collapse with potential life losses in future earthquakes in the country.

CONCLUSIONS

A study of a total of 22 sample survey buildings subjected to the 2015 Gorkha earthquake shaking in and around Kathmandu shows that the earthquake demand was much smaller than expected based on the code. The nonlinear static analysis of these RC low rise buildings shows that they would underperform in any moderate to heavy shaking that can be attributed to lack of ductility and reserve strength. These buildings do not meet the criteria of ductility and over strength stipulated in the current building code of Nepal NBC-2020. Since these buildings largely represent the current building stock in Kathmandu and other urban centers, future work is recommended to carryout large scale study including cities other than Kathmandu and develop a mitigation strategy to reduce the vulnerabilities to avoid collapse with potential life losses in future earthquakes.

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REFERENCES

[1] Brzev S., Pandey, B., Maharjan D.K., and Ventura C. (2017). "Seismic Vulnerability Assessment of Low-Rise Reinforced Concrete Buildings Affected by the 2015 Gorkha, Nepal Earthquake". *Earthquake Spectra,* 33(1_suppl):275-298. doi:10.1193/120116eqs218m.

[2] CAEE (2016). "The April 25, 2015, Gorkha, Nepal Earthquake Reconnaissance Report", ISBN: 978-0-9685376-2-6, Canadian Association for Earthquake Engineering, Vancouver, BC, Canada

[3] JICA (2002). "The Study on Earthquake Disaster Mitigation in the Kathmandu Valley, The Final Report", Tokyo, Japan.

[4] DUDBC (1994). *NBC 201-1994 Mandatory Rules of Thumb - Reinforced Concrete Buildings with Masonry Infill*, Nepal *National Building Code*, Kathmandu, Nepal

[5] BIS (2000). *Indian Standard Plain and Reinforced Concrete Code of Practice- IS456-2000*, Bureau of Indian Standards, New Delhi.

[6] DUDBC (1994). *NBC 105-1994 seismic design of buildings in Nepal*, Nepal National Building Code, Kathmandu, Nepal.

[7] DUDBC (1994a). Commentary on *NBC 105-1994 seismic design of buildings in Nepal*, Nepal National Building Code, Kathmandu, Nepal.

[8] DUDBC (2020). *Seismic Design of Buildings in Nepal, NBC 105: 2020*. Nepal National Building Code, Kathmandu, Nepal,

[9] NRC (2020). *National Building Code of Canada*. Canadian Commission on Building and Fire Codes. Canada.

[10] CSI (2020) "*SAP2000 Integrated Software for Structural Analysis and Design*," Computers and Structures Inc., Berkeley, California.

[11] Mehmet I., Hayri. B. (2006). "Effects of plastic hinge properties in nonlinear analysis of reinforced concrete buildings Engineering Structures" Volume 28, Issue 11, 1494-1502

[12] ASCE/SEI 41-17 (2017). *Seismic evaluation and retrofit of existing buildings*. American Society of Civil Engineers (ASCE), Reston, VA.

[13] BIS (2016) *Criteria for Earthquake Resistant Design of Structures*, BIS:1893:2016, Bureau of Indian Standards, New Delhi

[14] ATC (2005). *Improvement of nonlinear static seismic analysis procedure- FEMA 440*, ATC-55 Project. FEMA, Washington, USA

Annex -I

Selected Damage Photographs of Study Buildings in 2015 Gorkha (Nepal) Earthquake

 KAT-SIT-02	 KAT-SIT-41	 KAT-SIT-46
 KAT-BAL-01	 KAT-BAL-09	 KAT-BAL-11
 KAT-BAL-18	 NUW-BAT-02	 NUW-BAT-07
 NUW-BAT-08	 NUW-BAT-10	 NUW-BAT-15
 NUW-BAT-16	 NUW-BAT-26	 NUW-BAT-27