EARTHQUAKE HAZARD CENTRE
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Contents

Editorial

URM and confined masonry

The major article of this issue is a report on the performance of unreinforced masonry and concrete buildings during the 2015 Nepal earthquake. It is no surprise to read that many URM buildings collapsed or were badly damaged. The authors attribute much of the damage to a lack of horizontal tying within buildings. Where walls are not tied together and walls not tied to floors, the walls can easily fall away from the building in a catastrophic collapse mechanism. Confined masonry on the other hand offers a far safer alternative where horizontal tie beams at floors and roof not only help confine the masonry panels, but also tie all walls together. Provided confined masonry buildings are well designed and built they avoid the outward collapse of walls. Another benefit of confined masonry is from the presence of vertical reinforced concrete ties. These small columns, if you like to call them that, provide further confinement to the masonry and enable diagonal compression struts to form in the walls to enable them to act as shear walls. The second of half the paper on the Chilean experience of confined masonry will be published in the next issue of this newsletter.

Earthquake engineering education: offer of assistance

Around the end of 2017 the Editor of the Earthquake Hazard Centre Newsletter will be retiring from Victoria University of Wellington. This means I will be free to contribute my seismic design and architectural skills in other countries. I am very open to visiting developing countries in order to give lectures or participate in courses/workshops related to my expertise. I have authored or co-authored the three books *Structure as architecture*, *Seismic design for architects* and *Seismic isolation for architects*, am the co-developer of the RESIST software, and have won several tertiary teaching awards. The most strategic ways to use my skills and experience would be for me to contribute to workshops/seminars where I could work with the teachers of structures to architectural students from a number of universities. But I will be pleased to be based in a college of architecture somewhere to contribute to the Structures and even Studio programmes. I particularly enjoy helping architectural students resolve the structures of their design projects. There is no need to be paid for such work but accommodation would be appreciated. The duration of such visits could range from four to twelve weeks. When I will not be teaching I will pursue a research interest in non-technical approaches to improving the seismic safety of housing.

I could be available late this year or anytime in 2018 or beyond depending on what interest there is in this offer. Anyway, I will be pleased to consider any ideas you might have. Please forward this offer to any colleagues who may be interested. I will send my CV upon request. Contact me at andrew.charleson@vuw.ac.nz.

Virtual Site Visit No. 45: Retrofitting a reinforced concrete frame building with eccentrically-braced frames

This building is a mixed use residential and commercial building located on the outer edge of Wellington's CBD. It was constructed in the 1960s, some fifteen years before structural engineers began intentionally designing ductile buildings. It was only in the mid-1970s that the Capacity Design principle was codified. For frame buildings, like the subject of this site visit, this means columns stronger than the beams.

As seen in Figure 1, this is obviously not the case here. The lateral resistance is provided by frame action from the spandrel or upstand beams and relatively small section reinforced concrete columns. By today's standards, this building exhibits what we call 'critical structural weaknesses', and these have led to the building being seismically retrofitted.

The first stage of the retrofit comprised excavating and retaining a void around the building to place a new reinforced concrete foundation beam. This beam which is approximately two metres deep is strongly bonded into the existing building foundations (Figure 2). It provides the foundation for a massive one bay, one storey reinforced concrete moment frame on each face of the building. Above that level rises a three double-storey eccentrically braced frame (Figure 3). It has just been craned into position and the next step will be to strongly attach it at each beam level to the floor diaphragm. Note that the new steel frames will only collect inertial loads from every second floor. This means the bending and shear strength of the existing columns must resist the inertial forces from every alternate floor and transfer them up and down into diaphragms tied into the eccentrically braced frames.

In a future site visit we will consider the completed project after the large moment frames have been completed. It will be important that these frames are stronger than the braced frames who are expected to form structural fuses in the eccentric regions during seismic overload. This is another example of the application of the Capacity Design principle.

Figure 2. A view of the reinforcing of the new foundation beam with an existing corner column to the left.

Figure 1. Elevation of the building being retrofitted. Note weak columns and strong perimeter beams.

Figure 3. The new steel eccentrically braced frames have just been craned into position and now need to be strongly attached to each floor diaphragm at beam level.

Summary of the paper "Performance of Masonry and Concrete Buildings During M7.8 Gorkha (Nepal) Earthquake of April 25, 2015"

By D. C. Rai, V. Singhal, S. Bhushan Raj and S. Lalit Sagar. Presented at the 16th World Conference on Earthquake Engineering, 16WCEE 2017, Santiago Chile, January 9th to 13th 2017.

Introduction

Nepal and the neighbouring regions suffered a major earthquake on 25th April, 2015 which was followed by strong aftershocks even after a fortnight of the main event. The earthquake killed more than 8000 people, destroyed about half a million buildings completely and disrupted the road network in the mountainous terrain by surface ruptures and landslides. This paper aims at providing a brief overview of the earthquake and its effects on built environment especially masonry and concrete buildings, as observed in the affected areas of Nepal and adjoining Indian states of Uttar Pradesh and Bihar during the field trip undertaken by authors.

The Himalayan region is one of the most seismically active regions in the world producing significant number of earthquakes of M8.0+ magnitude in the past. The largest M8.1 event, known as the 1934 Nepal-Bihar earthquake caused widespread damage in Nepal and Bihar, and around 10,000 fatalities were reported. The M7.8 earthquake was not completely unexpected in the Central Nepal region, as several studies had indicated the likelihood of earthquakes of magnitude greater than 8.0 based on the slip deficit estimation and accumulation of strain energy in the region. This has been anticipated in early 1990's and confirmed by recent studies.

Where acceleration response spectra of the recorded motions are compared with the code prescribed elastic design response spectrum corresponding to zone A of the Nepal seismic code and the zone V of the Indian seismic code for the design basis earthquake (DBE) in soft soil site. It is clear that in the acceleration-controlled regime (i.e. short period range which is typical for low-rise unreinforced masonry and infilled RC frame construction), the ground motion has higher acceleration demand than the code-expected demand in the most severe seismic zone.

Seismic Performance of Masonry and Concrete Buildings

Unreinforced Masonry (URM) Structures

Unreinforced masonry buildings were the most prevalent building type before masonry infilled RC structures became popular in Nepal. Many 50-60 year old unreinforced masonry buildings in Bhaktapur were severely damaged not only due to their deteriorated strength but also due to their inherent structural defects. The box-like action achieved by integrating peripheral walls in unreinforced masonry buildings is an important earthquake resistant feature. The provision of continuous horizontal bands at different levels of the building helps in maintaining structural integrity with all walls and floor diaphragms acting together as a single unit under lateral loads. However, in most of the collapsed buildings, it was observed that there were no horizontal bands connecting the wall units (Fig. 4a). The cross walls in this type of construction were simply butt jointed and had no interlocking features which resulted in their separation by the formation of vertical cracks at the corners (Fig. 4b). However, as per the present Nepal National Building Code, at the junction of two or more walls, reinforcement in the

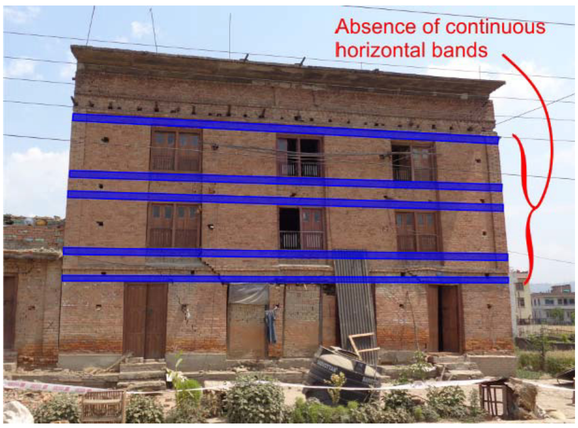

Figure 4a. Typical URM construction in Nikosera: absence of continuous horizontal bands

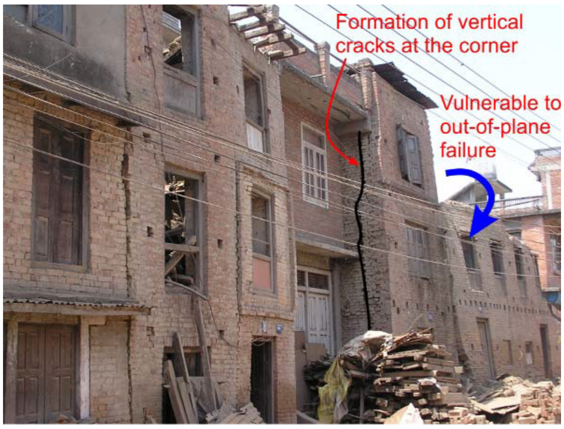

Figure 4b. Typical URM construction in Nikosera: formation of vertical cracks at corners which resulted in separation of cross wall from main wall

form of timber or steel should be provided to integrate the box action for the peripheral walls. Due to the absence of positive connection between the walls at corners and at T-junctions, these walls behaved as free-standing slender walls subjected to large out-of-plane seismic forces due to their heavy mass which often exceeded their capacity. Thus, these separated walls were vulnerable to out-of-plane collapse and many failed during the shaking.

Though the out-of-plane failure of walls in unreinforced masonry buildings was more common, the in-plane damage by step-type diagonal cracks in masonry walls extending to the full storey height was also observed

Figure 5a. URM buildings failures in Nikosera: collapse of three storey unreinforced masonry building

which further reduced the out-of-plane strength of walls and increased the risk of out-of-plane collapse (Fig. 5a). From the failure pattern of building shown in Fig. 5b it can be observed that the in-plane damage was followed by out-of-plane collapse. Closely spaced large openings are detrimental to the seismic performance of masonry structures, which was also observed in the partially collapsed URM buildings in Bhaktapur and provision of openings of irregular sizes is also not a good earthquake resistant practice.

Figure 5b. URM buildings failures in Nikosera: combined in-plane and out-of-plane failure of the wall

According to the mandatory rules of thumb of Nepal building code, for URM buildings built with mud mortar, the height of the wall should be less than eight times the thickness of the wall, openings should not be closer than 600 mm and compulsory timber or RC horizontal bands, collar bands and diagonal bands at the corners have to be provided in such buildings. However, it has been observed that many URM structures do not abide by such mandatory guidelines.

Traditional structural elements observed in the cultural heritage structures which survived in the earthquake, are proven examples for the resilience provided by seismic resistant features in building construction. The old masonry buildings, especially heritage structures which survived in this earthquake, were provided with the continuous timber bands at each storey levels. In addition, in the dega temples, wide timber bands were provided on the top of openings which act as lintels for carrying the loads from the upper storeys. The roof was connected to the walls by means of wooden pegs which enhance the box action of the building. The sloping/overhanging timber roof was supported by aesthetically carved timber struts which also act as structural members enhancing the rigidity of the floor/roof. However, it seems that this knowledge of earthquake resistant features was somehow lost during the last few decades leading to the poor seismic performance of the URM buildings. The lack of earthquake resistant features in these masonry structures could also be due to high cost and non-availability of structural timber in Himalayan regions. The use of material other than timber, such as precast RC and steel members, for confining masonry should be investigated for wider application.

Fig. 5 Two blocks in same school received different damage in the earthquake.

Concrete Buildings

In the past five decades, there has been a widespread conversion of traditional Newari houses and unreinforced masonry buildings to masonry infilled reinforced concrete (RC) structures in the Kathmandu valley. Many such RC buildings in Kathmandu suffered varying degree of damage, ranging from moderate damage to complete collapse during this earthquake. Presence of inherently poor construction features significantly added to the seismic vulnerability of these structures. These buildings though built with better construction materials were incapacitated in resisting seismic forces due to the lack of proper professional engineering consultation resulting in poor design details, ignorance of good earthquake resistant practices for RC construction, and poor workmanship. The devastating earthquake of M6.4 in 1988 led to the development of Nepal National Building Code (NBC) with the support of United Nations Development Programme, which was published in the mid 90's. The code was recommended as advisory for buildings in rural areas and mandatory for all public buildings and residential buildings in municipalities where building permit process exists. However, during this field visit, the authors observed numerous violations of codal provisions in the urban built environment, highlighting serious lack of enforcement of the code which is a familiar state of affairs in many regions where the general governance is weak. Substantial number of building collapses or damages could have been averted by complying with the building code provisions. The RC structures in Kathmandu valley can be broadly classified into engineered and non-engineered construction. The non-engineered low-rise buildings, popularly referred as pillar construction, suffered severe damage and complete collapse in many cases. The engineered constructions, though escaped with minor to moderate damage, were deficient in earthquake resistant features similar to the non-engineered construction.

Buildings with open ground and weak storeys are infamous for their poor behaviour in the past earthquakes and this event was not an exception. Many open ground storey buildings collapsed completely due to soft/weak storey mechanism (Fig. 6). Buildings which were partly used for commercial purposes collapsed, often with pancaking of floor slabs, due to open ground and intermediate storeys in the absence of infills. The collapse of these buildings was primarily triggered by the formation of soft/weak

Four storey buildings in Sitapaila, Kathmandu

Building with basement near Kalopul, Kathmandu.

Fig. 6. Open ground and weak storey failures

storey mechanism due to the inadequate wall area, small sizes of RC frame members and poor reinforcement detailing at critical locations.

For non-engineered buildings built by mid-level technicians, the Nepal building code specifies Mandatory Rules of Thumb for RC buildings with and without masonry infills. These documents provide ready to use dimensions and details of structural and non-structural elements, guidelines for the selection of site, the plan of building, the location of wall openings and their details, etc. However, the observed damages reveal the lack of awareness of such provisions among the public. Extensive damage in many houses were caused by the absence of confining members/columns at the critical locations such as at the intersection of walls, areas adjacent to door openings and at the outer periphery of the building. Complying with the mandatory requirement of horizontal RC bands at the lintel and sill levels of openings could have reduced the extent of damage to infill walls in many buildings. The walls projecting outside the framing elements failed in the in-plane and out-of-plane directions due to the absence of the integrating effect of RC bands with the frame elements under lateral loads. There were also damages due to poor site selection such as sloping ground, landfills, and riverbanks.

Large multi-storey commercial buildings and residential apartments which were supposed to be the engineered construction, design and built under professional guidance also suffered extensive damage though they did not collapse completely. The damage to the masonry infill walls such as large diagonal cracks in masonry panels and cracks at the frame-masonry interface was very common in these high-rise structures. The projection of walls outside the framing elements is widely prevalent in the rapidly urbanizing valley region driven by need to utilize the space to its maximum. However, these slender projecting walls when not positively connected or integrated with the building frame become extremely vulnerable to collapse along both in-plane and out-of-plane directions as observed in 15+ storey buildings in Kathmandu and Lalitpur. Moreover, such high-rise buildings weakened by the damage to infills posed serious danger to neighbouring buildings in densely built areas in the event of a strong aftershock ground motion. Diagonal shear cracks in masonry piers near openings were commonly observed in the wall panels where the continuous horizontal RC bands were not provided.

Vertical irregularity in buildings leading to discontinuous load transfer path is not preferred for ensuring good performance of buildings under seismic forces. The codal provisions also prohibit extending the floor area in upper storeys beyond the ground plan area. However, there are many buildings in the study region, where upper storeys

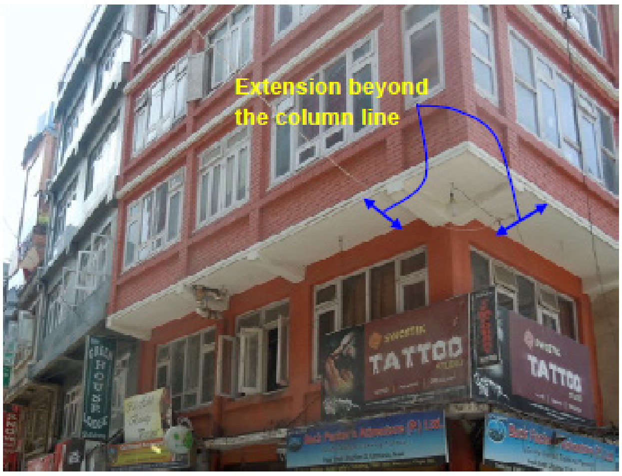

Fig. 7. An example of RC building with upper storeys supported on long cantilever slab or beam, and (b) a building with very large length to width ratio (L/B)

are supported on long cantilever slab or beams (Fig. 7a). Buildings with aspect ratio (such as length to width ratio and height to width ratio) much larger than the code prescribed value of three were surprisingly common in the region (Fig. 7b). Such configurations are generally weak in resisting lateral forces and buildings with such plan and vertical irregularities which did escape with minor damage this time are likely to suffer severe damage in case of stronger shaking expected in the design level earthquake. Many buildings in the worst affected areas were built very close to each other and in many cases with almost no gap between them as they extend up to property lines. Pounding of such buildings either led to chain of collapses involving surrounding buildings or left them leaning out of plumb.

An overview of the seismic performance of RC buildings suggests that some of the key features that contributed to the poor performance of the structures include the following; (a) inadequate size and poor reinforcement detailing of the RC frame members, (b) poor beam-column connection details, (c) weak and slender brick masonry partition walls, (d) extended floor plans in upper stories supported on cantilevered beams and slabs, (e) open ground and soft/weak storey, (f) large vertical and horizontal plan irregularities, (g) discontinuity in lateral load resisting system, (h) lack of soil investigation etc. Many of these poor construction features were also responsible for the widespread damage to RC buildings in Sikkim during the M6.9 India-Nepal border earthquake of September 2011.

Summary of the first half of the paper "Confined Masonry Buildings: The Chilean Experience"

By D. M. Astroza, F. Andrade and M.O. Moroni. Presented at the 16th World Conference on Earthquake Engineering, 16WCEE 2017, Santiago Chile, January 9th to 13th 2017.

Introduction

Confined masonry construction was introduced in Chile in the late 1930s. Confined masonry structures had a great performance during the 1939 Chilean earthquake, providing the first real test for this type of construction during large earthquakes. At that time, the Ordenanza General de Construcciones included some design requirements for low-rise confined masonry buildings up to 2 stories high. A more rational code based on the allowable stress method was published in 1997 and provided design requirements for buildings up to 4 stories high. The confined masonry structures built in Chile in the last 20 years have followed the prescriptions of this code.

The great earthquakes that hit the central and north part of Chile in 1985, 1987, 1997, 2010, 2014 and 2015 have shown the excellent behaviour of these structures subjected to seismic loading, when their design fully satisfied the code requirements. Contrarily, the use of partially confined masonry walls, built with a tie-column at one end of the masonry panel and a vertical tensile bar at the other end, may lead to collapse or, in many cases, to significant damage.

Key components of confined masonry buildings are the horizontal (tie-beam) and vertical (tie-column) reinforced concrete elements. These components are cast in place after the masonry wall panels are built. Typical values for shear strength of Chilean masonry are between 0.5 and 1.0 MPa and the average value of wall density index (ratio between the cross-sectional areas of all walls in one direction and the total floor area of the building) is about 3.5%.

Main Components of Confined Masonry Buildings

The main components of confined masonry buildings are masonry panels and RC confining elements (tie-columns and tie-beams). Unreinforced masonry panels are constructed first, one story at a time, followed by the cast in-place RC tie-columns, as shown in Fig. 8. Finally, RC tie-beams are constructed on top of the masonry panel, simultaneously with the floor/roof slab construction. The wall thickness depends on the type of masonry units used. Most common masonry units used in Chile for confined masonry walls are machine–made multi-perforated clay brick (usually 140 mm wall thickness), followed by the hand-made solid clay bricks (usually 150 mm wall thickness). Hollow concrete block units (usually 150 mm wall thickness) are rarely used because its high permeability and low bond between mortar and the unit have caused serious moisture problems and poor seismic behaviour.

A toothed interface between the masonry panel and tie-columns is used, as shown in Fig. 8, improving the integration between masonry panel and RC confining elements and preventing vertical cracking at the wall ends or out-of-plane collapse. The construction sequence, presence of toothing, and size and detailing of RC confining members are the main differences between confined masonry and RC frame construction.

Figure 5a. Confined masonry construction

(a) Masonry wall is built first

(b) a toothed wall-to-tie-column interface