EARTHQUAKE HAZARD CENTRE NEWSLETTER

Supported by Robinson Seismic Ltd
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Vol.13 No.4 APRIL 2010 ISSN:1174-3646

Contents

Editorial: Still so much to learn

Two weeks ago I attended the NZ Society for Earthquake Engineering annual conference. In the past I have left the conference feeling that more problems than solutions were raised, and this year was no exception. The first three papers in particular highlighted knowledge and practice gaps in earthquake engineering.

First we heard how shear walls of multi-storey buildings experience considerably increased shear forces due to higher mode effects than normal analyses suggest. Design requirements in the NZ code are to be increased to allow for this. Secondly, while still on the subject of shear walls, we learnt how walls attract higher than expected compression forces when they bend. As the tension end of a wall increases in length and rises vertically, surrounding slabs form yield lines in response and apply large compression forces to the wall. This phenomenon has recently been observed in Chile and to some extend accounts for the compression failure and buckling at the ends of shear walls. Although the lack of confinement in the compression ends of walls was also revealed in Chile (see Virtual Site Visit 20 over) the ability of slab strength to overload shear walls might be yet another factor for designers to consider.

In the third paper an experienced practicing structural engineer expressed his misgivings regarding misplaced accuracy in calculations and analyses. We need to appreciate the large uncertainties inherent in current seismicity models and design accelerations he said. There is no value in highly precise calculations nor in complex modal or elastic modelling. How relevant are such analyses where there are often high requirements for ductility in relatively low-strength structures? He advocated keeping structural models simple, avoiding complexity and using simple hand analyses to gain a better appreciation of buildings' seismic performance.

Another paper that jolted any complacency in the level of earthquake resilience of my own country discussed the seismic assessment of existing buildings. The NZ Building Act 2004 requires all local authorities to identify and manage earthquake-prone buildings. The purpose of the policies is to minimize risks to public safety posed by these buildings. One small provincial city has begun the process of evaluating its non-domestic building stock. The rapid method developed by the NZ Society for Earthquake Engineering produces a preliminary assessment of building strength in both orthogonal directions expressed as a percentage of the strength of an equivalent building designed to current standards. A building is considered "earthquake-prone" if its strength is less than 33% than that required today and "earthquake risk" if its strength is between 34% and 67%.

Of the 100 buildings inspected so far 70% have been assessed as "earthquake-prone" and another 24% "earthquake-risk". The factors resulting in such low assessments include building age (related to design strength), type of construction (many unreinforced masonry buildings), critical structural weaknesses like soft-storeys, and soft soil conditions. This is a serious situation and the City Council now has to develop policies to improve, perhaps over several decades, the seismic resilience of its building stock. There are considerable challenges ahead in improving the seismic safety of this city's existing buildings. This problem is shared by cities world-wide. One of the following articles in this newsletter reports on the condition of buildings in Tehran. The situation is similarly grim.

Virtual Site Visit No. 20: RC shear wall and floor diaphragm reinforcing

In this virtual site visit we revisit the RC shear wall building discussed in Virtual Site Visit No. 19. This time we look at several important construction details.

First we observe closely the reinforcement in a shear wall (Fig.1). Since this section of wall is near roof level, it is quite lightly reinforced. As well as vertical bars to resist bending moments, and horizontal steel that is well anchored at each end of the wall with 180 degreee hooks, confining ties at wall ends are visible. It is important to place confining steel at the ends of walls so the concrete in those areas is not badly damaged under the very large compression loads that occur when the wall resists gravity loads and bending moments during earthquake attack. The confining ties wrap around the larger wall compression and tension reinforcement and have 135 degree bends which will allow them to continue to provide confinement when the cover concrete spalls off due to high concrete compression strains.

Secondly, two types of reinforcing steel in a floor diaphragm are noted. Over most of the floor diaphragm reinforcing bars are placed at about 300 mm centres in both directions. This reinforcement is usually sufficient to transfer horizontal inertia forces through the diaphragm into vertical shear walls. This nominal reinforcement is shown in the foreground of Fig. 2 However to the right of Fig. 2 and to the left of Fig. 3 we can see a concentration of reinforcing steel. This steel is forming a tie to transfer inertia forces from certain areas of the floor slab into shear walls. Ties like this are required when shear walls are a considerable distance from some areas of a slab and where penetrations or notches in a slab require a strut and tie mechanism to reliably transfer horizontal forces through a diaphragm into a shear wall. If these ties were not provided the floor diaphragm would have insufficient strength to transfer its forces into the shear walls and would fail during an earthquake.

Fig. 1 Shear wall reinforcement towards top of building.

Fig. 2 Precast ribs and permanent timber infills with diaphragm reinforcement awaiting topping concrete pour. Note the band of heavy diaphragm reinforcement to the right.

Fig. 3 Suspended floor slab ready to be poured. Note band of heavy diaphragm reinforcement to the left.

Summary of the paper "Performance of School Buildings in Turkey During the 1999 Duze and the 2003 Bingol Earthquakes,"

by Turel Gur, Ali Cihan Pay, Julio A. Ramirez, Mete A. Sozen, Arvid M. Johnson, Ayhan Irfanoglu, and Antonio Bobet from Earthquake Spectra, Volume 25, No. 2, pages 239-256, May 2009.

ABSTRACT

Several school buildings were surveyed in the disaster areas of the Marmara (17 August 1999, Mw=7.4), Düzce (12 November 1999, Mw=7.2), and Bingöl (1 May 2003, Mw=6.4) earthquakes in Turkey. Among them, 21 reinforced concrete buildings were found to have an identical floor plan.

The lateral load resisting structural system consisted of reinforced concrete frames (moment-resisting frame) in 16 of the buildings and structural concrete walls integrated with the moment-resisting frame (dual system) in the remaining five buildings. The number of stories above ground in these buildings ranged from two to four. These school buildings provide a nearly ideal test of the effect of a single important structural characteristic on the performance of buildings with structural designs that are uniform in all other respects. Our observation is that the presence of structural walls improves the behavior of reinforced concrete systems drastically.

INTRODUCTION

Teams of researchers from various U.S. institutions and organizations led by Purdue University and in collaboration with researchers from the Middle East Technical University (Turkey) made three surveys of damage to concrete structures in the cities of Düzce, Kaynas¸lı, and Bolu.

This paper focuses on the findings of the survey of 21 of these buildings with identical floor plans developed by the Ministry of Education of Turkey. The structural system of the schools consisted of reinforced concrete frames (moment-resisting frame), except in five of the schools which had structural concrete walls integrated with the moment-resisting frame (dual system). The column layout in all of the schools was identical. The schools with dual systems are located in Düzce, Kaynas¸lı, and Bolu, all of which are within 40 km of the epicenter of the 1999 Düzce earthquake. All of the 16 schools with moment-resisting frames were in Bingöl. School buildings with dual systems in Bingöl were excluded in the comparative study because they had different floor plans. This paper describes the state of the buildings after the earthquakes. The experience points to a simple and obvious solution to avoid severe damage to such buildings.

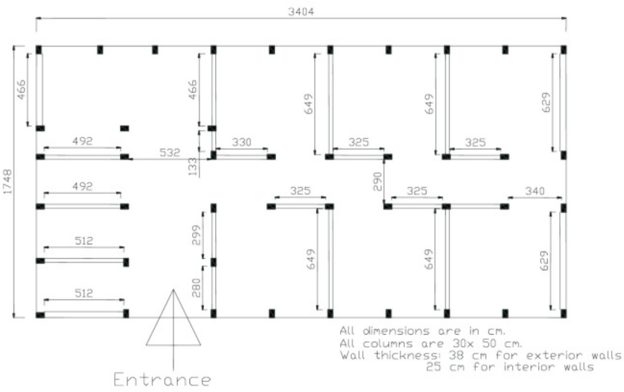

Fig. 4 Typical floor plan for moment-resisting frame systems.

STRUCTURAL PROPERTIES OF THE SCHOOL BUILDINGS AND THEIR DAMAGED STATES. (SCHOOLS WITH MOMENT-RESISTING FRAMES)

All the schools with moment-resisting frames were in or around Bingöl. The number of stories of the buildings ranged from two to four. A typical column layout of these buildings is shown in Figure 4. As the floor plan indicates, the lateral load resisting system in these buildings can be categorized as regular in plan. The majority of the columns were aligned in regular bays, and most of the beams framed into columns. The dimensions of the columns in the buildings were typically 0.3 m by 0.5 m. There are 43 columns in the typical floor plan; 24 of these columns had the strong axis oriented in the short direction of the school buildings. The typical dimensions of the beams are 0.3 m by 0.7 m. The locations of the masonry infill walls varied depending on the use of the space in each school. The thickness of the masonry infill, including the plaster, was estimated to be 0.25 m for interior walls and 0.38 m for exterior walls.

Fig. 5 Building C13-08 before (inset) and after the 2003 Bingöl earthquake. The first story is completely collapsed during the earthquake. The captive columns in the ground floor were shown at the right end of the school (inset).

The total column area at ground level in these moment-resisting frame buildings was approximately 1% of the floor area, regardless of the number of floors. Consequently, the performance of these structures during the earthquake was influenced significantly by the number of floors. The level of damage assigned to the lateral load resisting system with respect to the number of floors was categorized as follows:

- 4 two-story buildings: 3 moderately damaged and 1 lightly damaged
- 11 three-story buildings: 3 collapsed, 6 severely damaged, and 2 moderately damaged
- 1 four-story building: severely damaged

Figure 5 shows the photos of Building C13-08 before and after the earthquake. The first story of the three-story building completely collapsed. The columns of all three collapsed buildings appeared to have failed in shear. Figure 6 shows the extent of shear damage in columns in one of the severely damaged buildings. Damage to the masonry walls was rated separately. The three- and four-story buildings typically sustained severe masonry wall damage.

SCHOOLS WITH DUAL SYSTEMS

The five school buildings with dual systems have the same floor plan as those in Bingöl, with moment-resisting frames, except two bays in each orthogonal direction that are occupied by reinforced concrete walls (Figure 7). The thickness of these walls is 0.2 m. The total concrete wall area is estimated to be 0.4% of the floor area in the long direction and 0.5% in the short direction of the building. These buildings range from two-story to four-story. In addition, buildings with three and four stories had an additional stairwell separated from the main structure by an expansion joint. The most severely damaged dual system was one of the four-story buildings in Düzce. The damage was concentrated in the half-buried basement surrounded by partial height earth-retaining concrete walls. There are windows between the earth-retaining walls and the beams of the basement. The exterior basement columns, which were captive along their weak axis, failed in shear. Columns and structural walls in other stories did not suffer damage, and there were moderately damaged masonry infill walls in the basement. Unlike other buildings, the damage rating of the building was based on the damage state of the basement rather than that of the ground floor
.

The columns of the rest of the dual-system buildings (without a basement) had no visible damage. The structural system of the other four-story building in Düzce was rated to be lightly damaged because of the observed beam damage. Masonry infill walls in the building were moderately damaged. There was no damage observed to the structural and nonstructural elements of the two-story school buildings in Düzce and Kaynaşlı and the three-story school building in Bolu. It was only 50 m away from (south of) the main surface rupture of the Düzce earthquake.

DAMAGE COMPARISON

One of the most significant structural deficiencies commonly observed in the school buildings was the

Fig. 6 Remains of the corner column of the ground floor of the building shown in Figure 5. The corner column failed in shear.

presence of captive columns formed by openings for the small windows in the masonry infill walls. There were at least two captive columns adjacent to the windows in the lavatories and around the stairwells in each school building. In addition, in seven of the schools in Bingöl, there was a furnace room on the ground floor level where the presence of small window openings adjacent to columns increased the number of captive ones from two to eight.

The observed difference in the performance of the dual systems and that of the moment-resisting frame systems cannot be attributed to defects arising from construction quality, which was similar and quite uniform across all of the surveyed school buildings. Visual inspection of the concrete revealed honeycombs and the inclusion of disproportionately large pieces of aggregate. Unwashed river aggregate had been used in most of the buildings inspected. There were detailing problems; the ends of the transverse reinforcement were typically not anchored in the concrete core, and sufficient confinement was not observed in the column end regions or in the beam-column joints. In general, ductile detailing was lacking in the structural members. Comparison of the performance of the dual and moment-resisting systems during the earthquakes has been organized on the basis of number of stories in the buildings:

• Two-story buildings: The two structures with dual systems showed no signs of visible damage. The moment-resisting frame structures survived without any damage to their columns. The displacement demand on these structures by the earthquakes was not high enough to damage the captive columns and the masonry walls severely. The masonry walls remained intact and contributed to the stiffness of the structures. In these buildings, there was moderate damage to the masonry infill walls.

• Three- and four-story buildings: An indication of the essential structural difference of the two types of buildings is reflected in the structural wall areas. The dual system structures had structural walls in 0.4% and 0.5% of the floor area in the long and short directions of the buildings, respectively. The three dual-system structures suffered almost no damage to their concrete structural walls. Of the three buildings, only one building was rated as severely damaged. Unlike other buildings in the inventory, it suffered damage to its basement. Of the 12 schools with moment-resisting frames as the lateral load resisting system, ten of them were either severely damaged or collapsed. Only two survived without any damage to the columns. The displacement demand was high enough to result in severe damage to the masonry infill walls. Damage to the masonry walls appears to have affected the structural response in two ways: (1) the stiffness of the system was reduced, and (2) crumbling of the masonry at the wall corners resulted in the formation of additional captive columns. These conditions could cause the buildings with moment-resisting frames to sustain heavy damage and, when the gravity-load carrying capacity is lost, to collapse.

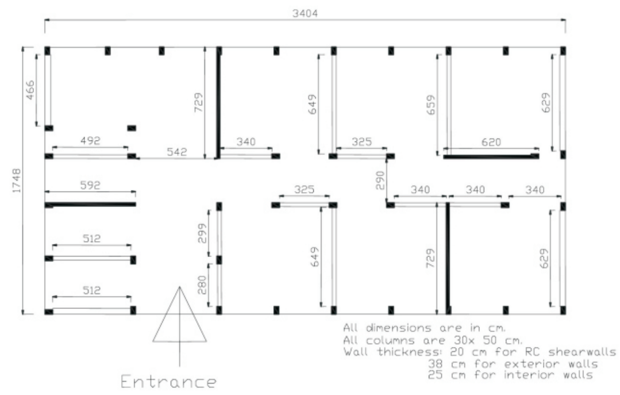

Fig. 7 Typical floor plan for the dual systems.

CONCLUSIONS

The damage survey of the school buildings in two earthquake areas in Turkey has re-emphasized a well-known principle of earthquake-resistant design. The collapse of multistory, reinforced concrete buildings with hollow clay-tile infill walls, typical of construction throughout Turkey, can be prevented by including a few properly located structural walls. Performance of the surveyed buildings with moment-resisting frames appears to be correlated with the number of floors in the buildings because the column size is uniform in all the schools. The two-story buildings could survive the earthquake without any damage to their columns. The three- and four-story buildings, however, did not perform satisfactorily. Of the 12 buildings with three stories or higher, ten buildings suffered shear damage to their columns. Three of them collapsed because of the columns failed in shear.

The efficacy of the structural walls to prevent building collapse is demonstrated by the fact that all school buildings in the inventory with dual-system frame structures, with the exception of one, were lightly damaged or not damaged at all. The sole severely damaged structure was damaged not by failure in the ground story, as all the other school buildings, but by failure of captive columns at basement level as a result of discontinuity of the foundation walls in height. The structural walls of the building, which were not damaged at all, prevented the collapse of the building by providing sufficient lateral strength and enhancing the gravity-load capacity. The observations in the school buildings showed that structural walls improve the behavior of reinforced concrete systems drastically. Accordingly, in school buildings, especially in those over two stories high, use of structural walls along with reinforced concrete frames is recommended. In all cases, captive columns should be avoided. These columns tend to fail in a brittle mode and may even lose gravity-load carrying capacity.

Summary of the paper "Building Seismic Loss Model for Tehran," by Babak Mansouri, Mohsen Ghafory-Ashtiany, Kambod Amini-Hosseini, Reza Nourjou, and Mehdi Mousavi, published in Earthquake Spectra, Vol 26, No.1, February 2010.

INTRODUCTION

The development of realistic urban inventories and more realistic vulnerability functions help in assessing seismic losses more accurately. Such studies are essential to a better understanding of the severity and the extent of the damage, the direct or indirect losses in the built environment, and consequently, human lives. It is notable that minimal research has been conducted in assessing the seismic urban risk for Iran.

The basic elements for conventional pre-event seismic loss estimation can be stated as the estimation of the surface ground motion, the compilation of the city inventory, the development of the vulnerability functions or structural fragility curves, and the implementation of a method in calculating and then evaluating the losses. A district of Tehran (Municipality District 17) is chosen as the area of interest, keeping in mind that the process is applicable for the entire region.

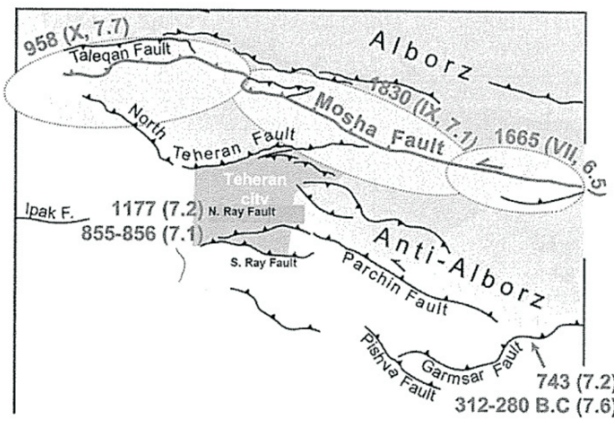

Fig. 8 Active fault map of central Alborz (after Berberian et al. 2001)

The vulnerability of this area was studied first by the Centre for Earthquake and Environmental Studies of Tehran and the Japan International Cooperation Agency in 2000, based on the data collected before the year 2000 in the form of city blocks. In this study, the city inventory has been updated with much higher resolution parcel-level details. The building inventory was compiled using high-resolution spatial and attribute data from cartographic aerial photos (digitally processed stereo photos) and city survey databases. Ancillary land use and building data were provided by an Iranian company. This inventory reflects field investigation with some limited descriptive attributes, e.g., "building structural typology" (categorized as steel, concrete or masonry), "number of stories," "building quality or age," and the associated land uses.

Ground motion data has been selected based on the available microzonation seismic hazard maps of Tehran developed by Jafari et al. (2005), reflecting the probabilistic contribution of all expected important earthquakes, attenuation relationships, and site effects for Tehran, and by scenario-based seismic hazard analysis of the region.

METHODOLOGY

The earthquake loss estimation procedure has been designed and implemented in GIS based on four main modules: Seismic Hazard, Building Inventory, Structural Vulnerability, and Risk Mapping, as described in the following sections.

HAZARD DATA - MICROZONATION MAPS

Active faults in the Tehran region have been investigated and mapped. A major source of earthquake hazard for Tehran is recognized as the "Mosha Fault" (an important fault in central Alborz with a length of 220 km) that is farther from the city but poses the highest level of hazard in Tehran. Figure 8 shows the map of active faults and the associated seismicity in central Alborz surrounding Tehran. "North Tehran Fault," with a length of 75 km, and the "South and North Ray Fault" (a possible earthquake source for the southern part of the city) are other important seismic sources for Tehran.

BUILDING INVENTORY

In 2000, JICA used the building data from 34,805 census blocks as provided by the Iranian Census Center. The database, predated 2000, was aggregated into 3,173 census zones. In this study, higher-resolution city data with parcel-level details, including the city topography and building height, have been used. The parcel maps and building height information were extracted from 1:2,000 scale digital maps provided by the National Cartographic Center (NCC) of Iran. These maps were created by processing aerial stereophotographs. The city parcel information has been processed and compiled from different data sets that needed both spatial adjustments and temporal-change considerations. Moreover, the data was complemented and corrected with a field survey.

The collected data have been classified based on "building type," "building age" and "building quality" as shown in Tables 1 and 2 and 3. The last two can be taken interchangeably, which means that buildings not older than ten years are regarded as high quality, while buildings constructed between ten to 30 years ago are considered low quality, and dwellings older than 30 years have the poorest quality. Tables 2 and 3 show the building stock categorization versus the number of stories and by the age/quality. The survey database includes some counts of unknown/unclassified structures, and these were filtered out for the computation process. Consequently, small differences exist between the total counts in Tables 2 and 3.

Table 1. Survey data classification for area of study (EMCO Iran co.)

Building	Masonry (brick and steel)
type	R/C frame
	Steel frame
Building	less or equal to 10 years
Age	between 10 to 30 years
	more than 30 years
Building	higher (newly built within last 10 years)
Quality	lower (repairable=10 to 30 years old)
	poorest (more than 30 years—must destroy)

Table 2. Building stock classification-structural typology versus height

Structural typology versus Height	1-story	2-story	3-story	4-story & up	Total count
Masonry (brick & steel)	4826	24874	3501	58	33259
R/C Frame	65	113	70	742	990
Steel Frame	98	467	1475	552	2592
Total count	4989	25454	5046	1352	36841

Table 3. Building stock classification-structural typology versus quality/age

Structural typology versus Quality/Age	Higher (<10 years)	Lower (10-30 years)	Poorest (>30 years)	Total count
Masonry (brick & steel)	120	4457	28687	33264
R/C frame	810	92	93	995
Steel frame	1354	1065	177	2596
Total count	2284	5614	28957	36855

STRUCTURAL VULNERABILITY FUNCTIONS

Despite the occurrence of many disastrous earthquakes in recent decades in Iran, very limited detailed spatial-loss data has been collected for the affected regions. Because of this shortcoming and because of the importance of pre-event loss estimation in disaster management, it seems inevitable that meaningful worldwide data be selected, modified or used together with existing domestic empirical or analytical results. Four approaches were used in deriving vulnerability curves that can best represent the existing building stock in Tehran. The curves used in Approach One are derived from the Manjil-Iran earthquake and other countries such as Turkey, with some modifications as reported by JICA (2000). Approach Two is based on damage surveys and expert judgment, as explained in the ATC-13 (1985) and ATC-13-1 (2002) reports where the results are downscaled for the nonstandard constructions (suggested by ATC-13 report). Approach Three uses the

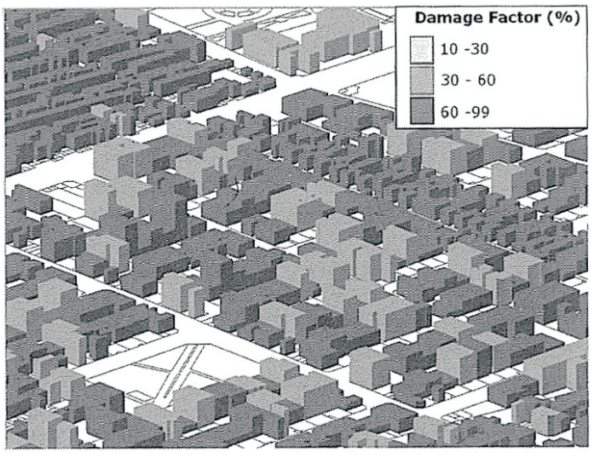

Fig. 9 3D building damage map of a region within the study area: Outcome of Approach Four and Ray fault earthquake scenario.

Costa Rican loss functions published by Sauter and Shah (1978) as a case that resembles the study area. Finally, in Approach Four, analytical fragility curves are created according to the HAZUS (Kircher et al. 1997) using the parameters judged for the local conditions of the area.

The most frequent building typology in southern Tehran is the traditional unreinforced bearing brick walls supporting steel beams with vaulted brick roofs and with an age of more than 30 years, which does not comply with seismic building codes, nor has it been retrofitted according to published expert recommendations.

CONCLUSION

Table 4 compares the outcomes for the four approaches, and they compare well. Two input microzonation maps, a probabilistic model and the Ray fault scenario, have been used as input hazard data for the seismic building loss estimation of Tehran. The database development, the vulnerability function derivation methodology, and the loss modeling were devised for Tehran. However, District 17 of Tehran was selected in order to compute the loss results. The existing building inventory was matched with different sets of building vulnerability functions. The building damage functions are extracted from four different approaches. All these vulnerability curves were matched, modified or derived for domestic parameters considering local conditions for Tehran. In order to visualize the results, the associated damage maps were created that represent parcels with their associated damage levels as seen, for example, in Figure 9. The total building loss is computed for the entire study area using all four approaches. The results show relatively a good level of agreement. For the probabilistic hazard map, the results show lower levels of damage as compared with the Ray fault scenario case.

These alarmingly high levels of building loss suggest that the study area is extremely susceptible and immediate measures must be taken to mitigate the effect of potential disastrous earthquakes. Building loss maps (similar to Figure 9) can be of great importance when greater Tehran, with all possible known earthquake scenarios, is considered and proper solutions to reduce the urban risk are kept in mind.

Table 4. Estimated total direct building loss for the entire building stock

Method/ Functions	Probabilistic Microzonation	Ray Scenario Microzonation
Approach One	61.1%	65.1%
Approach Two	66.3%	66.3%
Approach Three	68.2%	71%
Approach Four	68.4%	72%

Earthquake Hazard Centre

Promoting Earthquake-Resistant Construction in Developing Countries

The Centre is a non-profit organisation based at the School of Architecture, Victoria University of Wellington, New Zealand. It is supported financially by Robinson Seismic Ltd.

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