



# EARTHQUAKE HAZARD CENTRE NEWSLETTER

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## LESSONS FROM CANTERBURY EARTHQUAKE

On the 4th September 2010 the city of Christchurch, New Zealand, was struck by a shallow Magnitude 7.1 quake centred approximately 40 km from the central business district. The shaking itself caused light damage to buildings and houses, except those constructed from unreinforced masonry. Hundreds of brick chimneys fell onto roofs, and tens of both brick houses and commercial buildings suffered partial collapse. It was very encouraging to observe that previously seismically strengthened masonry buildings performed well, often with quite minor damage. In some cases the strength levels to which they had been strengthened were quite low. Reinforced concrete and steel framed buildings also performed well. Several buildings experienced shear cracks in columns and wall piers, but most survived the shaking with minimal damage. Among the steel buildings, some steel tension ties as part of tension-only cross-bracing in light industrial buildings yielded and a few connections failed.

The intensity of shaking was in the order of 70% of that used in the design of modern buildings. So it has to be said that this was not a real test of New Zealand design and construction approaches. New Zealand structural engineers are aware of several vulnerable areas of our building stock, and steps are underway to retrofit, but most of

these known potential problems, expected to manifest themselves in a Design Earthquake, remained hidden from the general public. Most of the damage to buildings not constructed from unreinforced masonry was due to liquefaction. Quite extensive areas, in some cases underlying recent housing subdivisions suffered uneven ground settlement due to a combination of liquefaction and lateral spreading. Many houses that had survived the shaking without structural distress were rendered uninhabitable by the often severe distortions caused by ground settlement. This damage has highlighted a weakness in our current house design and construction process. Although soil investigations are mandatory, they are not sophisticated enough to determine if a proposed site is prone to liquefaction. More rigorous geotechnical approaches will probably be required for future construction.

Although most buildings performed well due to good quality design, construction and quality assurance, there is no room for complacency. As already mentioned, this quake was smaller than the design event, and the lack of fatalities and relatively few injuries were due to the timing of the earthquake. By striking in the early morning hours (4.35 am) the vast majority of the area's inhabitants were sleeping inside their light-weight timber framed houses. A day-time or evening quake could have seen hundreds of fatalities due to people being crushed by falling brick facades and other elements. Although the Christchurch City Council has a policy of reducing the danger from buildings deemed to be seismically unsafe, it is clear that this programme needs to be continued, not only in Christchurch but also in all New Zealand cities and towns.

There are many other lessons to be learned from this quake. Apart from the reminder of the vulnerability of brick construction and building damage caused by liquefaction, researchers should be able to improve current mitigation of damage to non-structural elements and building contents. No doubt in the next year or so findings will enable Codes and Standards to be refined in an attempt to reduce future losses from the earthquakes that will surely strike.

## Virtual Site Visit No. 22 Foundations for buildings with seismic resistance

Particularly when the seismic load resisting system of a building consists of relatively short elements in plan, strong foundations are required. The structural system can consist of shear walls, braced steel frames or one bay moment frames with a short distance between columns. In each case, due to the limited distance between the end columns of the system, large tension and compression forces need to be resisted. When one of these vertical systems resists horizontal forces, mainly transferred from each floor and roof diaphragm, large overturning moments occur at its base. Without restraint against these moments the system will merely topple over and the building will collapse.

The building we are visiting on this site visit has short shear walls. To prevent their overturning the designers have designed strong foundations to make use of the whole width of the narrow site. First, piles have been cast approximately 15 m down to firm bearing to provide both compression and tension resistance. Reinforcement from the tops of these piles which is to be embedded into the foundation beams are shown in Figure 1. Then, interconnecting the piles and to transfer the very large tensions and compressions from the vertical chords of the two transverse shear walls, massive 3 m deep foundation beams are cast. In Figure 2 some of the reinforcement and the formwork for these beams can be seen. With such deep and



*Fig. 1 View of site with the piling operation nearing completion.*



*Fig. 2 Reinforcement and form work for the foundation beams.*



*Fig. 3 Completed foundation beams with the shear wall reinforcement projecting above, to the left.*

strong foundation beams all the piles along each side of the site can have their strengths mobilized to provide moment fixity at the base of the shear walls.

Due to the fact that the walls need to resist seismic loads and they are short, they have been designed for ductility. Therefore, in accordance with the principle of Capacity Design, a plastic hinge is expected to form at the base of each wall before the wall suffers shear failure or before the foundations are damaged. Consequently the foundations are designed to be stronger than the base of the wall so after a damaging earthquake there will be no need to repair the foundation system. This requirement leads to the necessity for even stronger foundations.

# **SUMMARY OF; “OBSERVED SEISMIC BEHAVIOR OF BUILDINGS IN NORTHERN PAKISTAN DURING THE 2005 KASHMIR EARTHQUAKE”**

By Amjad Naseer, Akhtar Naeem Khan, Zakir Hussain, and Qaisar Ali. From *Earthquake Spectra*, Vol. 26, No. 2, May 2010.

## **INTRODUCTION**

On 8 October 2005, an earthquake of magnitude  $M_w=7.6$  struck the northeastern part of the North-West Frontier Province (NWFP) of Pakistan and southwestern part of Kashmir. It caused extensive damage to manmade infrastructure over an area of 30,000 sq. Km. Balakot and Muzaffarad were the worst-hit cities because of their proximity to the epicenter of the earthquake. Masonry buildings constructed with stones, concrete blocks, and fired-clay bricks and concrete buildings were partially or severely damaged. The Kashmir earthquake resulted in more than 73,000 casualties, while 80,000 people were injured, and more than 3 million people were left homeless.

## **EFFECT OF EARTHQUAKES ON BUILT ENVIRONMENT**

The buildings which were partially or fully damaged by this earthquake were estimated at about 450,000 and includes buildings constructed with reinforced concrete and unreinforced stone, concrete block, and brick masonry. Buildings constructed with rubble stone masonry suffered the heaviest damage followed by concrete block masonry. Almost 95% of the stone masonry buildings in the earthquake-affected region were severely damaged. Damaged block masonry buildings were estimated to be 60–65%. Brick masonry buildings performed comparatively better with only 5% buildings collapsed and 35% were severely damaged. In reinforced concrete buildings, 50–60% were partially or severely damaged. In the following sections, damage to both reinforced concrete buildings and their causes are discussed.

## **REINFORCED CONCRETE BUILDINGS**

Reinforced concrete buildings (RC) could be categorized as semi-engineered because they were usually designed for gravity loads only. The causes of failure could be attributed to a combination of the poor design and construction practices. Lack of proper confinement in columns,

insufficient lap length, strong beam-weak column, short column, and soft story were among the poor design practices observed from the damaged buildings. Poor construction practices, including improper compaction of concrete, construction joint, and notching of concrete column for door and window lintel beam, were also observed to be responsible for damage to concrete buildings.

## **POOR DETAILING PRACTICE**

Because of the absence of a national seismic building code and its effective enforcement, buildings were poorly designed and detailed. This includes the lack of proper confinement in the column, insufficient lap splice length and its location in the beam and column, use of plain bars, and stirrups and ties with improper hooks. The ties in the column were uniformly provided at a spacing of 300 mm (12 in.) or more, irrespective of the structural demand or code requirements; according to UBC 97 code, for moment resisting frames in seismic Zones 3 and 4, the maximum tie spacing should be the minimum of either one-quarter minimum member dimension or 100 mm (4 in.). The lap splice in the columns, 240–720 mm (6–18 in.) in length, was normally provided at the base of the floor. However, according to UBC 97, the lap splice for concrete frames located in seismic Zones 3 and 4 should be provided within the center half of the column and should be proportioned as a tension splice. Because of the lack of confinement at the highly stressed zones (near the beam-column joint) and insufficient lap lengths, stresses in the reinforcing bars could not be developed which consequently resulted in the failure. These detailing deficiencies have been observed in almost 90–95% of reinforced concrete buildings in the cities of Abbottabad, Mansehra, Balakot, and Muzaffarabad.

## **STRONG BEAM-WEAK COLUMN**

The least dimension of the column was commonly selected as equal to the dimension of the infill masonry unit (230 mm (9 in.) with fired clay brick and 203 mm (8 in.) with concrete block) so that the column would not project outward from the wall. The desire to have maximum space in the horizontal direction usually resulted in fewer columns with smaller cross sections. Also, most of the concrete structures were designed for gravity loads only. Probably because of these reasons, buildings with strong beams and weak columns were constructed. The weak nonductile columns suffered damage because of the lateral seismic demand, which could further be aggravated by axial forces from the elastic beams. The example of buildings damaged having strong beams and weak columns is illustrated in Figure 4.



## SOFT-STORY EFFECT

The soft-story effect was more evident in commercial buildings than residential because the ground floor typically has more open spaces for shops and parking vehicles. This type of structural deficiency puts more deformation demand on the flexible lower story columns and because of the insufficient energy dissipation capacity of the columns, it resulted in the failure of lower stories.

## CONSTRUCTION/WORKMANSHIP PROBLEM

The construction-related problems, including improper compaction of concrete and improper treatment of construction joints, could not be ruled out as causes of failure in reinforced concrete buildings. Vibrators are rarely used for the compaction of concrete in columns. The concrete strength estimated by Schmidt hammer test during the reconnaissance survey in both private and public buildings rarely exceeded 13.75 MPa (2000 psi).

## MASONRY BUILDINGS

### IN-PLANE DAMAGE

#### Diagonal Shear Failure of Wall Piers

For heavily loaded masonry wall piers with low aspect ratios, in-plane shear forces of the wall causes diagonal shear failure in the form of X-cracks. This type of failure does not endanger the gravity load-carrying capacity of a wall unless cracking becomes severe or out-of-plane movement takes place. The diagonal shear failure is also associated with in-plane shear sliding of the masonry wall. Figure 5 shows building walls cracked in diagonal shear during the Kashmir earthquake. This type of failure could be seen throughout the affected area.

#### Flexure Failure of Pier

Piers with large aspect ratios fail in flexure under alternating



*Fig. 4 Collapsed building due to strong beams and weak columns.*



*Fig. 5 Diagonal shear failures of masonry wall piers.*

bending moments caused by the cyclic nature of the seismic forces. This type of failure manifests itself in the form of horizontal cracks at the top and bottom ends of the pier. The cracked pier moves as a rigid body having no lateral-load resisting capacity.

## OUT-OF-PLANE DAMAGE

### Lateral Thrust from Inclined Roofs

Walls supporting an inclined roof experience lateral thrusts in their out-of-plane directions.

During strong ground shaking produced by an earthquake, this thrust may be large enough to cause the walls to collapse in the out-of-plane direction or get seriously damaged. In the absence of reinforced concrete beams at the roof level, the building fails to produce a boxtype behavior resulting in greater vulnerability of masonry walls in the out-of-plane direction.

### Failure of Building Corners

The corners of walls supporting roofs inclined in both directions are damaged due to the lateral thrust applied by the roof in addition to the inertial forces. The in-plane rotation of a rigid diaphragm also induces this type of failures at the corners. The lack of proper connection between walls and between walls and floor make the corners more vulnerable to cracking. Figure 5 illustrate this type of failure.

### Separation of Orthogonal Walls

Due to the lack of connection between orthogonal walls, separation of walls occurs. The resistance offered by the in-plane walls to the bending of out-of plane walls depends on the tensile strength of masonry. Separation of orthogonal walls occurs whenever this tensile strength is exceeded. Shear stresses due to flange-action make the wall intersections more susceptible to cracking.



*Fig. 6 Damaged corner of a building in Mazaffarabad.*

#### **Damage at Walls Adjacent to Roof**

In many buildings, ventilators are constructed adjacent to the floor slab with small length of wall in between them. Because of the out-of-plane vibration, these short piers get severely damaged. In some instances the piers completely collapse and the floor settles down.

#### **Collapse of External Veneer of Masonry Walls**

Rubble stone masonry constructed in weak mortar and without through stones is liable to fail at a very low level of seismic excitation. The external veneer usually fails first in the out of plane direction. This type of failure (Figure 7) was prominent in the rural parts of the affected area.

#### **Out-of-Plane Failure of Gables**

The collapse of gable walls was observed in many buildings of Muzaffarabad. Absence of vertical loads and adequate connection between gable wall and roof, the gable walls are vulnerable to failure in the out-of-plane direction.

### **NON-STRUCTURAL MASONRY ELEMENTS**

#### **Out-of-Plane Failure of Infill Walls**

Brick or block masonry have been commonly used in the construction of infill walls in concrete framed buildings. After the construction of columns and beams, masonry infill panels are constructed with no separation between walls and columns while the gap between walls and beams are filled with mortar. Due to the lack of proper connection between walls and beams/columns, the infill walls are more susceptible to out of plane failure even for low level of ground shaking.

#### **In-Plane Failure of Infill Walls**

The concrete-framed buildings have typically been analyzed and designed as skeletal structures without any masonry infill walls. Buildings are constructed in such a manner that no separation is left between walls and beams/columns,

result in a composite structure in which both concrete frame and infill masonry walls resist together the seismic demand of the earthquake ground motion. Consequently, the infill walls fail in in-plane action and redistribution of loads occur, putting demand on the columns. The infill walls of many concrete framed buildings were found damaged in diagonal shear throughout the affected area. Crushing of masonry unit (concrete blocks) was also observed.

### **FAILURES OF APPENDAGES SUPPORTED ON UNREINFORCED MASONRY**

It has been a routine practice to build large water tanks at the top of buildings, and they are supported on vertical masonry elements. In concrete-framed structures, the water tanks are mostly simply supported on concrete columns. During ground shaking, supporting elements could not resist the increased structural demand resulting from inertial forces of the large mass of water. Consequently, dislocated water tanks severely damaged the buildings.

### **FAILURE OF BOUNDARY AND PARAPET WALLS**

Normally, boundary and parapet walls are constructed of single-leaf masonry wall. Boundary walls are 1.5 to 2.0 m (5 to 7 ft) in height, whereas parapet walls are 0.75 to 1.5 (2.5 to 5 ft) high in the residential buildings of the affected area. The boundary and parapet walls throughout the affected area were either collapsed or severely damaged.

### **CONCLUSIONS**

Despite the fact that Pakistan is situated in a seismically active region, no serious efforts had been undertaken for the assessment of seismic hazard and development of a seismic building code prior to the Kashmir earthquake. Buildings designed according to 1937 Quetta building code performed well during the 1941 earthquake. The 1986 Pakistan building code did not address the analysis and design of either concrete or masonry structures for lateral loads. Seismic



*Fig. 7 Collapse of external veneer of stone masonry wall.*

zoning maps given in the code underestimated the seismic hazard of the area affected by the Kashmir earthquake. The 2007 Pakistan Building Code, developed after the Kashmir earthquake, is mostly based on the 1997 Uniform Building Code. Indigenous research is required to evaluate analysis and design parameters specific to materials and structural systems used in Pakistan.

It was observed during the reconnaissance surveys that most of the buildings were non-engineered or semi-engineered. The stone masonry buildings that comprised 30–35% of the building stock were severely damaged because the use of rubble stones, mud mortar, or weak cement-sand mortar (1:8 or 1:10) and lack of connection between walls and walls and floor.

Concrete block masonry buildings were the second most damaged buildings. The damage could be attributed to use of very low strength blocks, lack of confinement of unreinforced block masonry wall panels and poor workmanship. Brick masonry buildings performed relatively well due to better workmanship and material properties. However, the damage in brick masonry buildings could be attributed to the lack of confinement of wall panels and poor configuration.

Reinforced concrete buildings were mostly designed for gravity loads. However, it was observed that reinforcement detailing did not even comply with code requirements for gravity load design. Improper confinement of the column joints, insufficient lap splice length and lap location, soft story effect, and the use of extremely weak concrete attributed to the damage observed.

It is important to carry out risk and loss assessment of the major cities. Low-cost retrofitting guidelines should be developed. Lastly, a strategy for the enforcement of a seismic code throughout Pakistan should be devised.

## **SUMMARY OF “LEARNING FROM EARTHQUAKES- THE M<sub>w</sub> 8.8 CHILE EARTHQUAKE OF FEBRUARY 27, 2010”.** From, EERI Special Earthquake Report—June 2010

### **INTRODUCTION**

On Saturday, February 27, 2010, at 03:34 a.m. local time an M<sub>w</sub> 8.8 earthquake struck the central south region of Chile, affecting an area with a population exceeding eight million people, including 6.1M, 0.8M, and 0.9M in the urban areas around Santiago, Valparaíso/ Viña del Mar, and Concepción, respectively.

As of May 2010, the number of confirmed deaths stood at 521, with 56 persons still missing. The earthquake and tsunami destroyed over 81,000 dwelling units and caused major damage to another 109,000. According to unconfirmed estimates, 50 multi-story reinforced concrete buildings were severely damaged, and four collapsed partially or totally. The earthquake caused damage to highways, railroads, ports, and airports due to ground shaking and liquefaction.

The earthquake was followed by a blackout that affected most of the population, with power outages affecting selected regions for days. Estimates of economic damage are around \$30 billion.

### **BUILDINGS**

Earthquake shaking caused extensive damage to many non-engineered and engineered buildings throughout the affected area. The team focused on concrete, masonry, and adobe construction, as this constitutes the vast majority of buildings. Some damage to steel buildings also was observed, but is not reported here.

The region contains a large number of older houses, churches, and other buildings constructed of adobe or unreinforced masonry. Seismic resistance typically is provided by walls located around the perimeter and, to a lesser extent, at the interior.

Absence of reinforcement and weak connections between adjoining walls apparently led to the collapse of walls and roofs in many buildings, contributing to some human fatalities. In addition, delamination of exterior stucco, while not jeopardizing the structural system, created the appearance of significant damage in many other buildings.

Confined masonry construction is also widely used for



buildings one to four stories tall. Exterior walls of clay bricks are first constructed on a concrete foundation and then reinforced concrete confining elements are cast around the brick walls, forming a tight bond between the masonry and concrete elements. These buildings generally performed very well; typical damage (where observed) included diagonal cracking of masonry walls and wall failure due to lack of confining elements around openings or poor quality of the confinements.

The vast majority of mid- to high-rise buildings in Chile are constructed of reinforced concrete. Most of these rely on structural walls to resist both gravity and earthquake loads; some more recent construction uses a dual system of walls and frames. A typical high-rise plan has corridor walls centered on the longitudinal axis, with transverse walls framing from the corridor to the building exterior. Typical ratios of wall to floor areas are relatively high compared with concrete building construction in the U.S. In 1996, Chile adopted a seismic code with analysis procedures similar to those in UBC-97, but there are no prohibitions or penalties related to vertical or horizontal system irregularities. NCh433-1996 also enforces provisions of ACI 318-95; however, in light of good building performance in the March 1985 earthquake, it was not required to provide closely spaced transverse reinforcement around wall vertical boundary bars.

Starting in 2008, the new Chilean Reinforced Concrete Code does require use of boundary elements. An apparent trend is to use thinner walls in recent years than in the past.

Figure 8 shows typical damage to a transverse wall in the first story of a ten-story building in Viña del Mar. Note the setback in the wall profile at this level, provided mainly to accommodate automobile access to parking spaces. This condition was observed in several buildings; in buildings with subterranean parking, this damage was likely



*Fig. 8. Wall damage, Vina del Mar (photo: P. Bonelli)*



*Fig. 9. Nonstructural damage in the Talca Supreme Court building constructed in 2003 (photo: E. Miranda)*

in the first level below grade.

Figure 9 shows a failed wall from a subterranean level of a 12-story building in which large steel pipe columns were being used to raise the building to enable repairs. Given the wide spacing of transverse reinforcement, there was little bearing section remaining in the thin wall and the entire wall section buckled laterally. In this example the longitudinal bars buckled without fracture; in many other examples the longitudinal bars were fractured. It was reported that, even though not required by the local building code, some engineers used transverse reinforcement conforming to the ACI Building Code. The team did not observe that type of reinforcement in any damaged buildings.

Several of the severely damaged mid- to high-rise buildings had permanent offsets at the roof, apparently due to subsidence of walls, raising questions about repairability. Four concrete buildings collapsed completely or partially. Two of these were nearly identical, proximate, five-story buildings in Maipú, Santiago. These buildings had four stories of condominium units atop a first-story parking level with a highly irregular wall layout. Wall failure likely contributed to the collapses.



*Fig. 10. Nonstructural damage at the Santiago International Airport terminal (photo: E. Miranda)*

## NON-STRUCTURAL COMPONENTS AND SYSTEMS

There was extensive non-structural damage in practically all types of buildings — residential, commercial, and industrial. Commonly observed was damage to glazing, ceilings, fire sprinkler systems, piping systems, elevators, partitions, air handling units, and cable trays. The widespread non-structural damage caused significant economic loss and major disruption to the normal functioning of Chilean society.

Non-structural damage resulted in the closure of the international airports in Santiago and Concepción, which together handle more than two thirds of the air traffic in Chile (Fig. 10).

This earthquake illustrates the importance of improving seismic performance of non-structural components, the failure of which can lead to injuries, loss of functionality, and substantial economic losses. This is especially important for critical facilities such as hospitals, airports, and water distribution systems.

## HOSPITALS AND HEALTH CARE

The 130 hospitals in the six regions affected by the earthquake account for 71% of all public hospitals in Chile. The Chilean Ministry of Health (MINSAL) found that of these, four hospitals became uninhabitable, twelve had greater than 75% loss of function, eight were operating only partially after the main shock, and 62% needed repairs or replacement. Of the beds in public hospitals, 18% continued to be out of service one month after the earthquake. MINSAL estimates the damage at \$2.8B, and expects the replacement of severely damaged hospitals to take three to four years.

Although structural damage was minimal in hospitals, most suffered non-structural damage, and frequently, loss of utilities. All hospitals in the study area lost municipal

electrical power and communication for several days, and 71% lost their municipal water supplies. All hospitals were equipped with backup power and water supplies, but such redundancy was not present in their communication system, creating enormous difficulties for aid coordination.

Additionally, most hospitals reported damage to their suspended ceilings, cracking of the plaster over brick walls, and partition damage. The collapse of ceilings and associated light fixtures and mechanical grills discomfited occupants and caused unsanitary conditions that led to many evacuations.

The team visited three seismically isolated hospital buildings in Santiago; none was damaged other than at joints with adjacent buildings or other structures. In two cases, immediately adjacent fixed base buildings had moderate non-structural damage.

## 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.

Director (honorary) and Editor: Andrew Charleson,  
ME.(Civil)(Dist), MIPENZ  
Research Assistant: Samantha McGavock, BAS

Mail: Earthquake Hazard Centre, School of Architecture,  
PO Box 600, Wellington, New Zealand.  
Location: 139 Vivian Street, Wellington.  
Phone +64-4-463 6200 Fax +64-4-463 6204  
E-mail: [quake@arch.vuw.ac.nz](mailto:quake@arch.vuw.ac.nz)

The Earthquake Hazard Centre Webpage is at :  
<http://www.vuw.ac.nz/architecture/research/ehc/>