

## GENERAL OBSERVATIONS OF EFFECTS OF THE 30th SEPTEMBER 2009 PADANG EARTHQUAKE, INDONESIA

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### SUMMARY

The Mw 7.5 Padang earthquake struck at 17:16 local time on 30th September 2009 with an epicentre offshore about 60 km west-northwest of Padang, capital of West Sumatra Province. More than 1,100 people were killed, and over 2,900 injured. The earthquake caused significant damage to public buildings and offices as well as to about 140,000 houses. It affected 250,000 families through the total or partial loss of their homes and livelihoods. More than half the earthquake fatalities occurred when several villages inland from Pariaman were buried by landslides. However, the damage and destruction of building structures was a major cause behind human and property losses. In addition to landslides, the earthquake triggered extensive liquefaction and lateral spreading in the region. A ten-member team from New Zealand visited the area under the auspices of NZAid and New Zealand Society for Earthquake Engineering to undertake building safety evaluations. The team spent most of their time in Padang city and other nearby earthquake-affected areas. This paper presents their observations and explores causes behind the damage and destruction of buildings by the moderate to strong earthquake shaking.

### 1. INTRODUCTION

The 2009 Padang earthquake occurred not far off the western coast of Sumatra, between Padang City and the coastal town of Pariaman (Figure 1.1). The major shock hit at 17:16:09 local time on September 30, 2009 (10:16:09 UTC, September 30). It registered a moment magnitude ( $M_w$ ) of 7.5 (USGS, 2009), making it similar in size to the 1906 San Francisco earthquake, the 1935 Quetta earthquake, the 2001 Gujarat earthquake, and the 2005 Kashmir earthquake. The epicentre was 60 km west-north-west of Padang and its focus was at a depth of 81 km. More than 1,100 people were killed, and over 2,900 injured (Antara, 2009).

The earthquake caused widespread building damage from shaking, as well as from earthquake-induced landslides, liquefaction, and lateral spreading. The overwhelming majority of the buildings that were damaged or destroyed in Padang city had reinforced concrete frames with unreinforced brick infill panels, some were unreinforced masonry, and a few were steel building structures. The field observations

indicate that much of the structural damage in this event has occurred in engineered structures, at least in Padang city. This is one of several earthquakes in Indonesia which has caused significant damage to engineered structures. It is important to note that even newer buildings constructed in concrete or steel suffered catastrophic destruction. Some important lessons can be learned from observations of the performance of these buildings.

Structurally deficient buildings are the cause of the catastrophe. The buildings were generally weak, because they were either constructed of weak materials, or were mostly likely constructed very poorly. For large buildings, inferior vertical configuration, short column effects, deficient structure sections and a lack of ductile detailing were the major causes behind the destruction. Even though there has been a history of earthquakes, and recent studies show the potential for major earthquakes in the region (Figure 1.2), the buildings clearly lacked earthquake-resistant features.

A team of ten New Zealand earthquake engineers travelled to Padang in October 2009 to undertake the assessment of

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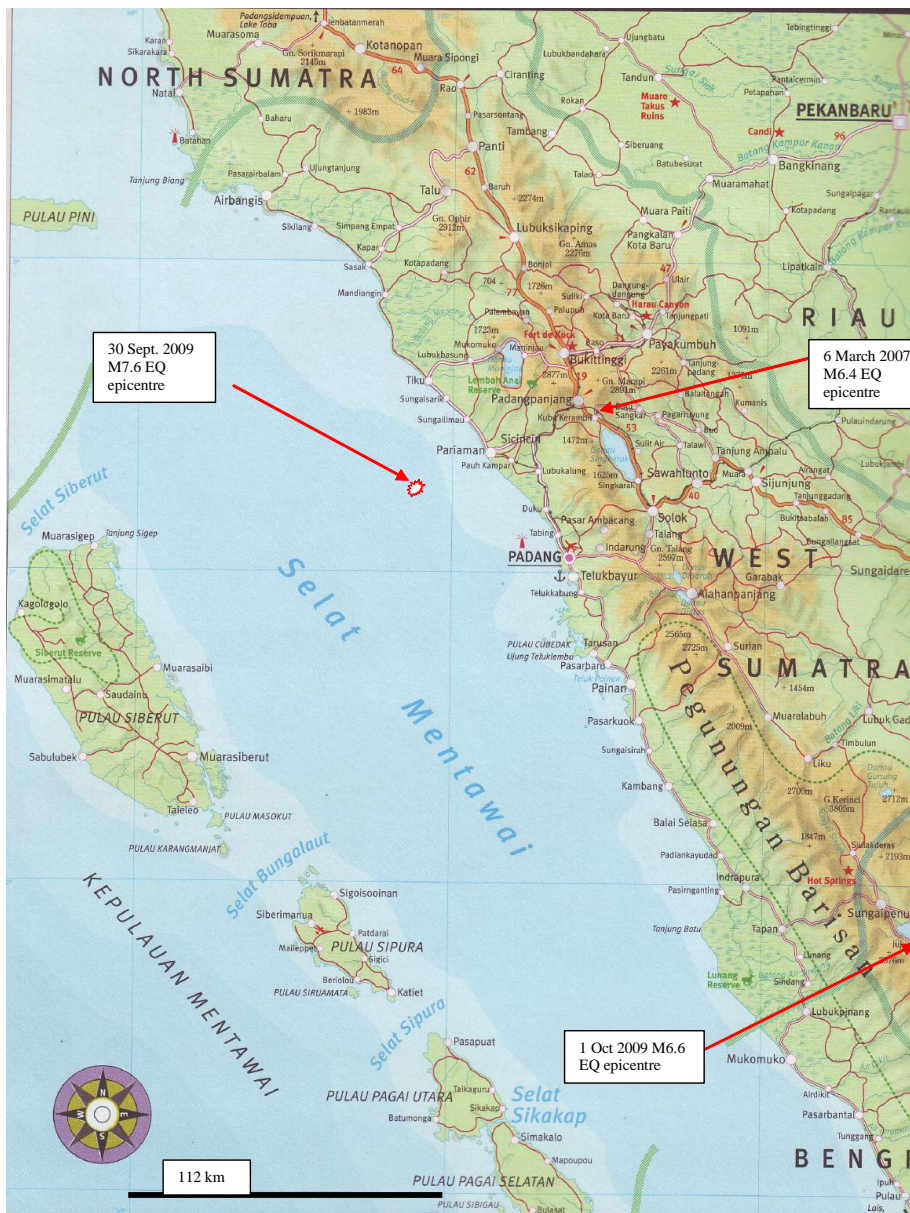
earthquake-damaged buildings. The assessment was focused on public buildings and public-accessed buildings in Padang and surrounding areas. The team, which consisted of all the authors of this paper, worked in the area from 13 October to 24 October, 2009. This paper is based on observations of buildings in Padang city, where the team mostly worked, and on limited observation of building damage outside Padang city.

An accompanying paper (Brunsdon *et al.*, 2010) outlines the deployment process, the arrangements that the team operated under in Padang, the tasks undertaken and the outputs and outcomes achieved. Lessons for steel structures are outlined in an additional paper by Hyland and Wijanto (2010). Our paper presents the observed performance of building structures in the earthquake-affected areas and analyses the reasons behind the catastrophe.

## 2. THE EARTHQUAKE

### 2.1 History of Earthquakes in the Region

Figure 1.2 summarises the known large earthquakes in the region. The high level of seismicity, regular large magnitude earthquakes—typically megathrusts with associated tsunami, and active volcanism along the Sumatran arc are associated with the subduction of the Indo-Australian plate beneath the overriding Sunda plate. Given the relative oblique plate motion rate of 50 to 57 mm per year (Figure 1.2), combined with locking of the plate interface, megathrust earthquakes are likely to occur every few hundred years and this is what the historic earthquake record indicates. Historical earthquakes in 1779 and 1833 of ~M8.7 and M8.8 respectively, along the subduction interface offshore from Padang, uplifted the Mentawai Islands and each created 5 to 10 m tsunami which



**Figure 1.1:** Relief map of the West Sumatra earthquake area showing the offshore Mentawai Islands, Padang City, local towns and the locations of other recent earthquakes. The lineament of the Great Sumatran Fault can be followed from Sungaipenuh in the south, through Liku, Muaralabuh, Surian, Solok, Lake Singkarak, Padangpanjang, Bukittinggi, Lubuksikaping and Panti. For scale, the grid lines are 112 km squares.

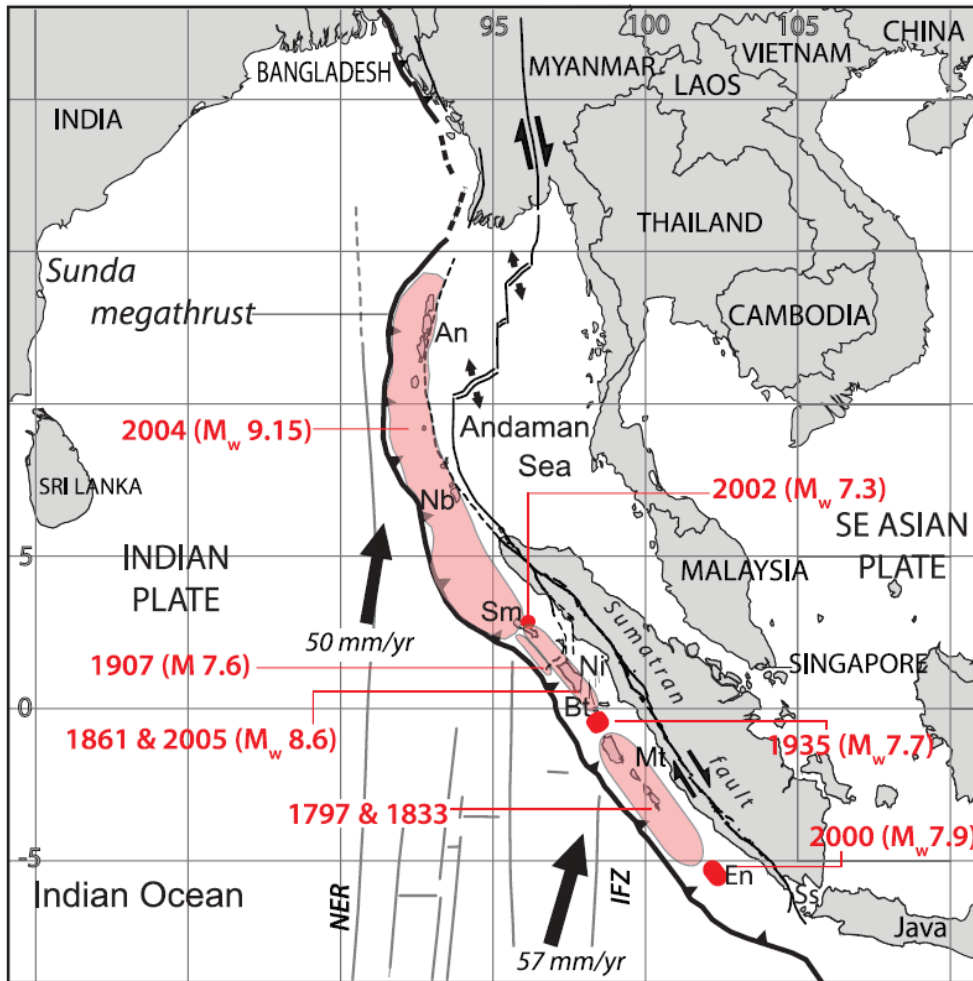


Figure 1.2: Historical earthquakes in the region (from Natawidjaja *et al.*, 2006).

swept inland up to 3 km (Natawidjaja *et al.*, 2006). As the subduction interface in this area currently shows a seismic gap and thus appears to be locked, a megathrust earthquake followed by a tsunami is predicted to have a high probability of occurrence in the next 30 years. Considerable effort has therefore gone into public education and evacuation routes for tsunami. Consequently, when the September 30 earthquake occurred, the public response was to evacuate, resulting in chaos as tens to hundreds of thousands of people took, in many cases, to rubble strewn lanes and roads on foot, motorbikes and cars. To compound the chaotic situation, some taller buildings that were set aside for vertical tsunami evacuation were damaged by the earthquake and some collapsed.

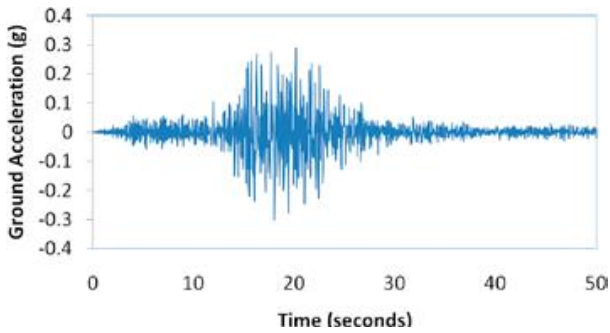
## 2.2 Seismicity and Ground Motions of the Earthquake

Earthquakes are common all along the megathrust plate interface between the Indo-Australian and Sunda plates. The December 2004, M9.15 Aceh and the March 2005, M8.6 Nias earthquakes suggested that the Sunda megathrust may be “unzipping” southwards, heightening concerns that a large magnitude megathrust earthquake with tsunami could occur on the “Padang” segment, which has not experienced megathrust earthquakes since 1797 and 1833. However, the Mw7.6 earthquake on 30 September 2009 was not a megathrust event and it did not generate a tsunami. It was located at a depth of 81 km, whereas the plate interface is located at 50 km depth at

that point. It was within the descending oceanic slab of the Indo-Australia plate and its epicentre was located offshore about 60 km WNW of Padang. The rupture zone of the earthquake is small and roughly circular, with a radius of about 15 km. Similar to other intra-slab high stress drop earthquakes, it is a result of the brittle rupture of relatively strong oceanic slab rock and it produced a predominance of high-frequency ground motions (EERI, 2009). However, its focal mechanism is unusual, indicating high-angle oblique thrust faulting due to internal buckling and compression of the descending oceanic lithosphere. A maximum slip of 9 m is indicated at the source, as well as strongly radiated energy with very few aftershocks, as is typical of sub-crustal earthquakes. No significant aftershocks were felt by the NZ Team, and the lack of aftershocks made the inspection of damaged buildings less risky.

## 2.3 Strong Motion Records and Intensity Distribution

There is only one strong ground motion record from the region (BMKG/USGS, 2009), which shows about 20 seconds of strong shaking with a peak ground acceleration (PGA) of 0.3 g (Figure 2.1). The instrument site was located at the base of the mountains, about 12 km inland from the coast and on stiff soil (EERI, 2009).



**Figure 2.1:** Ground acceleration (N-S component) (EERI, 2009).

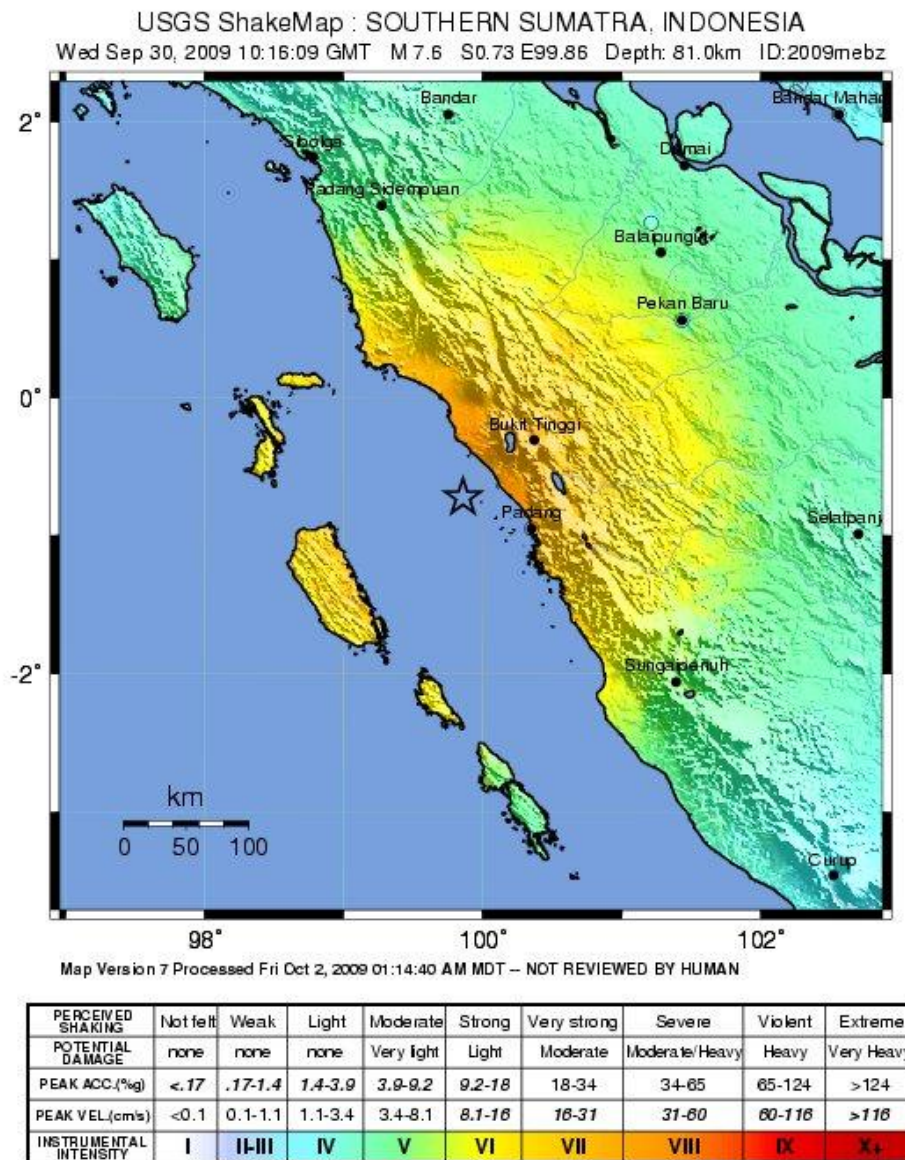
Amplification of ground shaking was observed in soft soils in many areas of Padang City, such as near river banks and in the old Chinatown district. Modified Mercalli Intensity damage assessments made by the International Team, which carried out a damage survey of 4,000 buildings in Padang immediately following the NZ Team, indicate a greater level of damage to structures in areas of soft ground where there

was liquefaction, ground settlement (often differential) and lateral spreading. The EERI report (2009) suggests that the ground accelerations shown in Figure 2.1 would have been significantly stronger on the deep and softer soils in the Padang City area

As observed in the field, the earthquake shaking effects were clearly directional. The stronger shaking direction was East-West, as was evidenced by soft storey collapse of buildings with an open bottom storey along E-W direction streets, whereas similar buildings with an open front along N-S direction streets suffered less damage and did not collapse.

**2.4 The Earthquake-Affected Areas**

The initial USGS Shake Map (Figure 2.2) showed that the 30 September earthquake was expected to be widely felt, with damaging shaking of MM6 to 7 spread over a wide area, not only along the Padang – Pariaman coastal strip, but also from the offshore Mentawi Islands in the west to inland Sumatra (Pekan Baru) in the east. However, on-ground surveys indicate that the earthquake energy was not uniformly spread as shown



**Figure 2.2:** The USGS Shake Map shows the earthquake epicentre and gives a derived Intensity of MM8 in Padang.

in Figure 2.2. Rather the energy was strongly directed to the Pariaman and Padang City areas east of the epicentre—here the intensity reached as high as MM9, causing extensive environmental and building damage in these areas, but little damage beyond. As well as the damaged structures in Padang City and Pariaman District, there were numerous examples of liquefaction, lateral spreading, ground settlement and large landslides. In Pariaman District, large earth-flow landslides triggered by the earthquake buried several villages, killing more than 600 inhabitants, thereby causing more than half of the 1,100 fatalities due to the earthquake.

There were no reports of damage or injuries on the Mentawai Islands to the west, while the NZ team did not observe any damage along Lake Singkarak, in the towns of Solok and Padangpanjang, or in the large inland city of Bukittinggi, where the indicated intensity was approximately MM4 or less.

### 3. GEOTECHNICAL ISSUES

#### 3.1 Liquefaction and Lateral Spreading of Soil

The geomorphology of the Padang City area gives a good indication of its likely response to earthquakes. Over the very recent geological past, the coastal plain on which Padang city and its surrounds is located has been built up of accumulated sediments eroded from the volcanic plateau and cones inland to the east. The rivers from the hills have meandered across the flat coastal plain, depositing loose, soft sediments in swampy areas behind prominent coastal beach ridges. Before European colonisation in the 1700s, the sophisticated and colourful Minangkabau people of West Sumatra Province mainly populated the fertile, cooler climate area of the inland volcanic plateau around the city of Bukittinggi, with only small fishing villages on the coastal plain. However, the Europeans in the later 1700s bought trade and the development of natural resources, such as mining, which required a port and a coastal town. Thus Padang city was born on the banks of the Arau River.

Since Indonesia's independence in 1949, Padang has grown rapidly from a town of 50,000 people to a city of 1 million today. To accommodate this growth the city has spread over large areas of low-lying coastal land, much of which was formerly swampy and consists of soft and weak soils, such as sands, silts and muds. As well as offering challenging foundation conditions for houses and larger city buildings, the low-lying coastal plains are highly susceptible to tsunami inundation, with the 5 m height contour as much as 3 km inland from the sea.

Soil settlement and liquefaction caused extensive damage in the September earthquake (Figures 3.1–3.5). In many cases building foundations were distorted due to lateral movements and differential settlements, so that the buildings will need to be rebuilt. Much of the land on which Padang is built would be classified under NZS 1170.5 (2002) as ground classes D (deep or soft soil sites) and E (very soft soil sites), while some of the better areas may be ground class C (shallow soil sites). In New Zealand these ground class soils would require specific site investigations and design, and may even require a site-specific seismic hazard assessment for important structures (such as hospitals and schools). Enquiries during the NZ Team visit indicate that site investigations for large buildings, other than hand-held Scala penetrometer tests, are rarely carried out and that the deepest foundations for any building are unlikely to exceed a “standard” 5 m depth caisson. This type of site investigation and foundation design is unsuitable for the predominant ground conditions in the city of Padang, where much of the observed earthquake damage to both houses and to larger buildings was exacerbated by

differential ground movements associated with liquefaction and lateral spreading. In order to “build back better”, not only should the earthquake-resistant design of structures and the quality of their construction improve, but there also should be greatly improved site investigations followed by a sound foundation design.



**Figure 3.1:** Settlement damage to building steps.



**Figure 3.2:** Settlement damage to a large building due to liquefaction and expulsion of water and fine sand. Buildings on sand strata near drains, creeks and rivers suffered from failure due to lateral spreading of soil. In one case half the length of a classroom building had dropped 800 mm compared to the other half.



**Figure 3.3:** Lateral spreading and settlement of the present-day beach ridge. The ocean is to the left and a lagoon area to the right.



**Figure 3.4:** Settlement of ground surrounding a building due to lateral movement and rotation of a retaining wall.



**Figure 3.5:** Differential settlement of foundation and floors of the Padang Bus Terminal has strained and deformed the entire building.

### 3.2 Landslides and Rockslides

Earthquake-induced landslides were common on the steep slopes that lead from the coastal plain to the inland volcanic plateau and they caused damage to the roads from Padang to Solok and the northern route from Sicincin to Padangpanjang. Susceptible steep escarpments around Lake Maninjau caldera collapsed and these had failed previously in the M6.4 “West Sumatra Earthquake” in 2007, which had its epicentre on the Great Sumatran Fault at the north end of Lake Singkarak.

The environmental descriptions of the NZ Modified Mercalli Intensity scale have liquefaction, lateral spreading and landsliding commencing at MM7 and becoming increasingly more severe with increasing intensity above this. The landslides and rockfalls along steep susceptible escarpments, such as those near the port of Telukbayer (Figure 3.6), along the hill highways to Solok and Padangpanjang, and from the escarpment at Lake Maninjau (Figure 3.7), indicate a MM strong shaking intensity of MM7 to 8, while the large earthflows from the low hills of volcanic tuffs east of Pariaman (Figure 3.8) which buried villages and killed more than 600 people may indicate a local intensity of approximately MM9.

## 4. TSUNAMI RISK

As noted in the preceding Section 2.1 and Figure 1.2, megathrust earthquakes are a common and regular event along

the Western coast of Sumatra, possibly occurring offshore from Padang with a recurrence interval of less than 200 years. Written records of historical earthquakes in 1779 and 1833, with magnitudes of M8.7 and M8.8 respectively, along the subduction interface offshore from Padang, uplifted the Mentawi Islands and created 5 to 10 m tsunami which swept inland up to 3 km (Natawidjaja *et al.*, 2006). As the subduction interface in this area currently indicates a seismic gap and thus appears to be locked, a megathrust earthquake generating ground motions possibly 30% stronger than those that occurred during the 30 September 2009 earthquake is predicted to have a high probability of occurrence in the next 30 years. It would be followed some 30 minutes later by a devastating tsunami with 5 to 10 m high waves that could sweep up to 3 km inland from the coast, inundating some 50% or more of Padang city. Therefore considerable effort has gone into public tsunami education and evacuation routes. This scenario event gives urgency and impetus to “building back better” in Padang. Buildings damaged in the 30 September 2009 earthquake should be repaired and strengthened to a high standard to be capable of withstanding this future earthquake and tsunami. New buildings, especially those of two or more levels, should be designed and built to a standard to remain undamaged after a worst-case future megathrust earthquake, so they can then be used for “vertical evacuation” from the pending tsunami.



**Figure 3.6:** *Rock fall from steep slopes behind the port of Telukbayur, some 12 km to the south of Padang city. Previous rockfalls were evident along the foot of these slopes.*



**Figure 3.7:** *Rock slides from part of the caldera rim which surrounds Lake Manijau.*



**Figure 3.8:** *Slope failures in volcanic tuffs some 70 m deep triggered by the earthquake. These unexpected earth flows from moderately steep slopes buried several villages and killed more than 600 people.*

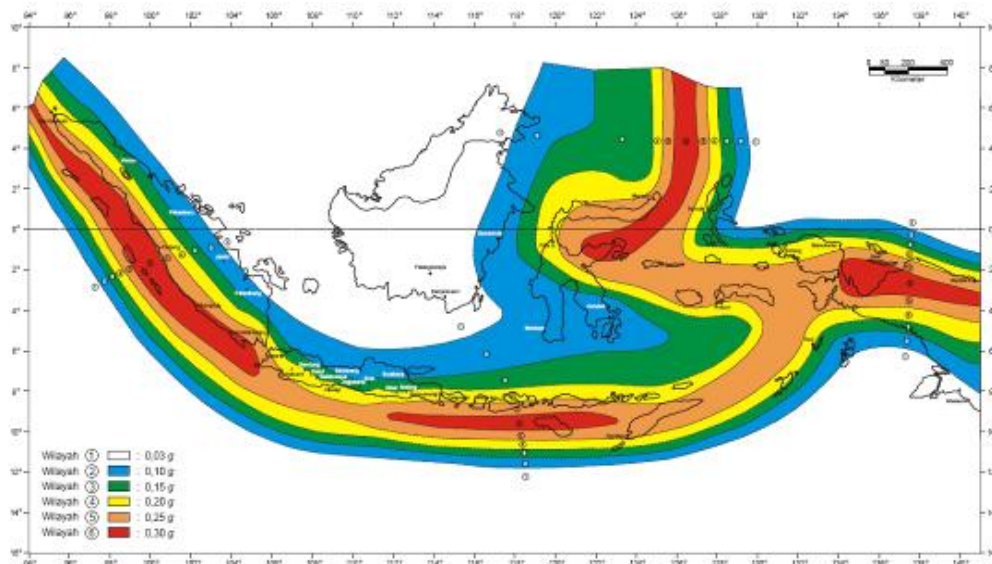


Figure 5.1: Seismic zoning map of Indonesia (SNI, 2002).

**5. OVERVIEW OF BUILDING DESIGN AND CONSTRUCTION IN WEST SUMATRA**

The most common construction materials in the earthquake-affected areas are lightly-fired weak brick, timber, stone, steel and concrete (sand, aggregate and cement) because of the local availability of these materials. Based on the materials used and structural aspects, the buildings in the areas visited by the team can be broadly classified under the following categories:

1. Unreinforced brick and block masonry with or without practical columns,
2. Stone masonry buildings,
3. Timber houses,
4. Reinforced concrete frame with masonry infill, and
5. Steel structures

Types 1, 2 and 3 are mainly used for house construction, while types 3 and 4 are typically used for larger buildings. However, these classifications merge with time and space, affordability, and accessibility as different structural systems and materials are mixed up. The most common binding material for masonry construction is cement mortar, though sand-lime mortar was also observed in stone masonry construction in Sicincin.

**5.1 Code Provisions and Structural Design**

Design and construction standards in Indonesia are specified by Standard Nasional Indonesia (SNI), which are administered by Badan Standardisasi Nasional (BSN). They have well-defined building design and construction standards. These include the loading standards, earthquake design standards and material specific design standards for reinforced concrete, steel, masonry, etc. The standards set out the minimum provisions for limit state design.

The seismic design provisions are provided by SNI 03-1726-2002 (SNI, 2002). The standard has been modelled on the 1997 Uniform Building Code (UBC). The standard divides Indonesia into six seismic zones (Figure 5.1), with zoning factors ranging between 0.03 and 0.3. Padang is located in the second most severe zone, with a zoning factor of 0.25. The standard also provides applicable ductility and overstrength factors for different materials and structural systems. The

present reinforced concrete standard is modelled on standards of the American Concrete Institute (ACI 318).

The first loading code in Indonesia was published in 1970 (Indonesian Loading Guidelines N.I.-18), where the design accelerations for Padang were 0.1g for use with working stress design (EERI, 2009), which was later revised and published in 1987 as SKBI 1.3.53.1987. The draft for this standard was developed in the 1970s by Beca under financial assistance from the New Zealand government. At that time Beca conducted a comprehensive study of the seismicity of Indonesia and developed both a seismic design standard (Beca, 1980a), and a specific design and construction manual for reinforced concrete and reinforced masonry structures (Beca, 1980b). The design standard divided the country into six zones and provided inelastic design spectra with modification factors. Padang and the surrounding region were classified as being in the second highest seismic zone. The guidelines provided general compliance requirements and simplified methods for design procedures for a number of common structural types including infill frames based on a compression strut mechanism.

Despite the change in methodology between 1987 and 2002 standards, the design earthquake load essentially remains the similar in Indonesia. Figure 5.2 presents a comparison of elastic design spectra proposed by the 1987 and 2002 earthquake loading standards for a hard soil, and also a spectrum developed from the measured 2009 ground acceleration which was presented in Figure 2.1. In the short period range, as shown in Figure 5.2, the elastic design spectra for the 2002 code are comparable to the spectral accelerations of the measured ground motions, whereas the values from the 1987 code are higher for the longer response period measured motions.

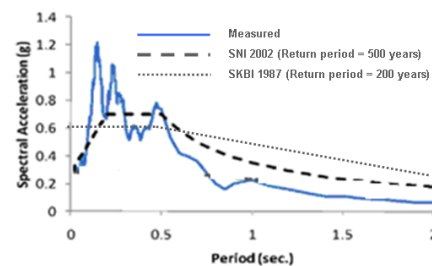


Figure 5.2: Comparison of design spectra (adopted from EERI, 2009).

It is not clear how much the simplified design guidelines for reinforced concrete infill frames developed in 1987 has been used in Indonesia. It appears that the construction of masonry buildings using practical columns and beams has developed in the course of time and has been widely used for one to two-storied houses, albeit with poor construction perhaps reducing quality. It also appears that other earthquake-resistant features, such as vertical steel reinforcement at building corners, were employed in masonry buildings as recommended by IAEE guidelines (IAEE, 1986).

It should be noted that most of the reinforced concrete buildings that suffered catastrophic failure were designed and constructed in the last 20-25 years, presumably following SKBI 1.3.53.1987 (SNI, 1987). However, the ductile detailing that is required by the seismic standard was not observed in any of the damaged or collapsed buildings, even though the seismic standard is a compulsory design code for all of Indonesia. However, due to a lack of enforcement in most areas outside of Jakarta and a few of the major cities in Indonesia, it appears that the standards are not rigorously applied in the design and construction process. From observations of the patterns of damage and destruction, it is apparent that the biggest problem is not the building standards, but their interpretation and implementation during both design and construction. A good knowledge of earthquake engineering is not apparent in West Sumatra. For example, it is common practice to design concrete or steel frame buildings as moment-resisting frames, while ignoring the strength and stiffness of the infill walls and their effect on the configuration of the building and on the overall seismic performance of the frame. The infill walls are constructed later wherever desired, as they are assumed to be non-structural elements. It also appears that there is a significant lack of understanding of the need for implementation of steel reinforcement detailing and for quality control during construction. Hence, although the national building code standards have been improved and the seismic design loads have increased with time, in general this does not mean a better building.

The building consenting process in most of Indonesia outside of Jakarta and other major cities has taken place more as a control of building planning rather than to control structural safety. The situation however differs in Jakarta, which has a building consent board that is well managed and reports to the provincial government. This includes a structural review panel (TPKB – Tim Pemeriksa Konstruksi Bangunan) which peer reviews designs as part of the construction approval process.

## 5.2 Building Production Mechanisms

For Indonesia in general and in the earthquake-affected areas in particular, two building production mechanisms work in parallel—engineered construction and non-engineered construction. Engineered construction involves structural design input and construction monitoring and covers mostly large and/or government buildings. At a conservative estimate, this sector produces less than 10 to 20% of the building stock.

The second sector is characterised by informal, owner-built, non-engineered construction. Most of the small buildings in the area, which conservatively constitute more than 80 to 90% of the building stock, are produced by this sector. It includes all types of buildings and material use (concrete, brick or stone masonry, etc) and occupancy (residential, commercial, institutional, etc). This sector is characterised by individualised decisions. Owner-builders seek advice from friends and neighbours, and local artisans, but professional engineering advice is rarely sought (even in urban areas), and if solicited is limited to the preparation of submission drawings for municipality permits. In this sector, construction

by convention rather than design is common. The owner themselves deal with materials, suppliers and issues of labour contracts. The owner-builders tend to do as much as possible themselves to keep costs down.

The traditional artisans, who usually have no formal training, play pivotal roles in construction activities in this sector, where they provide overall technical and organisational support. There seems to be no regulatory mechanism to control or monitor this class of building construction. It appears that the local engineers neither have enough knowledge, nor are they involved in this class of buildings because of the small scale of construction and because the building owners cannot afford engineers.

These building construction mechanisms have a bearing on the building's quality and for the legal enforcement of quality improvements. Any enforcement methods to improve the quality of non-engineered buildings through legal means alone is likely to be much more difficult than by introducing engineering design and artisan training requirements.

## 5.3 Construction Practices

From the damage patterns, it is evident that the poor quality of construction was one of the major reasons behind the catastrophic damage in the area. Even engineered buildings built with modern materials such as reinforced concrete and steel are of poor quality. An absence of quality control was seen at all stages of the construction cycle (Figure 5.3a). Substitution of construction materials without considering its effect on the performance of a building is common.

Use of plain bars, even up to 20 mm in diameter, for longitudinal reinforcement in columns and beams is common in the Padang area (Figure 5.3b). Mixing of deformed and plain bars is also common in the area (Figure 5.3c). In a number of cases, the reinforcement in columns (Figure 5.3d) were bent during construction—this led to reduction in tension capacity and often blew the concrete out when the bars tried to straighten, or came into compression during earthquake shaking. In a few cases, reinforcement splicing lengths were found to be about 20 diameters or even less for plain bars. Column bars are invariably lapped at the column base.

In detailing, stirrups were often observed at greater than 300 mm spacing throughout the column. It is not usual practice to provide stirrups in the beam-column joints (Figure 5.3e) and 90° hooks (Figure 5.4) are very common for stirrups, even in newer construction, although the seismic standard recommends 135° hooks. Ties are not used in columns.

Although plant-mixed concrete and concrete pumping has been observed in Padang, site-mixed concrete is the most common method of concrete preparation, and concrete vibrators are rarely used for compaction, particularly in small construction sites, resulting in low-grade honeycombed concrete. Even in larger construction sites, hand-mixed concrete is used for the columns (slabs/beams are normally a plant-mixed concrete) because a smaller pour is required in columns and so it is not economical to use plant-mixed concrete. The site-mixed concrete often has large aggregates (Figure 5.5), and can include rounded stones bigger than 50 mm. Lack of proper mix of concrete constituents, segregation and honeycombing of concrete was often seen in columns. It appears that the importance of adequate cover for durability is neither understood nor practised. This has led to heavily corroded reinforcement (as there is either no or poor quality concrete cover) because the area is near the sea and the atmosphere appears to be heavily corrosive.



*a) A typical RC frame building under construction with brick masonry infill panels. Note the deficient length of column reinforcement for future extension.*



*b) Plain reinforcing mild steel bars are commonly used.*



*c) Mixing of plain and deformed reinforcing bars.*



*d) Bent bars at the column base.*



*e) No stirrups in beam-column joint.*

**Figure 5.3: Lack of quality control in construction.**



**Figure 5.4:** A typical isolated foundation pad for a RC frame building. Note inferior quality of concrete and the wide spacing of stirrups with 90° hooks.



**Figure 5.5:** Segregation of concrete and large pebbles/cobbles in the concrete. Note 90° hooks and wide spacing of stirrups.



**Figure 5.6:** Missing nuts, bending of base plate resulting in large rotation of the column base.

Figure 5.6 shows a base plate of a steel column where half the nuts were missing and the other half were sluggishly tightened, which led to large rotation of the column leading to partial collapse of the roof. Damaged infill wall panels in a reinforced concrete frame buildings show evidence of thick mortar joints and inferior quality masonry.

In masonry buildings, the mortar is often weak enough to be crushed between the fingers—it was not able to bind the masonry units together. The weak mortar can be attributed to dirty sand, bad mixing, and use of a low ratio of cement to sand (1:8 or so). Thick mortar layers and non-filling of vertical joints are common in masonry construction. It is normally accepted that plaster will cover these deficiencies.

Figure 5.7 shows a wall that was damaged in the 6 March 2007 earthquake. Rather than strengthening the building, the cracks were covered by plaster. These opened again during the 2009 earthquake. Though the building survived, it shows the level of acceptance of poor construction techniques.



**Figure 5.7:** Diagonal cracks in a pier. This building was damaged in the 6 March 2007 M6.4 earthquake, but the building was not strengthened and the cracks were just covered.



**Figure 6.1:** Several examples of soft storey failure.

## 6. OBSERVED BEHAVIOUR OF BUILDING STRUCTURES

### 6.1 Configuration-Induced Damage and Destruction

It appears that the importance of configuration is not well understood for better seismic performance of a building—even of engineered ones. Existing configuration problems and mixed structural systems played a major role in the failure of many buildings. Soft or weak storey failure (Figure 6.1) was by far the most prevalent failure mode in heavily damaged buildings. These buildings in most cases have a soft storey at the base as they have an open ground floor and they were significantly stiffer on the upper levels because of infill walls. It is also fair to say that the buildings that have brick infill walls uniformly distributed both vertically and horizontally performed well. In a few cases, the brick walls saved the buildings, as these walls provided a gravity support system once the RC columns failed. Figure 6.2 shows a podium building in which the base of the tower section suffered significant damage due to amplification caused by the sudden stiffness change in the building.



**Figure 6.2:** Damage due to shaking amplification.

Figure 6.3 shows a building that collapsed above the first story. The first floor slab of the collapsed building was structurally integrated with the adjoining building, which was under construction when the earthquake struck. The integration with adjoining building created a vertical irregularity. Further, this building hammered the adjoining building which survived, although that building also was at the onset of soft storey collapse and suffered a residual interstorey displacement of around 30-40 mm between the first and

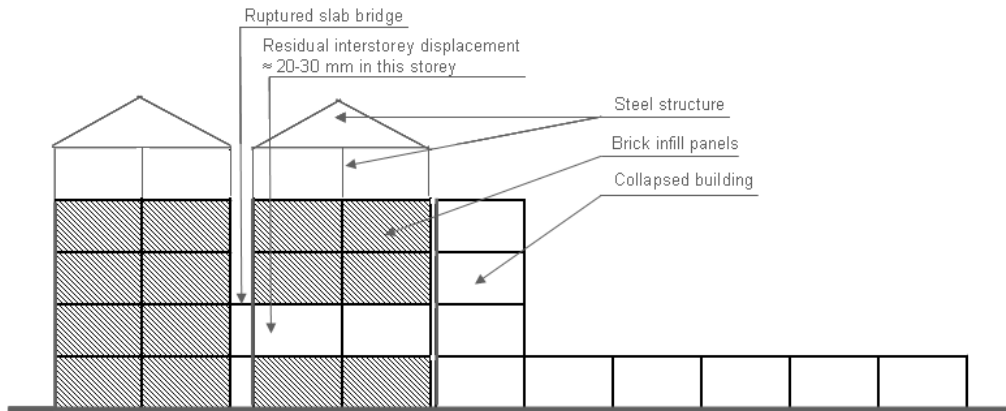
second floor slabs. Signs of pounding were visible on the building that survived.

Interaction between adjoining buildings was another problem observed in the area. Pounding between adjoining buildings was observed in row buildings without a seismic gap. Figure 6.4 shows two adjoining institutional buildings where adequate seismic gaps were not provided between the buildings. Signs of pounding can be seen at the interface of parapet walls. The floors slabs of both the building were aligned and both building were of the same height. Although their floors suffered pounding, both the building survived, presumably because of the aligned floors and the similar height of the buildings. In another case, two five storey high RC framed building were connected by a cast in-situ RC bridge slab. The bridge slab which was integral with the floor slabs of both the buildings, was ripped apart by shaking (Figure 6.5), which was presumably out-of-phase.

Damage in many buildings was observed due to plan irregularities. Buildings with L- and U-shaped plans and other torsionally active buildings showed evidence of torsion damage. Figure 6.6 (left) shows a small building with the front supported on a retaining wall and the back on slender stilts. Although the building survived, it suffered significant torsion, as evidenced by interaction between the building and the wall (Figure 6.6, right).

Short column effects were another commonly observed problem in RC framed buildings. In these buildings, the half-height walls restrained the columns and forced them to fail in shear, producing severe diagonal cracks and concrete spalling. This occurred in a number of buildings, particularly school, hospital and university blocks. Contrary to the accepted notion that the timber window frames are non-structural, it was interesting to observe interaction between timber window frames and columns. Figure 6.7 shows a short column in an RC frame building in which the mid-height window horizontal members restrained the column and forced it to fail in shear.

Shakedown spreading was one of the more interesting cases of failure seen. This failure type was encountered in one building where it has caused significant damage and a need for rebuilding. The building had a heavy porcelain tile roof supported on timber purlins and steel frames pitched at a steep angle to the middle. The steel frames did not have a collar tie (or any other tie) and so the cantilevered concrete columns on the perimeter carried the lateral spread loads from gravity loading. When the earthquake hit, the roof frames shook from side to side, imposing additional lateral loads to the top of the cantilevered columns. The columns have suffered significant flexural damage and have not recovered to their original position, allowing the roof to push out and spread around 300 mm out of position (Figure 6.8).



**Figure 6.3:** *View of a building collapse due to pounding and amplification, with the building layout shown below.*



**Figure 6.4:** *Pounding damage.*



**Figure 6.5:** *Rupture of a slab bridging two buildings (see Figure 6.3 for layout of the buildings).*



**Figure 6.6:** Building on stilts (arrow shows direction of shaking).

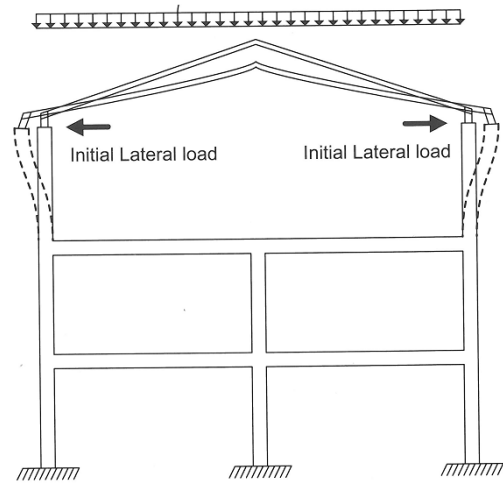


**Figure 6.7:** Short column failure. Note how the timber window frame has influenced the failure.

## 6.2 Unreinforced Brick Masonry (with and without practical columns).

Brick masonry buildings with variations are one of the most common building types in the area and are simple in form and construction. Damage to the masonry buildings in the area ranged from generally little damage (Figure 6.9a) to severe damage and total destruction (Figure 6.9b). Although many of the patterns of damage in the brick masonry buildings were similar to those of the stone masonry buildings, the brick masonry buildings in general performed better than the stone masonry ones.

From a construction viewpoint, the brick masonry buildings in the Padang area can be broadly classified into two groups: those constructed in the eighteenth and nineteenth century in the Dutch era (or of a similar construction type), and those constructed recently. The damage patterns in these buildings were distinctly different because of the technology employed.



**Figure 6.8:** Column spreading because of a lack of truss ties (Hyland and Wijanto, 2010).

Those buildings constructed in the Dutch era or of a similar construction type were used for residential, commercial, institutional, and warehouse purposes and had thick brick walls (in the order of 230 to 350 mm, Figure 6.10, left) constructed in cement with lime mortar. The buildings were up to three storeys high, and their floor and roof structure were constructed of timber (Figure 6.10, right), and jack arches. These buildings were apparently unreinforced, and did not incorporate any earthquake-resistant features. From a configuration point of view, these are mostly rectangular in plan. Generally, these buildings were simple in elevation, without any stiffness change if they were more than one storey high.

Masonry buildings constructed in recent time differ distinctly from Dutch era or similar type buildings. These newer buildings are one to two storeys high, with half-brick thick walls constructed in cement sand mortar. The most distinct feature of these building is that they incorporate practical



*a) Old Bank of Indonesia - a Dutch era building that survived without structural damage.*



*b) Total collapse of a similar unreinforced masonry building.*

**Figure 6.9:** *Different performance of masonry buildings.*



**Figure 6.10:** *Details of Dutch-era buildings: wall section, floor and roof (left), and a flexible timber floor diaphragm without any tie-up, with supporting walls (right).*

columns and beams (Figures 6.11a, b and 6.19a). Practical columns and beams are minor reinforced concrete columns and beams (columns and beams with small cross sections in order of  $\sim 100 \times 100$  mm with nominal steel reinforcement (with four 8 mm or 10 mm diameter longitudinal reinforcement and 5 to 6 mm diameter stirrups at every 250 mm centres, Figure 6.11c). These columns and beams act together to tie masonry and provide integrity to the structure. The floors of these buildings are usually constructed of cast-in-situ reinforced concrete slabs, although timber floors (Figure 6.10, right) are also seen. The roofs of these houses may be concrete slab but are generally constructed of metal sheet on a steel or timber structure.

In general, small houses constructed in brick masonry with practical columns performed better than their counterparts without practical columns because of the tie provided by the beams and columns, and their light weight. The most common observed failure mechanisms in brick masonry buildings were out-of-plane failure of walls, in-plane failure manifested by

diagonal shear cracks and rocking of piers, and floor and roof failure due to loss of support. In general, out-of-plane collapse of walls was more common in buildings with flexible floors and roofs, whereas buildings with reinforced concrete floors and roof slabs mostly suffered diagonal shear failures of in-plane walls.

In both types of masonry buildings, out-of-plane failure of walls was observed, caused by lack or failure of connection between orthogonal walls (Figure 6.12a), and the walls and floor/roof structure in the case of buildings with a flexible floor and roof, or lack of a diaphragm. Figure 6.12c shows a school building where walls toppled because of the lack of anchorage between walls and the roof truss and a lack of a diaphragm. Figure 6.12b shows a two-storey house constructed in brick masonry with practical columns that suffered out-of-plane toppling of walls because of the slenderness of the wall and a lack of a good connection between walls and beam. The problem was further accentuated



*a) A typical brick masonry building in the area.*



*b) A brick masonry building under construction.*



*c) Reinforcement ready for a practical beam. More stirrups would make a big improvement.*

**Figure 6.11:** *Modern brick masonry buildings.*

by a lack of a diaphragm, which imposed a large displacement at floor level and amplified the shaking. The columns of this house suffered hinging at their top and bottom, and the house had a residual displacement of around 100 mm at its first floor level. Toppling of unreinforced gable walls was common in duo-pitched roof buildings (Figure 6.12d), however timber gable walls performed well.

Diagonal shear cracks were common in masonry buildings, particularly in old masonry buildings and buildings with concrete slabs, whereas in new buildings with practical columns, these were restrained by the tie columns and beams. In many cases the diagonal shear cracks were combined with out-of-plane toppling of walls (Figure 6.13a). Figure 6.13b shows an old church which suffered diagonal shear cracks and out-of-plane toppling of walls. However, the roof survived because of the availability of an alternative load path. Figure 6.13c shows an unreinforced masonry old house where the front gable wall of the building split vertically into two parts because of a loss of support to the left side of the gable wall.

It is interesting to note that an old church which suffered partial collapse due to both out-of-plane and in-plane damage at its back, as well as sliding of roof tiles, did not suffer much

damage in the front, and even the bell tower of the church survived (Figure 6.14).

In a few cases, anchorage failure of practical beams was also observed in masonry buildings due to a lack of adequate anchorage of beam bars in the practical columns.

### **6.3 Stone Masonry Buildings**

Many damaged or collapsed stone masonry buildings were observed in Sicincin village, one of the heavily damaged earthquake-affected areas. Typical stone masonry buildings were simple one- to two-storey dwellings. The walls of these buildings were constructed of river boulders (around 100 mm diameter) in a lime or cement-sand mortar. The typical thickness of these walls was of the order of 250 mm. The floors and roofs of these buildings were constructed of timber. One of the dwellings visited had corners constructed of bricks. During the shaking, the brick masonry separated from the stone masonry (Figure 6.15).

The commonly observed failure modes were out-of-plane failure of walls because of a lack of connection between



*a) A building with out-of-plane collapse of walls.*



*b) Out-of-plane failure of a brick wall in a building with practical columns and beams.*



*c) Failure of a poorly connected roof due to loss of support.*



*d) Toppling of a gable wall.*

**Figure 6.12: Out-of-plane failure in brick masonry buildings.**



a) Diagonal shear crack combined with out-of-plane failure of an unreinforced brick wall in a school (note separation of in-plane and face loaded walls).



b) An old masonry brick church barely survived the earthquake. The roof survived because of alternative load paths.



c) Vertical cracking of a house wall.

Figure 6.13: Failure modes in brick-masonry buildings.



Figure 6.14: Church with undamaged bell tower.



*Figure 6.15: A stone masonry building with unreinforced brick columns.*



*Figure 6.16: Partial collapse of a stone masonry house building due to out-of-plane toppling of walls (note lack of connection between structural components and shear failure of piers).*



*Figure 6.17: Corner failure (note lack of connection between walls).*

orthogonal walls and connection with a diaphragm, corner failure, and diagonal shear failure of walls. These are shown in Figures 6.16 and 6.17.

Interestingly, even though delamination is one of the most common failure modes in stone masonry walls (Bothara and Hiçyılmaz, 2008), delamination of stone masonry walls was not observed in Sicincin, despite a lack of through stones. It could be attributed to thin walls with relatively good mortar, a lack of irregular unstable stones, the high volume of mortar which provided a cushioning effect for the stones, a lack of loose rubbish in the core of the wall, and/or the frequency of the shaking.

Another interesting observation in this village is that in a partially collapsed two-storey building (Figure 6.16) vertical reinforcement was observed in the walls at the corners of the building. The provision of vertical reinforcement at the wall junctions and door/window jambs is a part of earthquake-resistant construction of masonry buildings and has been recommended by IAEE guidelines for non-engineered masonry construction (IAEE, 1986). However it is apparent from the observations that the vertical reinforcement are not effective in providing any protection against shaking unless the building has reasonable diaphragms, a good connection between diaphragms and walls, connections between orthogonal walls at intersections, and horizontal reinforcement in the wall.

#### 6.4 Reinforced Concrete Frame Buildings with Infill Walls

Cast-in-situ reinforced concrete (RC) moment-resisting framed buildings are one of the most common construction types in the area for office, residential, and institutional uses. Most reinforced concrete-framed buildings have been constructed in the last 20 to 30 years. Construction of RC shear walls is not common, though seen (Figure 6.18). In general, these buildings were one to five storeys high with the roof and floors cast-in-situ reinforced concrete slabs, although a metal sheet roof on a timber or metal structure is also common. Construction of the frame first and the slab later is also common in the area. Invariably, the cladding and partition walls are constructed of brick or concrete block (Figure 6.19a), however mixing of walling materials was also seen

(Figure 6.19b). In these buildings the masonry forms an infilled frame with tie columns and beams. The partition walls are usually a brick width thick (~100 mm) or a block thick (150–200 mm). By convention, the walls are tied to the frame using practical columns and beams (Fig 6.19a), but this is not always the case.

Reinforced concrete-framed buildings in the area suffered both configuration problems, and strength and ductility problems. Column failure leading to total or partial destruction of the buildings was commonly seen. It was evident that the masonry infills in RC frame with masonry infill buildings acted as compression struts. The RC beam and column frame members behaved as tie members in these buildings, holding the masonry together rather than acting as a moment-resisting frame. This action put significant demand on the columns, forcing them to fail (Figure 6.19c). Even in moment-resisting frames a strong beam-weak column hierarchy appeared to be common, as there was very little evidence of beam failure

Figure 6.20 shows the Terminal Bingkuang (Bus terminal) buildings in Padang. The section visible in the photograph is actually three structures separated by construction joints. The structure has been constructed in farm land and is located around 20 km north-west of Padang city along the city bypass route. This building did not have many infill walls. The second storey of the central building suffered destruction due to strong beam-weak columns, anchorage failure, column reinforcement splicing failure and column bursting, rendering the building unsafe. However, it is interesting to note that performance of a few of the circular columns with spiral stirrups was excellent, and these maintained integrity even after large rotation at their base (Figure 6.21). The adjoining buildings on both sides of the central building suffered unequal settlement due to liquefaction, with soil spreading in and around the building (see Figure 3.5).

Figure 6.22 shows damage to a church building constructed with a RC frame with masonry infill in the perimeter wall. The church has a large hall with a large-span steel roof structure. It has a mezzanine floor tied to the perimeter frame. The violent shaking is manifested by the severe interaction between the infill walls and frame and significant damage to infill walls, anchor failure between the mezzanine floor and the perimeter frame, and roof support anchor failure (see Figure 6.30).



**Figure 6.18:** A rare example of a new RC frame building with concrete shear walls under construction.



*a) Brick infill panels with practical columns and beams.*



*b) Mixing of infill wall materials.*



*c) Column failure due to shear imposed by infill wall structure.*

**Figure 6.19: Construction practices for RC frame buildings.**



**Figure 6.20:** Padang's bus terminal building suffered extensive damage due to differential ground settlements and soft storey collapse of the central block. Note destruction of the second storey; roof ridge should be horizontal.



**Figure 6.21:** Excellent performance of a column with spiral stirrups in the bus terminal building.



**Figure 6.22:** A badly damaged Dutch-era masonry brick church building with a recently added internal RC frame and slab mezzanine floor—which probably prevented complete collapse.

Among the different failure mechanisms, soft storey failure (see Figure 6.1) in the bottom storey was one of the most commonly observed failure modes. This was due to either an open bottom storey compared to upper ones or to degradation of stiffness and strength of the bottom storey after initiation of damage there. The other observed failure modes (Figure 6.23) were shear failure of columns and beams, beam bar anchorage failure, splicing failure of reinforcement, opening of stirrups, cold joints, and crushing of concrete, out-of-plane collapse and severe in-plane shear damage of walls.

Plastic hinging of columns was mostly concentrated in the first storey, however in one four-storey high school building, these were distributed up the height, although these were mostly concentrated in the bottom two storeys (Figure 6.24a). Figure 6.24b shows the bottom storey of a five-storey high

institutional building with reasonably large columns and beams, and a good proportion of vertical steel in the columns but was significantly deficient in stirrups and cross ties in the columns. The building suffered plastic hinging at both top and bottom of the columns in the bottom storey, and a few columns suffered buckling of reinforcement.

The infill panels suffered both in-plane damage and out-of-plane instability. In general, practical columns and beams helped in stabilising these panels. Figure 6.25 presents a sample of images showing the different performance of infill walls. Figure 6.25a shows an infill wall that suffered corner crushing and diagonal shear cracks due to compression strut action, however the wall did not lose its stability because of the presence of practical columns and a beam at mid-height of



*a) Failure of column bar splicing.*



*b) Cold-joint problem.*



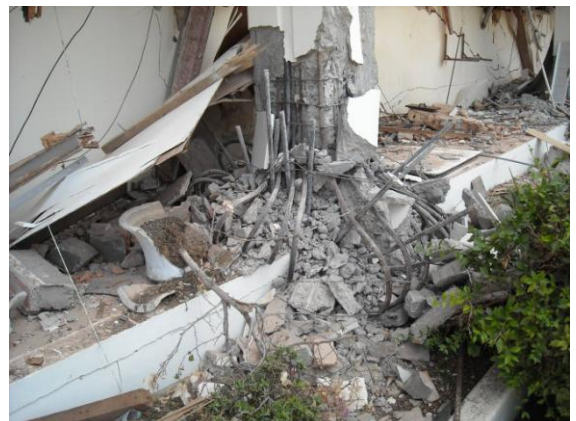
*c) Failure of column due to sudden change of stiffness.*



*d) Beam bar anchorage failure (at roof level) and crushing of column in the bus terminal building.*



*f) Failure of columns leading to failure of high-bond floor.*



*e) Column base crushing and buckling of reinforcement due to opening of stirrups.*

**Figure 6.23: Typical RC frame failures.**



*a) Plastic hinges distributed along the height of the building, presumably due to the similar strength and stiffness of each level.*



*b) Plastic hinging of columns of an open bottom storey.*



*c) Column hinging at top and bottom. The building survived due to the alternate gravity load-carrying system provided by the walls. Note the large rotation of columns imposed by shortening of columns due to the sill wall.*

**Figure 6.24: Plastic hinging of columns.**

the wall. Figure 6.25b shows a few brick infill panels that suffered diagonal shear cracks because of interaction with the main structure, even though the panels were not confined by the adjoining columns. It is evident that a connection between the wall panels and the structure existed through the floor structure. Although many infill panels toppled under face loading during the shaking, many others survived, apparently because of the arching of the panels with connections between the wall panels and the perimeter frames. The out-of-plane failures of the infill panels (Figure 6.25c) were more common in upper levels because of amplification of the shaking.

### 6.5 Timber Buildings

Timber is one of the commonly used materials in traditional building construction. Typical timber buildings are one to two storeys high with an attic (Figure 6.26a). Minang houses (Figure 6.26b) are traditional timber houses common in the Padang area. The bottom floor of this house type is typically raised off the ground, leaving an open bottom storey.

Although the team did not focus on timber buildings, no earthquake-induced damage was observed in the buildings visited (Figures 6.26 and 6.27). The seismic performance of

the observed timber buildings in Padang and other parts of the earthquake-affected areas was far better than other types of buildings.

### 6.6 Steel Structures

There were also a number of multi-level buildings or additions to existing buildings that used two-way structural steel moment-resisting frames with composite concrete and steel floor slabs. Structural steel was predominantly used for roof and portal frame structures. In steel structures, roof members are commonly attached to cast-in plates by site welding. Hooked rod bracing tensioners are also common. No use of concentrically braced or eccentrically braced frames in multi-level buildings was observed. Infill walls of unreinforced brick similar to those used in concrete frame construction were common, with no separation from the structure.

Steel columns in one two-way moment-resisting frame that performed well were concrete encased. During this earthquake, a five-storey two-way steel frame with metal decking collapsed in a pancaking manner (Figure 6.28a), with



*a) Wall panel is stable, even after severe damage.*

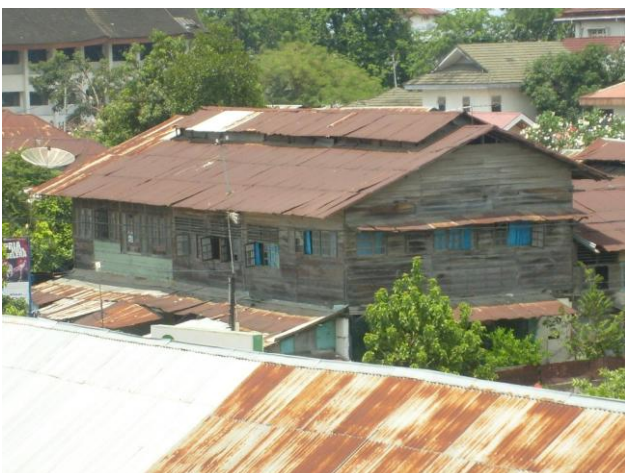


*b) Diagonal failure of infill walls separated from columns.*



*c) Out-of-plane failure of infill walls.*

**Figure 6.25: Performance of infill panels.**



*a) An old timber building in Padang.*



*b) A traditional carved timber Minang house.*

**Figure 6.26: Typical timber houses.**



*a) A timber house without structural damage.*



*b) A large span timber frame structure without any damage.*

**Figure 6.27: Generally good performance of timber buildings.**

weak axis column hinging. This indicated strong beam/weak column design, inadequate strength and a lack of sufficient consideration of second-order effects. Another steel frame with infill brick panels built as an extension to an existing reinforced concrete frame with infilled masonry collapsed in what was likely to have been Level 1 to 2 columns, with combined bending/compression buckling (Figure 6.28b). It appears that structural designers do not consider the lateral load amplification effect of brick in-fill panel walls in shortening the natural period of moment-resisting frame structures, as the infills are deemed to be non-structural. The displacement compatibility of the structure with the cladding system and infill walls, or with existing structures, does not appear to be well considered. Diagonal roof rod bracing often appeared to be very slender and the use of hooked tensioners was common. The failure of roof bracing in a portal-framed warehouse with heavy masonry wall panels raised questions about the displacement compatibility of the roof rod bracing

with that of the walls under face loading during the earthquake.

Infill cladding panels between the top floor of a multi-level concrete frame structure and its steel truss roof weren't adequate, resulting in partial disconnection from the roof structure and a serious overhead falling hazard. In the bus terminal, one of the roof trusses of one of the buildings was anchored to an adjoining building. During the shaking, the anchor bolts snapped (Figure 6.29) because of the relative displacement between these buildings. Base plate anchor failure was observed in other buildings as well. Figure 6.30 shows such a failure in an old church building constructed with a tall RC frame with brick masonry infill due to a lack of displacement compatibility between trusses and the tall RC cantilever columns. It is common practice to design these anchors for wind uplift. During earthquake shaking the tall walls imposed a much larger shear demand, leading to the failure of these anchors.



*a) Pancaking after weak axis column bending failure.*



*b) Building collapse due to column failure.*

**Figure 6.28: Steel structure failure.**



*a) Anchor and roof bracing ties failure.*



*b) Anchor plate failure (note the plate was anchored to an adjoining building).*

**Figure 6.29: Failure of bracing ties at Padang bus terminal.**



**Figure 6.30: Failure of anchors between a steel roof truss and concrete column in an old church.**

## 6.7 Performance of Staircases

In the earthquake area it is not the practice to isolate the staircase structurally from the main structure, even when the structure is flexible. Thus, the waist slab acted as a compression strut when the building was violently shaken by the earthquake. Due to the integration of the staircase with the structure, staircases suffered damage due to strut action. However, in many cases, it may have helped the building to remain standing by providing a lateral support through bracing action.

In straight waist slab stairs, the damage was mostly concentrated at the ends of the waist slab or/and at the mid span. The waist slab suffered support failure, and compression-flexural failures. Figure 6.31a shows a waist slab that suffered damage at all three supports—bottom, mid and top. A close-up view of the mid support is shown in Figure 6.31b, where the supports and the slab both suffered damage. Figure 6.32 shows compression-flexural crack between the supporting beam and the waist slab of a staircase of another building. In Figure 6.33, the waist slab suffered severe damage due to compression-flexure and is on verge of collapse. The

problem in this staircase was further compounded by severely deficient concrete quality.

Dog-leg staircases without a half-landing support were also seen in a few school buildings. This type of staircase performed better than the stairs with a half-landing support because of the lack of strut action. These stairs hammered with the adjoining walls and damaged the walls because of the lack of a separation gap between the wall and the landing, but the staircase did not suffer any damage.



*a) Interaction between structure and waist slab.*



*b) Close-up view of staircase in (a).*

**Figure 6.31: Typical staircase failure during an earthquake.**



**Figure 6.32:** *Compression-flexural crack between waist slab and supporting beam.*



**Figure 6.33:** *Compression failure of a waist slab.*

**7. PERFORMANCE OF NON-STRUCTURAL AND OTHER ELEMENTS**

It is not usual practice in the area to tie non-structural elements to the building structure, which leads to dislocation or damage to non-structural elements. Figures 7.1–7.3 present several

photographs showing non-structural damage. Figure 7.1 shows a museum that shed roof tiles due to shaking because tiles were not tied to the purlins. Figure 7.2 shows a “pinnacle” at the top of a building leaning on one side because of amplification of severe shaking. Damage to false ceilings was observed in many buildings (Figure 7.3) because of a lack of proper ties and detailing.



**Figure 7.1:** *Shedding of untied tiles.*



**Figure 7.2:** *Tilting “pinnacle” and toppled gable wall.*



**Figure 7.3:** *Damage to false ceilings was common.*

Construction of unreinforced boundary walls is common in the area and many of these walls suffered damage or destruction. These walls failed at the ground level where they were restrained. Figure 7.4a shows part of a toppled boundary wall which failed at its base where the construction materials

changed from stone masonry to concrete and there was no anchorage with the foundation. Figure 7.4b shows a leaning gate because of movement of the gate columns and the loose soil around the columns.



**a) Toppled boundary walls were common.**



**b) Leaning towers at a gate.**

**Figure 7.4:** *Performance of boundary walls.*

## 8. LESSONS LEARNT

There are some overarching lessons that are considered most relevant from the observation of damage and destruction following the Padang earthquake. These are summarised below:

- The only way to reduce risk to humans and their property is to improve the quality of the building stock by implementing appropriate building standards.
- Non-structural elements can significantly alter the seismic performance of the main structure and need to be addressed during the design phase.
- A robust quality control system for both design and construction is necessary. The system needs to cover the two parallel building production systems: engineered and non-engineered.
- Any policy frame work should be able to acknowledge and address these parallel building mechanisms:
  - Engineer-controlled regime (top down approach): This would be applicable for large building construction where engineers could enforce quality control for better construction. This sector should go through formal mechanisms of implementation (structural design, building consent, construction monitoring, etc). However, it appears from the observed damage patterns and discussions with local engineers that they need some sort of training on earthquake resilient reconstruction, repair and strengthening. This training will help their understanding of earthquake-resilient design and reconstruction and will help with the trickle down of knowledge.
  - Craftsmen-controlled regime (bottom up approach): For small houses where owners can not afford (in general) an engineering design or monitoring of the site, the craftsmen can help to significantly improve the situation if they are taken into trust and trained well in earthquake-resilient reconstruction.
- From the point of view of implementation of safer construction, a suitable system could be a stratified system that classifies the buildings based on their size, occupancy, etc and defines compliance levels based on the risk profile of the building.
- To achieve the goal of safer construction, knowledge dissemination is needed to all levels of stakeholders in general and trades-level people in particular, as they play a pivotal role in the construction of non-engineered buildings. Further, they are the end implementers of the technology. Unless they understand the technology and are empowered with the necessary skills, even a well engineered design can not be implemented.
- Experience shows that unless local people are aware of the impending risk, or their risk appetite is raised, any good technology can not be implemented in a sustainable way through the legal system alone. An awareness of risk will help create a demand for safer technology. What is required is decentralisation of risk knowledge down to the grassroots level.
- To implement seismic safety, a top-down (legal framework, engineered design, controlled constructions, etc) and bottom-up process (awareness, training of tradesman, decentralisation of technology and skills, etc) should work in tandem.
- A few easy to implement but robust construction steps and a simple system of checks could help address the large majority of building weaknesses without much intervention.
- For historical buildings, a separate strategy is needed that defines the seismic design philosophy and approach for these buildings for their long-term survival.
- Short-term, mid-term and long-term programmes are needed.
- We believe that the lessons learnt are equally applicable to all earthquake-prone regions and countries in general, and developing countries in particular, for the reduction of earthquake risk as well as risk from other disasters.

## 9. CONCLUSION

This paper has presented the observed behaviour of building structures during the 30th September 2009 Padang Earthquake. Although much of the damage and destruction is attributed to structural failures, these are just the manifestation of much deeper problems in the built environment of the earthquake-affected area. The root causes for the scale of damage were a failure to appreciate the earthquake hazard in the area, a lack of understanding and dissemination of techniques for earthquake-resistant construction, the absence of a building control framework and process, and the poor quality of control mechanisms. Often, there was no real understanding of how building quality was affected by the various construction methods (and, in particular the more recent methods of construction), nor was there any real evidence of an understanding of how structures behave during earthquakes. Socio-cultural and economic factors further exacerbated the problem.

A predominant reason for the scale of damage and destruction is considered to be the total failure of dissemination of knowledge of earthquake-resistant construction through virtually all levels of society and, in particular, in the engineering community. The authors suggest that the engineering professionals need to take the lead in improving the social status and importance of the construction industry. The earthquake engineering challenges affect the entire society, and require a collective agreement that earthquake safety is a desirable social target.

This earthquake is a reminder of the serious impending seismic threats being faced by Padang and West Sumatra. To mitigate the risk, political support and clearly defined separate strategies to address the problems of engineered and non-engineered buildings, is required.

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### REFERENCES

- Antara (2009) Number of fatalities in W Sumatra quake now 1,115, <http://www.antaraneews.com/en/news/1255472809/number-of-fatalities-in-w-sumatra-quake-now-1-115>.
- Beca (1980a) Draft Code of Practice for Seismic design of Buildings in Indonesia, A report prepared by Beca Carter Hollings & Ferner Ltd and The Indonesia Counterpart Team under the direction of The NZ Steering Committee as part of NZ Bilateral Assistance Programme to Indonesia.
- Beca (1980b) Manual for the Design of Normal Reinforced Concrete and Reinforced Masonry structures, A report prepared by Beca Carter Hollings & Ferner Ltd and The Indonesia Counterpart Team under the direction of The NZ Steering Committee as part of NZ Bilateral Assistance Programme to Indonesia.
- Bothara, J.K., Hiçyılmaz, K.M.O. (2008) General Observations of the Building Behaviour during the 8th October 2005 Pakistan Earthquake. *Bulletin of the New Zealand Society for Earthquake Engineering* **41** (4): 209-233.
- Brunsdon, D., Bothara, J.K., Stannard, M., Beetham, D., Brown, R., Hyland, C.W.K., Lewis, W., Miller, S., Sanders, R., Sulistio, Y. (2010) Building Safety Evaluation Following the 30th September 2009 Padang Earthquake, Indonesia. *Bulletin of the New Zealand Society for Earthquake Engineering*, **43**(3): 174-181.
- EERI (2009) The Mw 7.6 Western Sumatra Earthquake of September 30, 2009, The Learning from Earthquakes, *EERI Special Earthquake Report*, [http://www.eeri.org/site/images/eeri\\_newsletter/2009\\_pdf/Padang-eq-report-NL-insert.pdf](http://www.eeri.org/site/images/eeri_newsletter/2009_pdf/Padang-eq-report-NL-insert.pdf)
- Hyland, C.W.K., Wijanto, S. (2010) Lessons for Steel Structures from the 2009 Earthquake Damage in Padang, *Bulletin of the New Zealand Society for Earthquake Engineering*, **43**(2): 134-139.
- IAEE (1986) Guidelines for Earthquake Non-engineered Construction, International Association for Earthquake Engineering, reprinted by National Information Centre of Earthquake Engineering, at IIT Kanpur, India, [http://www.nicee.org/IAEE\\_English.php](http://www.nicee.org/IAEE_English.php)
- Natawidjaja, Hilman, D., Sieh, K., Mohamed Chlieh, M., Ggaletzka, J., Bambang, Suwargadi, W., Cheng, H., Edwards, R.L., Avouac, J.-P., Ward, S.N. (2006) Source parameters of the great Sumatran megathrust earthquakes of 1797 and 1833 inferred from coral microatolls. *Journal of Geophysical Research*, **111**, B06403, doi:10.1029/2005JB004025: 1-37.
- SKBI (1987) Pedoman Perencanaan Ketahanan Gempa untuk Rumah dan Gedung (Earthquake Resistance Building and Housing Design Guide), SKBI – 1.3.53.1987, Standar Konstruksi Bangunan, Jakarta.
- SNI (2002) Earthquake Resistance Design Guidelines for Buildings (unofficial translation in English), SNI 03-1726-2002, Standar Nasional Indonesia, Jakarta, 278 p.
- USGS (2009) Magnitude 7.5 - Southern Sumatra, Indonesia, 2009 September 30 10:16:09 UTC, <http://earthquake.usgs.gov/earthquakes/recenteqsww/Quakes/us2009mebz.php>