Disaster Risk Management for the Historic City of Patan, Nepal

Final Report of the Kathmandu Research Project

March, 2012

Research Center for Disaster Mitigation of Urban Cultural Heritage, Ritsumeikan University, Kyoto, Japan

Institute of Engineering,Tribhuvan University,Kathmandu, Nepal

Final Report of the Kathmandu Research Project
Disaster Risk Management for the Historic City of Patan, Nepal

Rits-DMUCH (Research Center for Disaster Mitigation of Urban Cultural Heritage),
Ritsumeikan University, Kyoto, Japan
IoE (Institute of Engineering),
Tribhuvan University,Kathmandu, Nepal

This work was supported in part by Global COE Program ”Global COE for Education, Research and Development of Strategy on Disaster Mitigation of Cultural Heritage and Historic Cities”, MEXT, Japan.

Date of Publication: March 2012
Published by: Rits-DMUCH
Research Center for Disaster Mitigation of Urban Cultural Heritage, Ritsumeikan University
Komatsubarakita-Machi, Kita-Ku, Kyoto, 603-8341 Japan
Tel: +81-75-467-8801

Edited by: Rohit Jigyasu, Naoko Itaya

All illustration and diagrams have been taken from the respective presentations. Where possible credit has been given.

PREFACE

Global Center of Excellence (GCoE) for Education, Research and Development of Strategy on Disaster Mitigation of Cultural Heritage and Historic Cities was established in 2008 to implement advanced education and research for the protection of cultural heritage and historic cities that have them and to develop these concepts on an international scale. One of the goals of GCoE program is research and development of universally practical, applicable technologies for mitigating the effects of disasters on cultural heritage.

Under this program, the Research Center for Disaster Mitigation of Urban Cultural Heritage, Ritsumeikan University, Kyoto and the Institute of Engineering, Tribhuvan University, Kathmandu, Nepal initiated a collaborative research project, established in spirit through the Memorandum of Understanding signed in 2009. The research aims at formulating comprehensive risk mitigation planning strategies for historic urban area of Lalitpur (Patan), which is located within the core area of one of the seven World Heritage Sites of Kathmandu Valley, Patan Durbar Square.

As part of this research project, researchers from cultural heritage and disaster management fields from both the institutions have undertaken joint research activities within two broad areas – assessment of structural safety of traditional buildings and disaster mitigation planning of the historic area. Various activities undertaken as part of this research project include field surveys aimed at mapping, documentation, interviews, workshops, experiments as well as laboratory analysis.

The research outcomes of the both groups are integrated to develop the disaster management plan and the risk mitigation proposal for the historic urban areas aimed at saving human lives as well as cultural heritage in the event of an earthquake that may strike the area in the near future.

We would like to congratulate all those who have been involved in this important research project for their hard work and dedication that has led to this outstanding work that would contribute immensely towards reducing disaster risks to our irreplaceable cultural heritage in the World.

Kenzo Toki
Director,
Research Center for Disaster Mitigation of
Urban Cultural Heritage,
Ritsumeikan University,
Kyoto, Japan

Bharat Pahari
Dean,
Institute of Engineering,
Tribhuvan University,
Pulchowk Campus, Lalitpur,
Kathmandu, Nepal

CONTENTS

1. Introduction

The Kathmandu Valley, accommodating the capital city, and being the cultural, commercial and political center of Nepal, lies in a highly seismically vulnerable zone. All the major earthquakes of past centuries have badly affected the valley. Some of the major earthquakes have even devastated its cities. The Kathmandu Valley is well known for its traditional historical buildings and monumental structures, most of them built in the Medieval Period of Nepal. The rich history, importance and original peculiarity of arts and architecture of these structures have made their sites listed in the World Heritage by UNESCO. Protection of these structures against major earthquakes has been one of the major concerns in the conservation of the cultural heritage.

The Kathmandu Valley is situated in the middle of the country at an elevation of about 1300 m above sea level. The valley, surrounded by high hills of average elevation ranging from 2100 m to 2765 m, has maximum sizes of 25 km and 20 km, respectively in the east - west and north - south directions. According to the history of the origin of Nepal, the valley to the boundaries of which the then Nepal was limited, used to be a large lake before the accumulated water was drained off. Kathmandu Valley has been considered as an in-filled basin formed during the Quaternary period, mainly by the rising of the Mahabharat Lekh, the mountain range forming the southern boundary of the Valley.

Geographically, the Kathmandu Valley, is considered as an in-filled basin, and has sediment deposits of different layers varying from one part to another. If the valley is identified as highly seismic due to its geographical location, geological and seismo-tectonic reasons, its different parts have different seismic effects due to considerable variation in soil conditions.

The Kathmandu Valley has three major cities of historical importance. All three cities of Kathmandu, Lalitpur and Bhaktapur accommodate many monument zones. The cultural heritage of Kathmandu Valley represents the art, architecture and civilization of the country. The ever increasing population, uncontrolled planning, indiscriminate reconstruction and the haphazard development have an adverse impact on the historic-cultural heritage of the valley, in the meantime, substantially increasing the seismic risk factor of the valley. In view of the need of conservation of the heritage sites with clusters of heritage structures, old and new traditional buildings, and with an objective of mitigating the disaster risk due to possible earthquakes in future, the development of earthquake disaster risk management plan for the valley has become an issue of utmost importance. In order to establish such a disaster risk mitigation plan for the historic urban areas, a comprehensive risk assessment of them considering the risk to inhabitants, properties as well as heritage values needs to be carried out. As a part of the risk mitigation planning measure, apart from building an awareness of vulnerabilities as well as capacities, it is important to establish social and financial programs to address them.

With an aim to develop a comprehensive risk mitigation strategy for historic urban areas of Kathmandu Valley, the Research Centre of Disaster Mitigation of Urban Cultural Heritage, Ritsumeikan University (Rits-DMUCH), Kyoto, Japan in collaboration with the Institute of Engineering (IOE), Tribhuvan University, Kathmandu, Nepal has undertaken the research. The collaborative research was initiated under the IOE – Rits-DMUCH Collaborative Research Project, established in spirit with the Memorandum of Understanding signed in 2009. The research is aimed at formulating comprehensive risk mitigation planning strategies for historic urban area of Lalitpur (Patan), which is located within the core area of one of the seven World Heritage Sites of Kathmandu Valley, Patan Durbar Square (Fig.1.1).

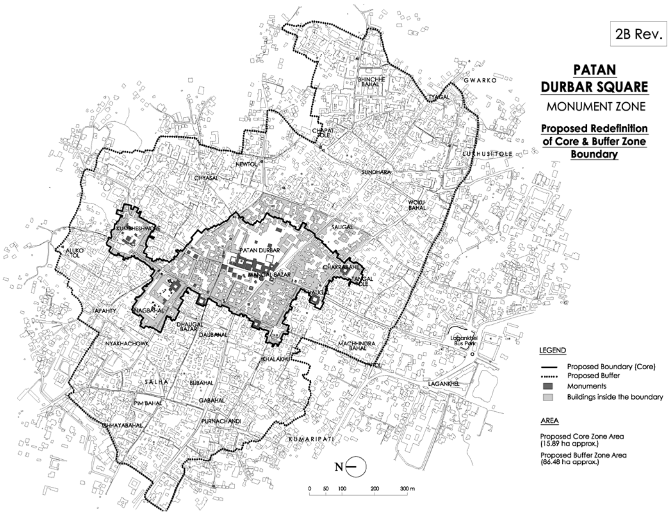


Fig.1.1 Patan Durbar Square core and buffer zone

The historic urban areas, in general, consist basically of two components: existing traditional structures, and the community with open spaces and service (life) lines. Accordingly the research is divided into two broad groups of scope – structural assessment of traditional buildings and disaster mitigation planning of the area. The research outcomes of the both groups are integrated to develop the disaster management plan and the risk mitigation proposal for the historic areas. The major components of the overall research are outlined in Fig.1.2.

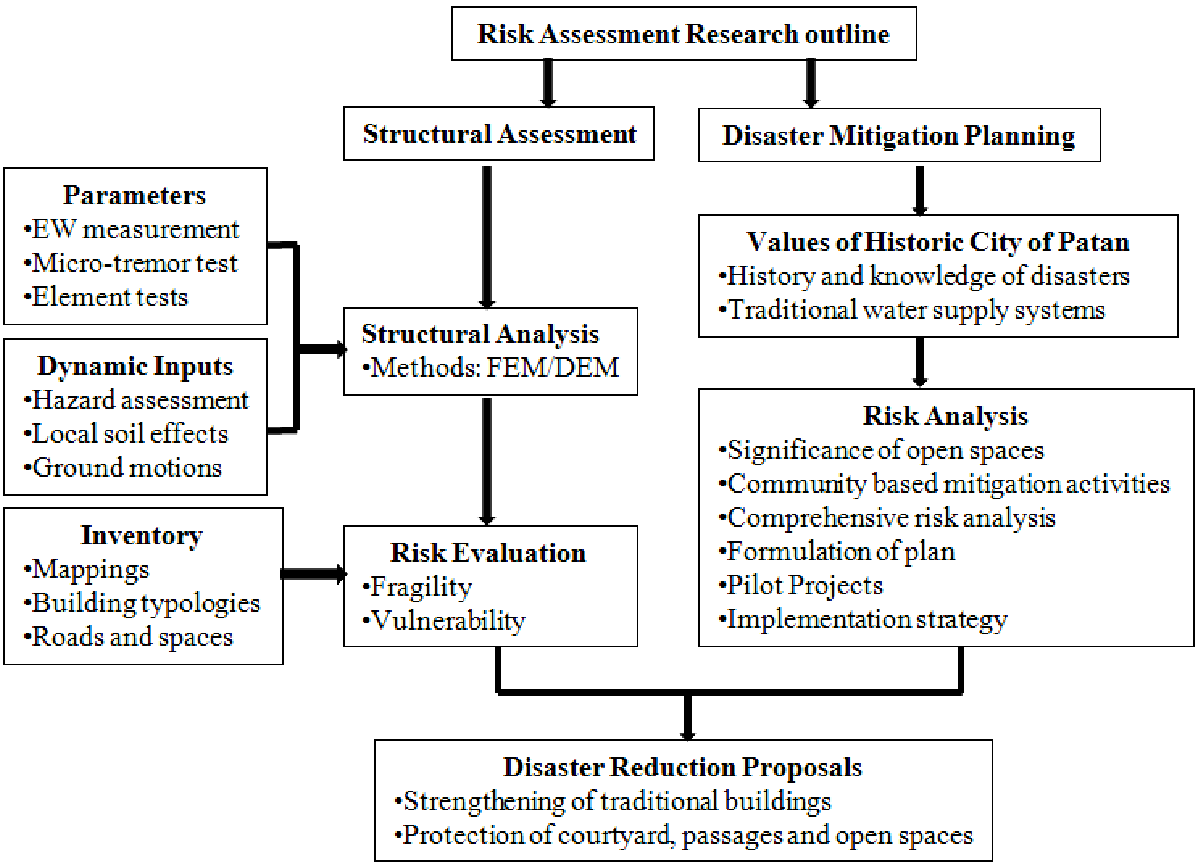


Fig. 1.2 Research components of disaster risk management of historical urban areas

1-1. Seismic activity in Nepal

Hari Ram Parajuli
Prem Nath Maskey
Junji Kiyono

Nepal lies in a high seismic risk zone. It has a long history of destructive earthquakes. All the major earthquakes of past centuries have badly affected the urban areas including the Kathmandu Valley; some have even devastated its cities. A major earthquake of magnitude 8.4 in Richter Scale in the year 1934 (The Great Bihar – Nepal Earthquake) had destroyed many areas of the country, especially large parts of Kathmandu Valley, with a considerable loss of lives and property, although its epicenter was located far off from the Valley, about 240 km in the eastern Nepal. Prior to this also, there have been historical records of large earthquakes in years 1253, 1407, 1681, 1803, 1824, 1833 and 1835. The first major earthquake in Nepal, ever recorded in the documents, dates back to 1253 and is estimated to have had a Richter magnitude of 7.7. The history indicates the considerable damage made by the earthquake of 26 August 1833. This earthquake is believed to have its epicenter close to Kathmandu within or close to the inferred rupture zone of the Bihar Nepal earthquake of 1934. The earthquake of Richter magnitude 6.6 occurred in August 1988 with its epicenter at Udayapur, 165 km away to the southeast from Kathmandu, had destroyed many parts of eastern Nepal with some damages in the Kathmandu Valley. The recent earthquake of September 18, 2011 of Richter magnitude of 6.8 with the epicenter at the border with Sikkim and its shaking effects in many parts of Nepal is the latest event of the seismic activity in Nepal. These historic earthquakes have established the active seismicity of Nepal a long time ago (Fig.1.3).

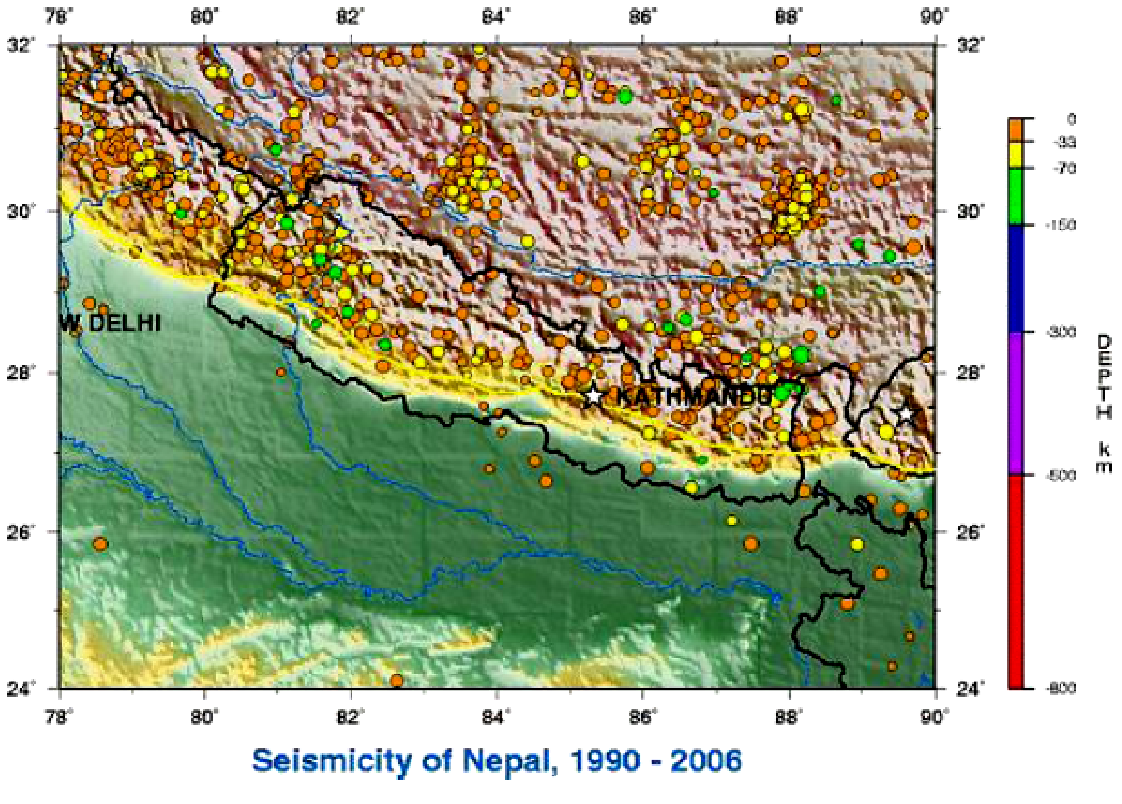


Fig.1.3: Seismicity map of Nepal. Source: 1). *Note:* Each circle represents a recorded earthquake. Most of Nepal's earthquakes are relatively shallow (≤ 33 Km depth) and located near the estimated boundary between the India - Eurasian Plates.

Earthquake records in Nepal since 1253 indicate that Nepal was hit by at least 20 large earthquakes of, the last major earthquake being that of 2011 (Table 1.1) with various digress of damage.

Table 1.1: Historical earthquakes in Nepal. source: 2), 3),4)

Year (A.D.)	Deaths	Damages
1255	Estimated magnitude around 7.7 in Richter scale. One third of the total population of Kathmandu were killed including Abahya Malla , the King of Kathmandu valley	A lot of damages to residential buildings and temples
1260	Many people died, famine after the earthquake	A lot of damages to residential buildings and temples
1408	Many people died	A lot of damages to temples, residential buildings, fissures developed in the ground
1681	Many people died	A lot of damages to residential buildings
1767	No record available on deaths	No record available on damage
1810	Many lives were lost particularly in Bhaktapur	A lot of damages to buildings and temples
1823	No record of deaths	Some damage to houses
1833	Estimated magnitude 7.7, 414 people died in the vicinity of the Kathmandu valley	Nearly 4040 houses destroyed in Kathmandu, Bhaktapur, and Patan in the valley and adjoining Banepa and a total of 18,000 buildings damaged in the whole country.
1834	No good record available	Many buildings collapsed
1837	No good record available	No damage in Nepal recorded but greatly affected Patna and other parts of Bihar, India.
1869	No good record available	No good record available
1897	No good record available	No good record available
1917(1918?)	No record deaths	No record on damage
1934	Estimated Magnitude 8.3 (epicenter, eastern Nepal). 8,519 people died out of which 4,296 died in Kathmandu valley alone	Over 200,000 buildings and temples etc damaged out of which nearly 81 thousand completely destroyed in the country. Max Intensity X. 55,000 building affected in Kathmandu (12,397 completely destroyed).
1936	No good record available	No good record available
1954	No good record available	No good record available
1966	24 people died	1,300 houses collapsed
1980	Magnitude 6.5 (epicenter far western Nepal). 103 people died	12, 817 buildings completely destroyed, 2,500
1988	Magnitude 6.5 (epicenter in SE Nepal). 721 people died	66,382 buildings collapsed or seriously
2011	Magnitude 6.9 (epicenter in NE Nepal). 11 people died in Nepal	Damages in rural houses

In view of the geographical location of the country, seismotectonic characteristics of the region, and the geotechnical condition, it has long been established that Nepal is very much susceptible to large earthquake risks. Bilham et al 4) has indicated that there lies a seismic gap of 500-800 km between the epicenters of the great 1934 Bihar Nepal earthquake and the 1905 Kangra earthquake and this gap has not experienced major earthquakes for the last two centuries. It indicates that a devastating earthquake is inevitable in the long term and it is most likely to occur in the near future.

The seismicity of Nepal is basically attributed to the seismo-tectonic activities in the region. Nepal is located at the boundary between seismically active Indian and Tibetan tectonic plates, which are moving towards each other with about a relative movement of about 20 mm or so per annum. The continued tectonic activities beneath the Himalayan region attribute to the buildup of tectonic stresses accompanying the interpolate collision, and leads to the accumulation of strains and the abrupt failure of rock in the earth's crust along Major fault systems. The major fault systems are recognized as lying parallel to the axis of Himalayan range and consist of Indus Suture Zone (ISZ), the Main Central Thrust (MCT), the Main Boundary Thrust (MBT), and the Himalayan Frontal Fault (HFF). Of these major faults, the Main Boundary Thrust (MBT) separating the Sub-Himalayas from the Lesser Himalayas, and the Main Central Thrust (MCT), separating the Lesser Himalayas from the Higher/Tethyan Himalayas have been identified as the crucial faults in the seismicity of the region. The Main Boundary Thrust (MBT) is the most active major fault system. The tectonically young Himalayas and the continued tectonic activities have made the whole country of Nepal and the surrounding area seismically lying in a very active zone. Fig. 1.4 presents the distribution of the active faults in Nepal.

Fig. 1.4 Active fault distribution in Nepal (Source: 5)

During a study recently conducted, five faults, at various locations and of different lengths and nature, within the valley itself have been identified as active. Some faults located within 20 to 50 km away from the Kathmandu Valley have been found as active, capable of generating an earthquake of magnitude from 6.9 to 7.1 Richter scale. Any earthquake along these faults will be affecting the Valley also.

The features of the active faults within and in the vicinity of the valley indicate that they could be the sources of strong ground motion and surface rupture. Accordingly, apart from the primary seismic hazard, any earthquake may result into three main types of secondary seismic hazards, namely, surface fault rupture, soil liquefaction and earthquake-induced landslide in the Valley. The report describing the effects of The Bihar - Nepal Earthquake of 1934 suggests the occurrence of widespread liquefaction, development of fissures in roads and paddy fields with ejection of water from them. The susceptibility of the soil condition of the valley to liquefaction during an earthquake is strongly evidenced by the description.

Different studies carried out earlier suggest that one of the major factors influencing the level of ground motion within Kathmandu Valley is the site amplification due to unconsolidated sediments. The northern and north-eastern part of the valley, in general, if are characterized by the site amplification similar to that of surface bed rock conditions, the central part of the valley has a relative amplification factor of 4 to 7 as compared to sites in the northern part. The maximum amplification is believed to be confined to certain belt in the southern part of the valley where the high intensity was reported after the Great Earthquake of 1934.

1-2. History and construction system of traditional buildings in Kathmandu Valley

Prem Nath Maskey

The prominent settlements of the Kathmandu Valley date back to the beginning of the Malla Period (13th to 18th centuries), and have remained in the compact forms that were established then. All the important structures created in that period represent the high status of art, architecture and culture, and thus possess the heritage value. These are traditional dwelling houses as well as numerous structures related with public services and religion. Apart from traditional dwelling houses, built in brick masonry, as a perfect part of the town planning, the cities of the Valley are found liberally interspersed and beautified with religious structures. These are different types of temples, viharas, chaityas, and water conduits. The massive palaces with peculiar architecture and decorative facades, public squares with places of rest and gathering (Sattals and Patis), public places for travellers are other monumental structures of importance. These historical structures, all with suitable forms of architecture, and full of arts exhibiting their craftsmanship and architecture in the best manner, represent today the culture and civilization of the country. It is evident from the study of these structures that the basic concept of seismic resistance design (performance against lateral loading) was well understood, and the best available materials and technology were utilized in the construction of the temples and other structures of public importance including dwelling houses. Out of these monumental and traditional structures, the tiered temples are not only common but also possess very peculiar to the Valley. A tiered temple, representing the best of the original Nepali style structure, is usually a structure with diminishing dimensions having a series of pyramidal roofs. Such tiered temples, as well as other historical structures are constructed with basic materials like, brick, rich mud mortar, timber and stone.

The traditional building style of the Kathmandu Valley is represented by the Newar style 6). The style of these houses is more than 2 centuries old making the houses historic monuments, some of these houses are master pieces of the builders' art.6).

In general, the traditional building has a basic unit of brick-walled rectangle of about 6m width and of variable length. The lengths range from a minimum of 1.5 m to 15 meters, although 4 to 8 meters is the general feature. The buildings, typically, are three stories high with a half storey for an attic. However, two and four storeyed buildings are not uncommon. The four storeyed buildings are mostly in the core area of the cities with relatively better economic standard, while the two storeyed buildings are mostly distributed in the city areas with poorer economy. In most of the localities, the buildings are joined end to end, paralleling the streets. Mostly in the community, these buildings are connected with each other in orthogonal directions forming a courtyard, with one or more access gateways in the ground floor to the street.

The structural system of the traditional buildings is based on the load bearing unreinforced brick masonry in mud mortar. Typically, the walls usually consist of three different vertical layers of masonry, with a total width varying from 45 to 75cm. The outer face is made of a wythe of burnt brick, locally known as *ma apa*. For the main front of important buildings or temples, a special wedge-shaped brick (*daci apa*) is employed quite often to create a veneer-like façade. The taper of these bricks results in a varying thickness of the mortar joint from the front to the back. The inner face of the wall is made of one wythe of burnt or even unburnt brickwork (*kaci apa*). Only rarely do those thick walls consist of continuous solid brickwork. The bricks are traditionally laid in mud mortar. The special clay for the mortar is obtained from specified places, where the plastic clay is taken from the deposit after digging for a depth of more than 3m. Commonly, all bricks of the outer face are laid as runners, so that there is no or only little bonding between the different leafs. Cross walls are often simply butt jointed with no interlocking at all.

Frequently timber arcades replace a part of the masonry walls at the ground floor to provide better access to shops or storage space. Here, wooden pillars set above stone piers support a larger spandrel beam on which the upper wall portion is resting. Frames for wall openings such as doors and windows are pieced together from many wooden elements, joined with sophisticated connections. The preassembled free-standing frames are placed on the brickwork and thereafter integrated into the masonry wall.

The foundations of the buildings are typically shallow and often have a depth of about 150 cm below ground level. These foundations are often slightly stepped, and the width of the footing is about 75 cm. The base is generally formed by a few layers of large pebbles or broken stones. This is followed by the usual brickwork, which continues into the upper structure. No damp proofing was used in traditional masonry structures.

The floors are generally timber floors. The timber floors are supported by closely spaced timber joists with a layer of terra cotta tiles or wooden planks. This subfloor is topped with a 10cm thick cover of fine yellow clay. The joists rest on continuous wooden wall plates that are embedded in the masonry. The wedge like wooden pegs are provided at the ends of the timber joists at their supports, normally wall plates or walls, making them the shear-locks to prevent possible horizontal dislodgement. The floor heights of the buildings are about 2.20m.

The roofs in the traditional buildings are double-pitched roofs covered with the local ceramic roof tiles. The roof structure is based on timber rafters or rarely on timber trusses supported on the walls. The roofs normally have wide projections, indicating the most prominent feature of the traditional buildings of the Kathmandu Valley. These roofs, which typically protrude more than one meter from the façade; in the case of larger buildings and temples this can be more than 2 meters.

The traditional buildings are characterized by a limited numbers and sizes of openings for doors and windows, which are tied with the walls by timber ties at strategic levels. Almost all the door and window frames are provided with double frames. All the decorative members also function as important structural elements. Timber is extensively used in windows, doors and almost all the structural elements other than brick walls.

The traditional architecture and construction of the buildings of the Kathmandu Valley Malla were extensively developed during the Malla Period (1200-1768 A.D.). The numerous monumental structures and historical buildings were constructed in this Medieval Period. All these different types of structures have their own distinct character and utility, however, these have a common feature of traditional materials and technology. These structures were built with the best materials available with the best workmanship and knowledge available at the time of their construction. However, the basic materials like bricks and mud mortar, as known, have limited strengths to withstand earthquake excitation, making vulnerable to earthquake excitation.

The traditional building architecture of the Kathmandu valley remained relatively unchanged from the 16th century until the beginning of westernization in the middle of the last century. Other building types, such as palaces, monasteries, houses for priests or merchants, and shrines are similar in terms of building materials and construction principles, although different in size and splendor 6) 7).

The style of the traditional buildings of the Kathmandu Valley did not change much in the periods following the Malla Period. During the early part of the Shah Period (1768 A.D.- to date), particularly during the period of (1806-1837A.D.), the then Prime Minister Bhimsen Thapa first introduced the exotic architecture of European style in the Kathmandu Valley. During the Rana Period (1846-1951 A.D.), the then powerful rulers introduced the neo-classical architecture during in a large scale in the Kathmandu Valley. The introduction of these new architectural styles is apparent in the construction of new palaces and residences of the elite group only. The influence of the exotic European style of architecture and the neo classical architecture in the common traditional building is not remarkable except for some forms of the traditional elements and the building materials. The influence in the traditional buildings of the Kathmandu Valley can be assessed by looking at the three main types of buildings in terms of the finish of their facades. The original and the oldest or medieval style of a common traditional building are characterized by a horizontal window style with square latticed openings (decorative *Sajhya* of the Malla style). About two hundred years ago the lattice window style started to change towards a more vertical form, and the *Sajhya* became less ornamented (the Shah style). Before the end of the century the windows started being lighter and larger, resulting into French windows, making the height of the windows almost one story high, and the latticed form of window and so the Sajhya disappeared (the Rana style). The increase in the opening sizes and removal of the latticed structures in the frames, as the results of the changes, decrease the lateral stiffness and the shear areas of the walls, making the traditional buildings more vulnerable to the seismic excitations.

References

1) USGS http://earthquake.usgs.gov/earthquakes/world/nepal/seismicity.php
2) UN/UNCHS: Seismic hazard mapping and risk assessment for Nepal. Unpublished Report submitted to His Majesty's government of Nepal, Ministry of Housing and Physical Planning, 1993.
3) Pandey, M.R. and Molnar, P.: The distribution of intensity of the Bihar-Nepal earthquake of 15 January 1934 and bounds on the extent of the rupture zone, *Jour. Nepal Geol. So*c., Vol. 5(1), pp. 22-44, 1988.
4) Bilham, R.P, Bodin, P. and Jackson, M.: Entertaining great earthquakes in Western Nepal: Historic inactivity and geodetic test for development of strain. *Journal of Nepal Geological Society*, 11, 73-78, 1995.
5) Upreti, B.N., Kumahara, Y. and Nakata, T.: Paleoseismological study in the Nepal Himalaya – present status, *Proceedings of the Korea-Nepal Joint symposium on slope stability and landslides*, April, 1, pp. 1-9, 2007.
6) Slusser, M.: *Nepal Mandala*, Princeton, Princeton University Press, 1982.
7) Hutt, M.: Nepal Gartmore, Kiscadale Publications, 1994.

2. Seismic Hazard Assessment

Hari Ram Parajuli
Prem Nath Maskey
Junji Kiyono

2-1. Earthquake sources and recurrences

The collision of India into Asia, 50 million years ago, caused Eurasian plate to crumble up and override the Indian plate 1). After collision, the slow convergence of two plates over millions of years pushed up the Himalayas and the Tibetan plateau to their present heights. The Himalayas, approximately two thousand two hundred kilometers long, is the youngest and fragile geology, and high rise mountain in the world extends from west to the east of the northern part of Nepal. It is called Nepal Himalaya and approximately one thousand kilometers in length. Within the narrow width of Nepal (Fig. 2.1), three fault systems, Main Central Thrust (MCT), Main Boundary Thrust (MBT) and Himalayan Frontal Thrust (HFT), pass east to west throughout the length of Nepal.

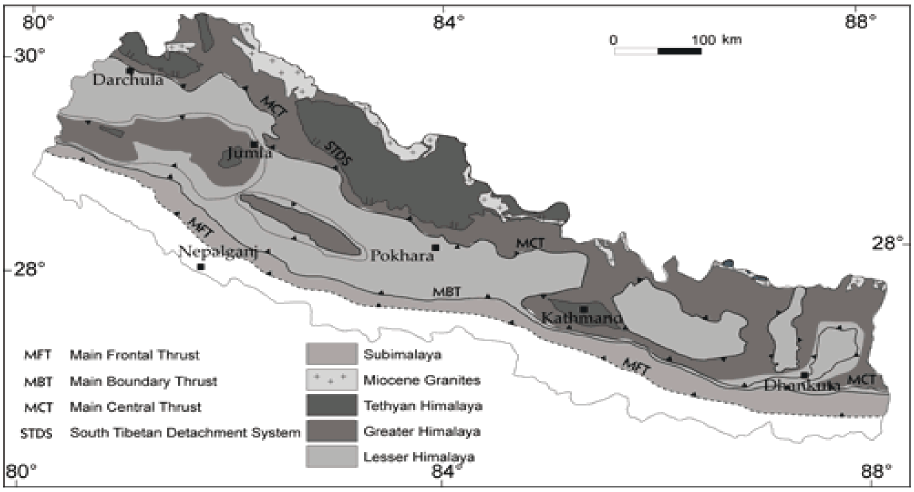


Fig. 2.1a Geological map of Nepal

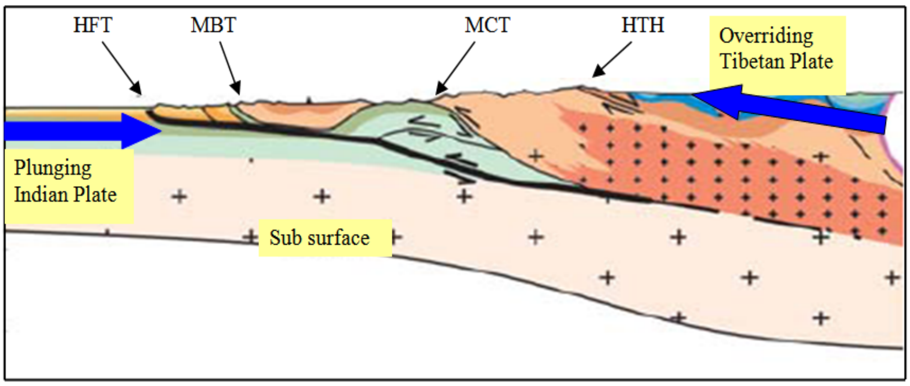


Fig. 2.1b North-South cross section of Nepal showing depth and faulting systems. Source: 2)

Along the sides of these three greater fault systems and in the Tibetan Himalayan region, ninety two small faults have been identified 3). Most of these faults might have formed during past 10 million years and lie in the interface of Indian and Tibetan plate. The geology of Nepal is also differed by the great faults. The southern part is plane with soft alluvium deposits, the middle part is low rise mountains and northern part is high rise rocky mountains. The convergence of two plates has caused many earthquakes in the past with moment magnitude greater than M8 4,5), which indicates a high rate of deformation and seismicity in the region necessitating urgent need of seismic hazard estimation and mitigation.

Some studies have also been done for Kathmandu and for Nepal. Hazard curve and risk consistent response spectrum was obtained by Maskey and Datta 6) for typical location of Kathmandu valley. Free field ground motion accelerations were obtained considering the soil linearity and non linearity. As a part of preparation of national building code of Nepal, seismic hazard mapping and risk assessment for Nepal 3) was carried out under the ministry of housing and physical planning of Nepal, the output of which is the most specific document addressing the seismic hazard issue of Nepal. It has identified ninety two smaller faults within and around Nepal. The whole area has been divided into three sub zones depending on the earthquake density obtained from available and assumed earthquakes, however, details have not been provided on what basis earthquake data was considered. Seismic hazard assessment has been estimated considering both historical data and assigned characteristic earthquakes after subdividing it into twenty one small zones. It has assigned different coefficients for magnitude frequency relationship to the identified ninety two smaller faults, though details have not been given. BECA 3) further divides the whole study area into small area sources, assigns the different maximum magnitudes for each region, but none of areas has maximum magnitude greater than M8, whereas three earthquakes in excess of M8 have already occurred and one of which is also included in their catalogue. Distribution of earthquake densities in different sub-zones has not been well explained. Division of big area into three types of zones and considering uniform distribution of earthquakes within the zone smears out the earthquakes into greater area and the hazard estimates on that basis will result always a low value. Other issue on that study is attenuation law. It has used Kawashima et al. 7) equation. Large numbers of studies on developing attenuation laws have been done since then. Investigations regarding tectonic slip between Tibetan and Indian plates, earthquake size scaling relations have been done on this decade. JICA 8) undertook a comprehensive study for disaster mitigation of Kathmandu valley. It has pointed out that risk at Kathmandu is so high that great earthquake may occur any time. If an earthquake occurs, damage scenario would be worst and Kathmandu valley cannot be functioning as capital city. Recently, probabilistic seismic hazard assessment 9, 10) for Kathmandu considering intra-plate rate observed from GIS information has been done.

Major issues such as advances in the development of ground motion equations, investigation of magnitude scaling of earthquakes in the Himalayan region, and confirmation of location and size of the great 1505 earthquake, have been done after BECA 3). Thus, a seismic hazard estimate addressing these issues has been presented here. This study covers formation of earthquake catalogue merging the data from various sources, development of magnitude frequency relationship at various locations of the rectangular grids and estimation of probabilistic peak ground and spectral accelerations at various natural periods of the structures.

2-1.1 Earthquake catalogue and recurrences

The historical earthquake data within 300 km radius around Nepal, area enclosed by 26.5-30.5N latitude and 80-89E longitude through 2009 was collected. The earthquake catalogue was formed merging the data from U.S. Geological Survey, National earthquake Information Centre (NEIC) 11), BECA 3), Pant 12), Ambraseys and Douglas 13), Ambraseys and Jackson 5). The earthquakes data have been reported in terms of different magnitudes and or intensity scales. To make uniformity, all data were converted to moment magnitude as per Hank and Kanamori 14), using various relationships as proposed by McGuire 15) and scaling relationship for Himalayan region by Ambraseys and Douglas 13). The plot of these earthquake data with the faults is presented in Fig. 2.2.

The usual practice for seismic hazard analysis is to allocate the earthquakes to the nearest faults, rearrange the data into various magnitude groups and year intervals, develop recurrence relationships, calculate mean rate of exceedence. However, the faults are so close and it is difficult to judge which earthquake belongs to which fault. The other point is there are two categories of faults- greater fault systems (MFT, MBT, MCT, and STDS-Fig. 2.1) and smaller faults, 92 in numbers as defined in BECA 3). The smaller faults has been said to be part of greater one, however, they might be multiple rupture segments of greater fault systems. If only the historical earthquake is considered, on the one hand, it is difficult to allocate them in the fault being close to multiple faults and on the other hand, some of the faults are empty, and some of them hold only few numbers of data which is insufficient to define the recurrence relationship for individual faults.

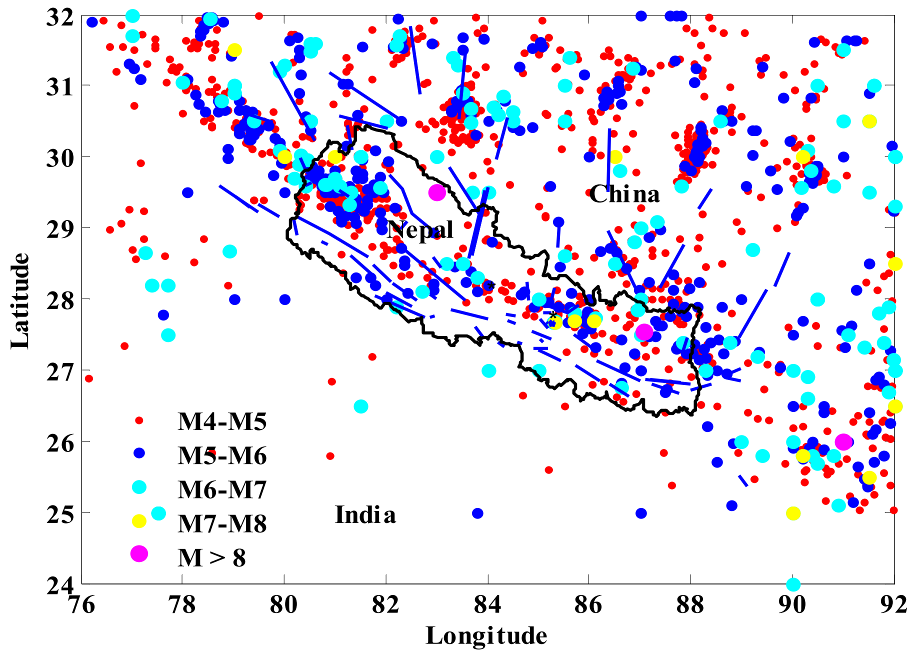


Fig. 2.2 Historical earthquakes (points) and faults (line)

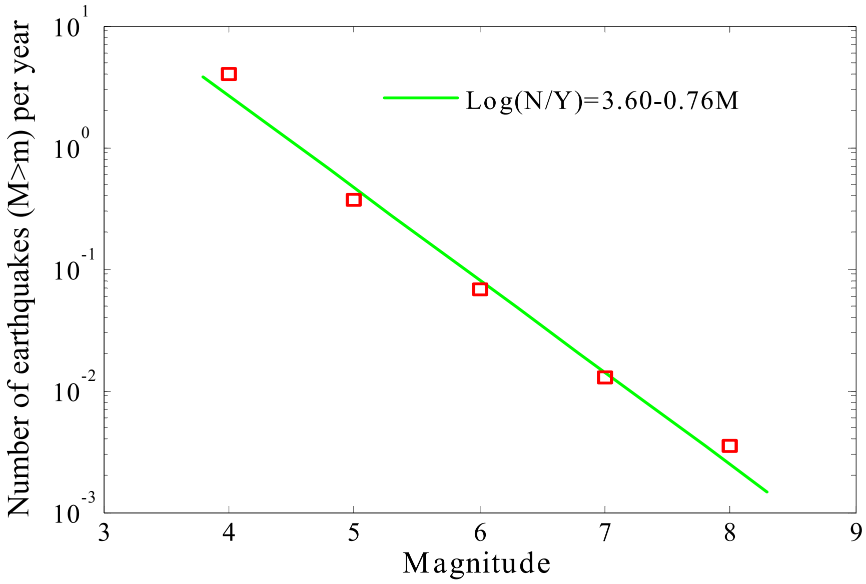


Fig. 2.3 Magnitude - frequency relationship

From engineering prospective, peak ground or spectral acceleration and time history of ground motions at a particular site are necessary and important for design of new and strengthening of existing structures rather than detailing individual small faults. Thus, whole area is divided into grids and cells at intervals of 0.5 and 1.0 degrees in latitude and longitude respectively. Centre of each cell is considered a site. All the earthquakes with magnitude greater than M4 within the radius of 300km from each site are grouped in the range with a 0.5M interval. Earthquake data are not uniformly distributed, only few records available in the early periods are followed by increasing numbers of records towards the end. In order to do completeness analysis of data as proposed by Stepp 16), events are grouped according to small intervals of time. Each range of magnitude is judged separately, and the rate of occurrences of earthquakes exceeding certain magnitude is calculated for all the sites. A total of 197earthquake events with magnitude greater than M4 were found. Considering the lower threshold magnitude M5, and maximum magnitude M8.8 (Lave et al 2005) 30), and with 50% intra plate slip magnitude frequency relationship was developed (Fig. 2.3) following Stepp 1972 29)

It is found that the slope of the recurrence relationship, represented by *b* value is almost unity for the eastern side of Nepal, whereas the value is higher for the western part of Nepal. This means sufficient data are available for the eastern part, while for the western side, either earthquake data is missing or big quake may occur soon as the intra plate slip deficits exist in the region as suggested by Bilham 17).

2-1.2 Attenuation of Ground motion

Ground motion attenuation relations so far developed, can be categorized into few groups, these are for shallow crustal earthquakes in active regions, shallow crustal earthquakes in stable regions and subduction zones focusing America and Japan where large earthquake database is available.

No earthquake attenuation relations have been developed so far for the Himalayan region specifically because of unavailability of adequate earthquake data. Hence the available empirical relationships developed for other regions are reviewed for the suitability according to the tectonic conditions of Nepal. Out of these attenuation relationships, the equations for subduction zone developed by Crouse 18), Fukushima and Tanaka 19), Molas and Yamazaki 20), Youngs et al. [21), Gregor et al. 22), Atkinson and Boore 23)], Atkinson and Boore 24), Kanno et al. 25), and Zhao et al. 26), which support the tectonics, geology and faulting systems of Nepal are studied.

Youngs et al. 21) has been developed from worldwide seismic environment including Crouse 18) catalogue. Zhao et al 26) relation uses Fukushima and Tanaka 19), and Molas and Yamazaki 20) and is derived from Japanese earthquake database. Kanno et al. 25) relationship has also been developed based on Japanese catalogue adding shallow crustal earthquakes from outside Japan. Atkinson and Boore 23) compiled the database of both Youngs and Crouse, added many recent earthquakes data from Japan through 2001, formed four times bigger database for subduction zone events and developed new ground motion relationship. Gregor et al. 22) relationship also has been developed for Cascadia subduction zone. Both attenuation equations have focused on Cascadia fault geometry and ground motion parameter is estimated based on the fault distances. Based on the evaluation of these relationships in relation with the tectonic environment of Nepal, five attenuation laws developed by Youngs et al. 21), Gregor et al. 22), Atkinson and Boore 23), Kanno et al. 25) and Zhao et al. 26) are selected for this study. Among these, the relationship proposed by Atkinson and Boore 23) predicts the lowest, whereas Zhao et al. 26) predicts the highest value. It is uncertain that in any future earthquake the ground motion will attenuate according to any particular attenuation relationship. Thus, seismic hazard for the region is estimated considering all the five attenuation laws giving equal weight.

In subduction zones, there is possibility of occurring both interface and intra-plate earthquakes. None of the past earthquakes in the Himalayan regions has been categorized as interface or intra-plate earthquake. Regarding the information available in the region, there is shallow angle thrust faults which is very similar situation of subduction interface earthquakes as in other parts of the world. For intra-plate, earthquakes are basically categorized by deep focus and volcanic activities. So, subduction interface ground motion relations are considered in this study.

2-2. Probabilistic seismic hazard analysis

For a site, an area of 600km x 600km is considered as the seismic source, and it is further divided into smaller sub-areas or cells of 10km x 10km size. The distances between the centres of the cells and the site are determined. Only the cells within a radius of 300km from the site are considered for the earthquake recurrence relationship. The annual mean rate of occurrence of the ground motion intensity parameter (PGA) exceeding a certain value of intensity parameter is determined by equation 1.

$$\nu_{y^*} = \sum_{i=1}^{N_s} \nu_{i_{M\min}} \int\int P\left[Y > y^* \mid m, r\right] f_{Mi}(m) f_{Ri}(r) dmdr \tag{2.1}$$

Where, N_s is number of sources in the region,

$\nu_{i_{M\min}} = \exp(\alpha_i - \beta_i m_{\min})$ is total rate of exceedence of threshold magnitude (M=5.0 is taken in this study), with $\alpha = 2.303a$, $\beta = 2.303b$.

$P\left[Y > y^* \mid m, r\right]$ is the conditional probability that chosen acceleration exceeded for a given magnitude (M) and distance (R), and

f_{Mi} (m) and f_{Ri} (r) are probability density functions of magnitude and distance respectively.
Here, M and m are used as the random variable and the specific value for magnitude respectively.

The probability density function for Gunterberg-Richter law with lower and upper bound magnitudes is expressed as

$$f_M(m) = \frac{\beta \exp[-\beta(m - m_{\min})]}{1 - \exp[-\beta(m_{\max} - m_{\min})]} \tag{2.2}$$

Since the recurrence relation is developed for 300km square area, it is applicable for all the cells around the site. However, the earthquake data is not uniformly distributed over whole area (Fig. 2). Higher concentration exists in the narrow zone along MCT, MBT than Tibetan Himalayan and lower concentration is in the southern alluvium.

2-2.1 Earthquake densities

Earthquake density is simply the number of earthquakes per unit area. However, the size of earthquake makes major influence in terms of effects. The effect of a single big event would be far greater than thousands of smaller events. Thus, activity rate based upon size of earthquake is calculated using Kernel estimation method Woo 27). Considering the total rate around the particular site as unity, the fraction of the activity rate, which is earthquake density here, for all the sources around the site, is calculated based on the numbers and sizes of the earthquakes from the nearby cells. The mean activity rate $\lambda(m,x)$, at a cell is taken as a kernel estimation sum considering the contribution of N events inversely weighted by its effective return period which satisfies the condition (eqn. 3), can be obtained from equations 2.3 to 2.6.

$$r \le h(m_j) \tag{2.3}$$

$$\lambda(m,x)_i = \sum_{j=1}^{N} \frac{K(m_j, r_j)}{T(r_j)} \tag{2.4}$$

$$K(m,r)_j = \left[\frac{D}{2\pi h(m_j)}\right]\left\{\frac{h(m_j)}{r_j}\right\}^{2-D} \tag{2.5}$$

$$h(m_j) = H \exp(Cm_j) \tag{2.6}$$

$$\rho_i = \frac{\lambda(m,x)_i}{\sum_{i=1}^{N_s} \lambda(m,x)_i} \tag{2.7}$$

Where, $K(m,x)$ is the kernel function, $T(r)$ is the return period of the event located at a distance of r, $h(m)$ is the kernel band width scaling parameter shorter for smaller magnitude and vice-versa, which may be regarded as the fault length given by Chen et. al. 28) and D is the fractal dimension, taken as 1.7. H and C are constants equal to 1.45 and 0.64. From Fig. 2.2, we can see that the distribution of earthquakes are not only uneven, none of the data has fallen near some faults. The faults are geological evidences of sources of earthquakes, even though, earthquakes data may not have fallen in short time span. Thus the activity rate may be based partly on the geological and partly on the historical data as suggested by Woo 27), and the density of each cell is calculated from eqn. 2.7. The earthquake densities are illustrated in Fig. 2.4, in which the historical earthquakes and faults as well as the densities calculated from earthquake data, and faults individually, and combination of both are shown. For each fault, an equivalent earthquake represented by its maximum magnitude as given by Wells and Coppersmith 29) was assigned to calculate the density.

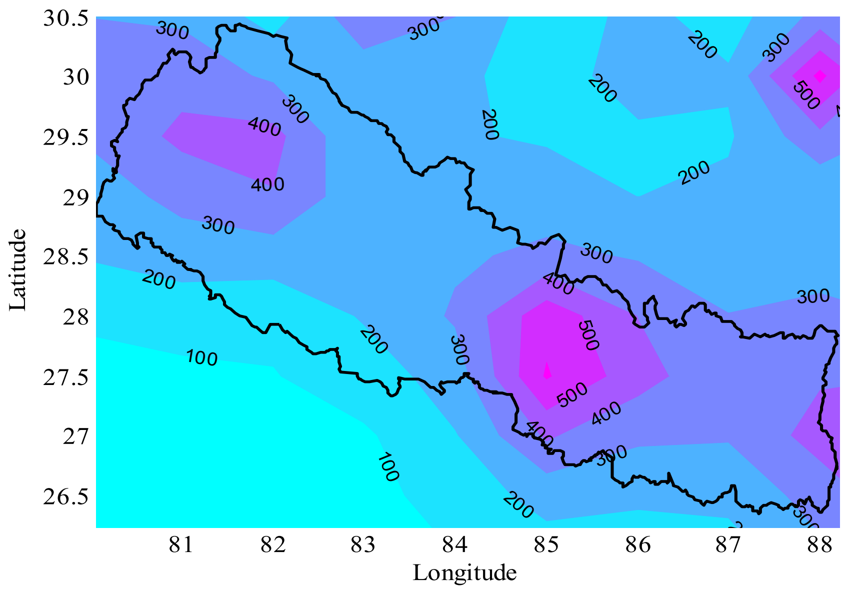


Fig. 2.4 PGA soft soil (5% damping and 10% in 50 years).

The densities calculated from the historical earthquakes are higher around the regions where great earthquakes have occurred in the past, whereas the densities calculated from maximum magnitude of fault are higher around the faults which account for the future possibilities of occurrences even though there might not be the records of historical earthquakes and combination of two accounts, both past history and future possibilities.

The magnitude is divided into 0.1M and distance into 10km intervals. N_m and N_r are the total numbers of magnitude and distance bins. Thus, the mean rate of occurrences can be obtained by eqn. (2.8).

$$\nu_{y^*} = \sum_{i=1}^{N_s}\sum_{j=1}^{N_r}\sum_{k=1}^{N_m} \nu_{i_{M\min}} \rho_i P\left[Y > y^* \mid m, r\right] P\left[M = m\right] P\left[R = r\right] \Delta m \Delta r \tag{2.8}$$

2-2.2 Maximum magnitude

In the seismic hazard assessment another important parameter is maximum magnitude. In the earthquake catalogue, maximum earthquake size is M8.2. However, recent study by Lave et al. 2005 30) has shown that big earthquakes of magnitude M8.8 must have occurred in the same area where 1934 Nepal-Bihar earthquake occurred. Though, it still requires further investigation to determine its exact size and location, it give there is possibility of bigger earthquake. Thus, in this study, maximum earthquake is considered as M8.8.

2-3. Probabilistic spectra

The mean rate of exceedence of peak ground accelerations and the spectral accelerations are calculated using all five attenuation laws. Assuming the occurrence of earthquake according to the Poisson's process, the accelerations for three probabilities of exceedence in 50 years are calculated with all the five attenuation laws for each sites, and the integrated value was obtained giving equal weights to each of the attenuation laws. The peak ground acceleration corresponding to 10 % of probability of exceedence in 50 years (475 years return period) in soft soil condition for 5% damping obtained at various sites is plotted in Fig.2.4. The values shown in the contours are in gals. PGA for distribution is higher around Kathmandu than in other parts of the country (Fig. 2.4). The PGA of 500 gals near Kathmandu, 400 gals in the western part and around 300 gal in the rest part of the country are obtained from the calculations.

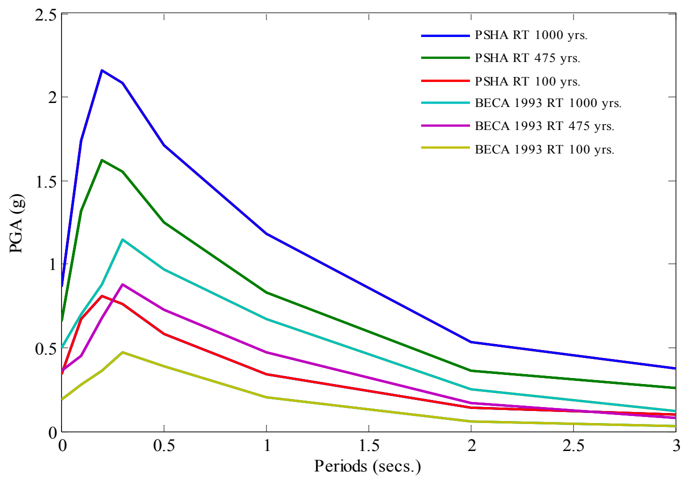


Fig. 2.5 Comparison of spectra for Kathmandu.

To compare the results, the highest values (soil group 2) from BECA 3) from among the three types of soil conditions are taken and the results obtained from this study are plotted together in Fig. 2.5.

The purpose of using return period 100 (39% in 50 years-100 RT yrs.-Fig 5), 475 (10% in 50 years- 475 RT –Fig. 2.5) and 1000 (4.85% in 50 years-1000 RT yrs.-Fig. 2.5) years is to make clarity with BECA 3). In the plot the values in the horizontal axis are natural periods in seconds and in the vertical axis accelerations are in the fraction of g (acceleration due to gravity).

In Fig. 2.4, higher concentration around Kathmandu can be seen than other part of the country illustrating the highest risk. However, in the western part, the PGA seems lower. The reason might be more historical earthquake records are available in the central part than other areas. Kathmandu has very soft soil and ground motions would have been amplified very much than in other places and the people would have though earthquake occurred near or in the Kathmandu which might be the case all the earthquake records centered near Kathmandu. This kind of misinterpretation is very likely because similar case happened in 1934 earthquake. Because soft alluvium deposit in the Ganges plain, heavy damage occurred in Bihar and earthquake was supposed to occur in Bihar as envisaged by Rana 4) in the beginning. But, it was relocated at 10km south of Mount Everest later. However, relocation of earthquakes within few kilometers does not make any significant differences in hazard estimates. The recent investigations of Bilham 17) based upon the GIS reveal that there is a big seismic gap especially in the western part of Nepal Himalaya. The available earthquake data support only one third of its slip rate. There are many possible alternate explanations for that. Earthquakes may have gone missing, the slip between the two plates may be aseismic that could die out without producing any earthquakes or earthquake may occur in near future. Thus, despite the smaller value obtained from the analysis, for design and code implementation purpose, it is better to consider similar values as obtained for the area around Kathmandu.

In the Fig. 2.5, the peaks are not aligning in the same axis. This means that different attenuation laws estimate different peak values of spectral acceleration at different periods. Any specific attenuation law is developed based upon its own hypothesis and tectonic environment. They have different earthquake database, use different distances such as distance from site to faults surface, epicentre, hypocentre, rupture location etc. The depth has also significant effect on the functional form of attenuation law. Thus, slight differences in the results by different attenuation laws are obvious. Almost all of the recent attenuation laws developed for subduction zones have used depth and interface and intra-plate events as separate variables in the equations which have not been used in Kawashima et al. 7) equation.

The site soil classifications also give significant effects to the results. As the period of vibration increases the contributions of bigger events go on increasing. BECA 3) does not consider events of earthquakes whose magnitude is greater than M8, thus it may be the other reason for not aligning the peaks. BECA 3) has increased earthquakes data by 2.5 times and has distributed the density uniformly in subzones. This may be the cause for difference in results.

2-4. Simulation of Ground Motions

2-4.1 Deaggregation of hazard

The main advantage of probabilistic seismic hazard analysis (PSHA) over other representations of earthquake threats is it integrates over all possible earthquake occurrences and ground motions to calculate a combined probability of exceedence that incorporate relative frequencies of occurrence of different earthquakes. The disadvantage of PSHA is that the concept of a design earthquake is lost. There is no single event in terms of magnitude and distance that can be identified as dominating the hazard. However, significant earthquakes are defined from deaggregation of mean rate of exceedence 31,32,33). It reallocates the total contribution of mean rate of exceedence in terms of magnitudes and distances. Then, distance and magnitude of significant earthquake is defined on the basis of highest contribution in deaggregation matrix. This earthquake is also called design earthquake. Acceleration history is developed on the basis of the magnitude and distance. But, a single earthquake never represents the whole return period or alternately, the same earthquake never becomes dominant for structures with different natural periods. Thus, as an alternate method, a procedure to simulate earthquake ground motions which satisfy whole return period is presented. The simulation process requires target spectra, thus, for the study, probabilistic spectra developed for Kathmandu is taken as target. At first, all the contributions in response acceleration are relocated into corresponding distance and magnitude cells. As an example, the deaggregation of the peak ground acceleration for 10% probability of exceedence in 50 years is shown in Fig. 2.6. Considering the distance, magnitude and weight of each cells duration of earthquake ground motion is estimated which is explained in next section.

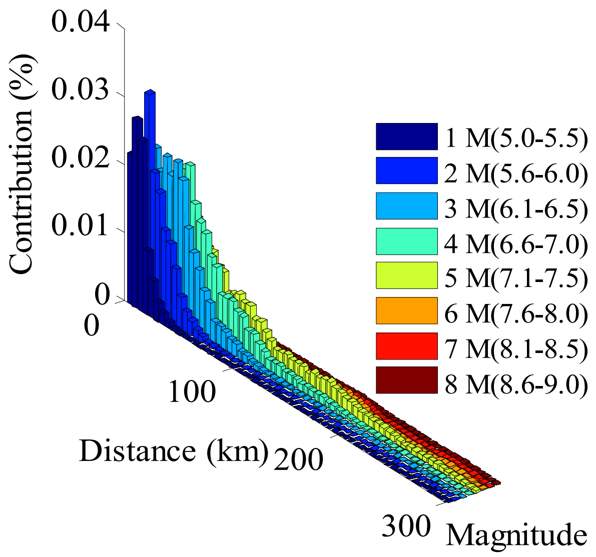


Fig. 2.6 Deaggregation of hazard for peak ground acceleration 10% probability of exceedences in 50 years

2-4.2 Duration and envelope function

The duration of the earthquake can be calculated from the magnitude and the epicentral distance. It means, each earthquake should have separate duration. A probabilistic spectrum consists of many earthquakes. Even within duration span acceleration amplitudes are not uniform. Thus total duration (T_D) is divided into three parts as shown in Fig.2.7. In the first part up to T_B, acceleration will be ascending, between T_B to T_C, it is much effective and after T_C, it starts descending. T_B and T_C are calculated using equations 2.9 to 2.10 following Osaki 34) and T_D is calculated using equation 11 following Kemption and Stewart 35) . It is significant duration which is defined as the time interval across which 5 to 95% of total energy is dissipated.

$$T_B = [0.12 - 0.04(M - 7)]T_D \tag{2.9}$$

$$T_c = [0.50 - 0.04(M - 7)]T_D \tag{2.10}$$

$$\ln T_D = \ln\left[\frac{\left(\frac{\exp(b_1 + b_2(M-6))}{10^{1.5M+16.05}}\right)^{-1/3}}{4.9.10^6\,\beta} + c_2 r + c_1 s\right] \tag{2.11}$$

where, b_1, b_2, c_1, c_2 are coefficients equal to 2.79, 0.82, 1.91, 0.15 respectively, β is shear wave velocity equal to 3.2km/sec. s is soil type and equal to 1 for soil, and zero for rock, M is the magnitude of earthquake and r is the epicentral distance. The envelope function, E(t) presented in Fig. 2.7, is calculated using eqs. 12 to 14.

$$0 \le t \le T_B: \quad E(t) = \left(t/T_B\right)^2 \tag{2.12}$$

$$T_B \le t \le T_c: \quad E(t) = 1 \tag{2.13}$$

$$0 \le t \le T_B: \quad E(t) = \exp\left(\frac{\ln 0.1}{T_D - T_C}(t - T_C)\right) \tag{2.14}$$

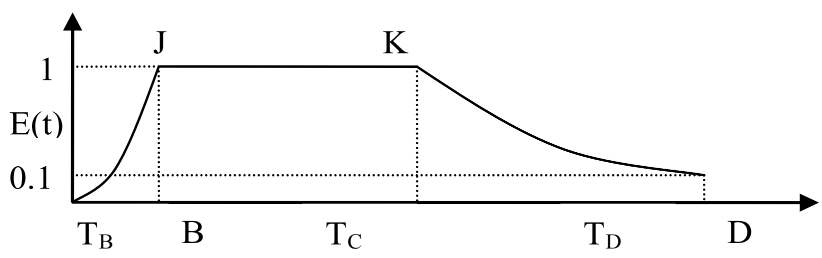


Fig. 2.7 Envelope function

To find out appropriate duration, all accelerations are disaggregated in terms of magnitudes and distances as shown in Fig. 2.6. Disaggregated cells have separate magnitudes, distances and weights. Separate significant durations for all cells using equations 2.9 to 11 were calculated, then, obtained durations are multiplied by corresponding cell's weight obtained from disaggregation and weighted durations are found summing up all bins' durations. We obtain duration for particular value of response acceleration corresponds to specific period. However, we need a single value applicable for all acceleration that fall in specified probability of exceedence in specified period of years. Thus, using similar procedure, durations for all accelerations are calculated. These calculated durations have combined effects of distance, magnitude and weight of deaggregation. For lower accelerations, contribution of lower magnitude earthquakes in hazard is significant but duration is short. For distant earthquakes duration is long but its contribution to hazard is low. For frequent earthquakes, smaller magnitude earthquake also contribute more but when probability decreases contribution of higher magnitude earthquake increases. In an average, the duration remains almost same for all earthquakes which fall in same return period. Thus, the weighted durations corresponding to same return period are in close margin. Then weighted average duration is calculated from all accelerations. For spectra shown in Fig. 2.4, durations are presented in Table 2.1.

Table 2.1 Calculated durations

Durations	40% in 50 yrs	10% in 50 yrs	5% in 50 yrs
TD	39.0	50.0	53.0
TB	5.00	5.50	6.00
TC	18.0	26.0	28.0

2-4.3 Design earthquakes

The design earthquakes are simulated for calculated durations (Table 1). The total duration is first divided into small interval dividing by number N (equation 7). Using incremental time, Fourier transform pair C_k and x_m are evaluated through equations 2.16 to 2.19 as proposed by Osaki 34).

$$\Delta t = \frac{T_D}{N} \tag{2.15}$$

$$C_k = \frac{1}{N}\sum_{m=0}^{N-1} x_m \exp\left(-i\left(2\pi km/N\right)\right),\ \text{k=0, 1, 2, \dots , N-1} \tag{2.16}$$

$$x_m = \sum_{k=0}^{N-1} C_k \exp\left(i\left(2\pi km/N\right)\right),\ \text{m=0, 1, 2, \dots , N-1} \tag{2.17}$$

$$C_k = F_k\left(\cos\phi_k + i\sin\phi_k\right) \tag{2.18}$$

$$R_d = \frac{S_{ds}}{S_{dt}} \tag{2.19}$$

where F_k is Fourier amplitude, ϕ is phase angle, R_d, is ratio of simulated (S_{ds}) to target spectra

(S_{dt}). Envelope plot (Fig. 2.7) was divided into small increments and using equations (2.12 – 2.14) value of envelope function E(t) was calculated. Cumulative value of E(t) was calculated at each interval and these all values are divided by final sum which give cumulative probability density function from zero to one.

Considering total duration as three hundred sixty degree, phase angles for every time steps were determined randomly using probability density function. Phase angles can be obtained from previous earthquake records. However, there are no recorded earthquake acceleration histories for the region. Thus, random phase angles estimated from envelope function were used.

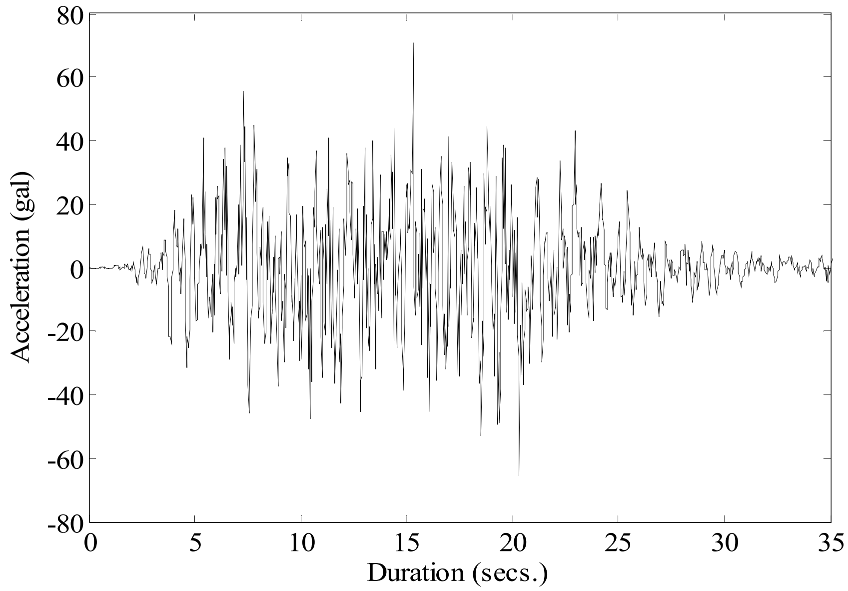


Fig. 2. 8 Simulated acceleration histories for 40% probability of exceedence in 50 years

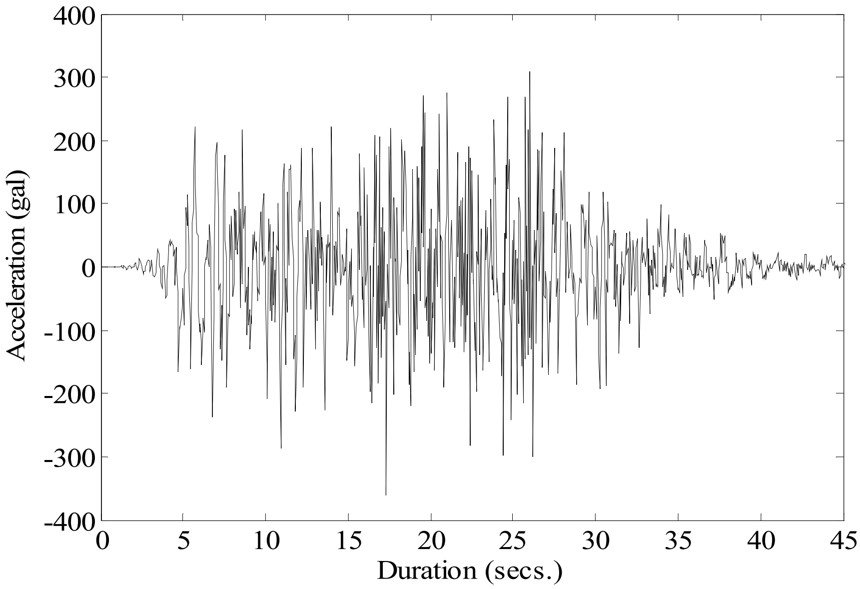


Fig. 2.9 Simulated acceleration histories for 10% probability of exceedence in 50 years

At first, Fourier amplitude F_k is assumed unity. From, Fast Fourier Transform (FFT), accelerations at each interval was calculated. Accelerations were obtained from Inverse Fourier Transform. To make the simulated ground motion similar to natural earthquakes, the acceleration obtained from inverse Fourier transform were multiplied by envelope function. Using the calculated accelerations, response spectra was determined and compared with original spectra called target spectra. Ratio from simulated spectra to target spectra at each interval was obtained. New Fourier amplitude was then calculated multiplying old amplitude by obtained ratio. Again, FFT were calculated and accelerations were determined. The process was repeated until the simulated spectra and target spectra fit well.

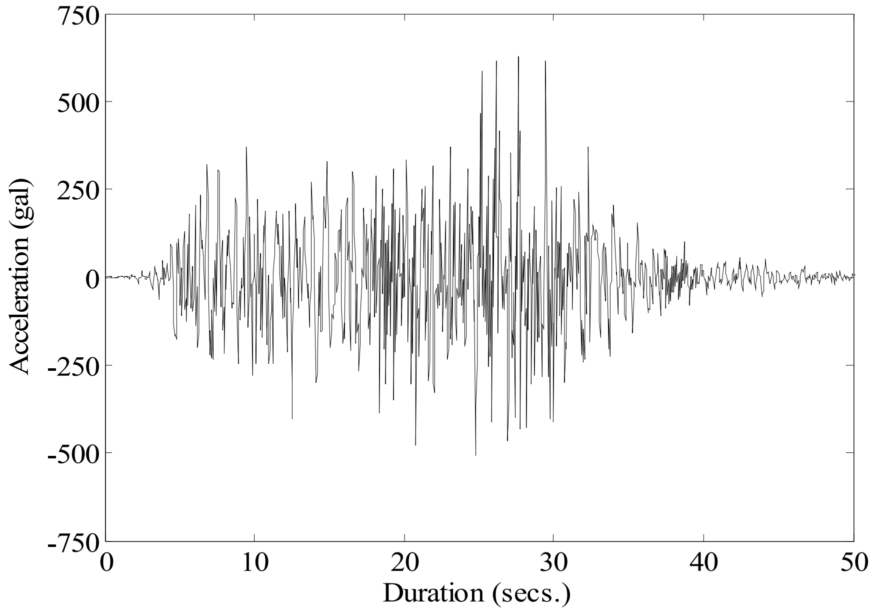


Fig. 2.10 Simulated acceleration histories for 5% probability of exceedence in 50 years

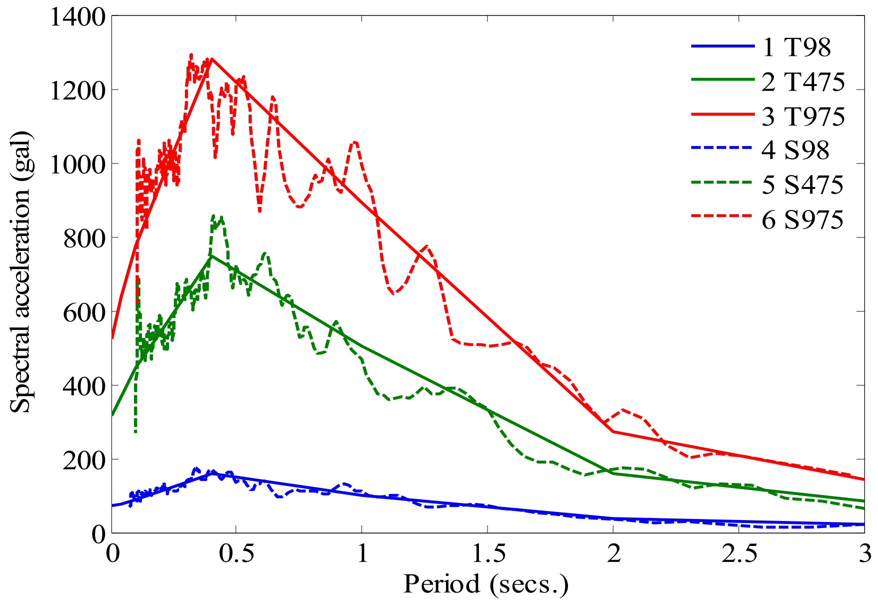


Fig. 2.11 Comparison of obtained and target spectra

The accelerations histories developed by this procedure for three values of the probabilities of exceedence - 40%, 10% and 5% probabilities of exceedence in 50 years are plotted in Figs. 2.8 – 2-

10. From simulated acceleration histories, acceleration response spectra are calculated and plotted against target spectra (Fig. 2.11).

The spectra from simulated earthquake ground motions and originally calculated from attenuation law are in good agreement. In real earthquakes, accelerations start from zero, increase gradually, attains peak values and decrease to zero finally. Simulated earthquakes also look similar to natural earthquakes. Amplitudes and nature of simulated ground motions are totally depend on target spectra.

The earthquake ground motions are very essential for non linear dynamic analyses. However, for the regions like Kathamndu, there are no adequate records of historical earthquake accelerograms which could be used for structural analysis and assessment. Simulation of the earthquakes ground motions compatible to response spectra estimated for specific region may be a rational way out. The proposed method may be useful to estimate hazard where sufficient data are not available. Three separate probabilistic earthquakes which satisfy the target spectra obtained from probabilistic seismic hazard assessment were simulated. These earthquakes can be applied as seismic input for dynamic analyses of different structures.

2-5. Local Soil Effects

The effect of soil sediment overlying the bedrock is substantial in determination of free field ground motion at the surface. It has been well recognized, in general, that soil deposits can significantly amplify the ground motion. Ground response analyses are carried out to modify the ground motions obtained from the hazard analysis for the bedrock taking into consideration the local soil amplifications at a site.

The wide spread destruction caused by the large earthquakes in different parts of the world evidently shows that the local soil amplification effect in most cases is the basic reason for the disaster. For example, in the 2001 Gujarat-Bhuj earthquake in India, there were substantial damages at places even at a distance of 250km from the epicenter 36).

The present section deals with an approach for estimating seismic design parameters like ground surface acceleration time history, response spectrum, design response spectrum etc for the cultural heritage site of Jhatopol, in the vicinity of the World Heritage Site of Patan Durbar square of Kathmandu valley.

2-5.1 Soil Profile Data Acquisition and Interpolation

The initial step for site response analysis involves the collection and validation of the geotechnical and geological information of the site. It includes the identification soil profile information such as number of layers, depth of the water table and layer information such as material name, thickness, unit weight, plasticity index, fines content etc and their damping value, which influence the seismic responses. In the absence of the site specific soil sediment data with required explorations and tests, readily available data from boreholes in the vicinity and from various literatures are used to arrive at the soil profile of the site of concern.

In this study, data of 27 deep boreholes with depth more than 30m are collected from private agencies, government offices, and from literatures. These data basically consist of material of layers, thickness, water table, elevation of boreholes etc. All these collected borehole log data are connected/matched by drawing profiles by hole - to- hole cross-section method. Such technique helps to display logs of the boreholes along the desired direction passing through the site. Moreover, a subsurface geological database prepared by Piya 37) is also used to interpolate the soil profile at the site of interest. Piya had collected borehole data from various organizations during field work of Kathmandu Valley nearly 192 deep boreholes with 45 up to bed rock level, 300 shallow bore holes of 5 to 30m depth. He had developed GIS layer model of the surface and the creation of borehole logical cross-section by using a generalized subsurface geology model with X and Y coordinates and depth values. For this work, he used ILWIS (Integrated land and water information system, 1988) and Rockworks 99 to use borehole data in the generation of a subsurface geological database. ILWIS is used for the generation hole to hole fence diagrams in same direction as done before. This generalized geology was interpolated to a four-layer model and was used as the basis of the soil modeling. The four layers were recent post lake deposits (alluvial), lake deposits, pre-lake deposits and bedrock respectively. The first layer from the surface consisted mainly of sandy and silty sediments, and the second layer consisted of clay sediments. The third layer consisted mainly of gravel. The fourth layer was bedrock, which consisted mainly of limestone, sandstone, slates and phyllites.

Since the site specific geotechnical properties of soil strata are unknown, soil strata is interpolated with the help of geotechnical information obtained from fence diagrams done on the GIS model and plotted hole to hole cross section These two different outputs are cross checked and finally established the soil column for our desired site.

2-5.2 Characterization of the Soil

For the proper modeling of seismic responses of the area it is necessary to identify all required basic input data for SHAKE 2000 software. The basic input regarding soil profile like soil type, thickness, shear wave velocity or shear modulus, unit weight of material. After interpolating the borehole log profile with soil type and thickness at specified site, other character of soil is derived as described in the following paragraphs.

(1) Estimation of Shear wave velocity

Out of 15 various empirical equations, investigated by Ohta and Goto 38), the one relation for shear wave velocity with the depth, soil type and geological age, is selected for use in the study. The selected equation has a correlation coefficient of 0.86. The selected relationship is given by:

$$V_s = 84.36 \times H^{.246} \begin{bmatrix} 1.000 \\ 1.455 \end{bmatrix}_E \begin{bmatrix} 1.000 \\ 1.202 \\ 1.261 \\ 1.412 \\ 1.482 \\ 1.927 \end{bmatrix}_F \quad (2.20)$$

$$\text{Where,}\quad \begin{bmatrix}\text{alluvium}\\ \text{diluvium}\end{bmatrix}_E \begin{bmatrix}\text{clay}\\ \text{fine sand}\\ \text{medium sand}\\ \text{coarse sand}\\ \text{sand and gravel}\\ \text{gravel}\end{bmatrix}_F$$

The shear wave velocity for the bed rock is considered to be approximately 1520 m/s based on the available different literatures.

(2) Density of Soil Strata

The study team on Earthquake Disaster Mitigation in the Kathmandu Valley 39), under Japan International Cooperation Agency (JICA) measured shear wave velocities of five drilled boreholes using P and S waves (PS) logging. The study team recognized the relation between N values and shear wave velocities from previous tests results in Japan and also recommended the same relations for Kathmandu valley and gave the representative bulk density values of the different soil strata with shear wave velocities shown in Table 2.2. The density of soil layer is derived from this table according to the shear wave velocity.

Table 2.2 Shear wave velocities and density for different soil strata in Kathmandu valley (Source: 39)

Soil type	Shear wave velocity (m/s)	Density (KN/m3)	kcf
Clay to clay silt	Vs <175	15	0.093
	175<Vs<300	16	0.099
	300<Vs	17	0.105
Silt to fine sand	Vs <200	16	0.099
	200<Vs<350	17	0.105
	350<Vs	18	0.112
Medium to course sand	Vs <200	17	0.105
	200<Vs<350	18	0.112
	350<Vs	19	0.118
Sand and gravel	Vs <200	19	0.118
	350<Vs	20	0.124

In the case of dynamic shear modulus curves and damping curves, these curves are selected from the list given in shake2000 according the type and properties of soil

2-5.3 One Dimensional Site Response Analysis

One dimensional wave propagation analysis is carried out to determine the ground response of the soil sediment of the study area. For the purpose, the earthquake ground motion acceleration time history at the bedrock, simulated for the site is taken as the input. The ground response analysis is carried out using the software package SHAKE 2000, considering the linear as well as equivalent linear properties of the soil layers. The obtained soil amplification factor is used to determine the free field ground motion parameters.

The locations of the 27 deep boreholes with their depth varying from 30 m to 475m, the data of which were collected from various agencies and from literatures, are presented in Fig. 2.12.

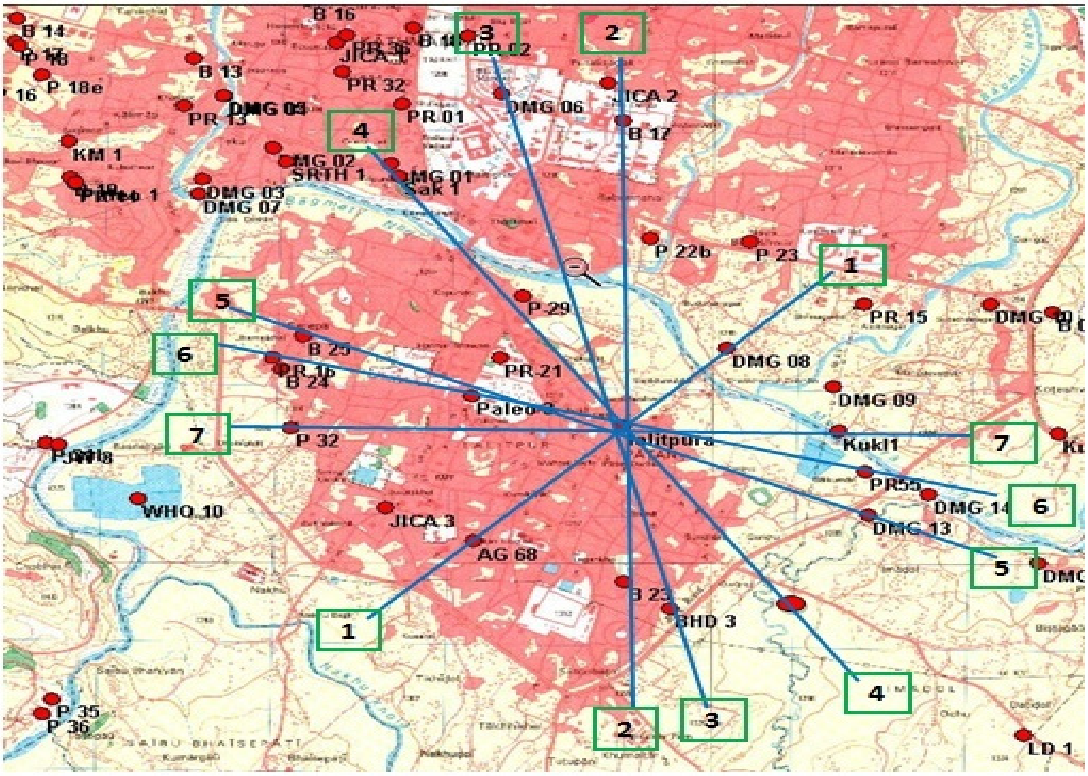


Fig.2.12 Bore Hole Locations and Directions of Different Hole to Hole Cross-sections

The hole to hole borehole log columns profile are made in seven different directions as shown in Fig. 2.12, and the cross section profiles are shown in Figs. 2.13a – 2.13g The fence diagrams are also computed from IWLIS software in combination of Rockworks 99 for same seven directions as described. The borehole log profile cross-sections with fence diagrams of respective direction are shown in Figs. 2.14a – 2.14 g. The soil profile is interpolated for the location of the specified site, based on the above borehole log profile hole to hole sections and fence diagrams, and presented in Table 2.3.

Table 2.3 Soil Layer Interpolation Detail

Layer No.	Layer Type	Interpolated from Sections	Remarks
1	Sandy Clay	1-1 and 2-2	Thickness of layer is taken as average thickness of used interpolated sections
2	Fine to Coarse Sand	1-1 and 2-2	
3	Black Clay	1-1, 2-2, 3-3, 4-4 and 7-7	
4	Sand and Gravel	2-2, 4-4 and 7-7	
5	Black Clay	2-2 and DMG8-B23	
6	Bed Rock		

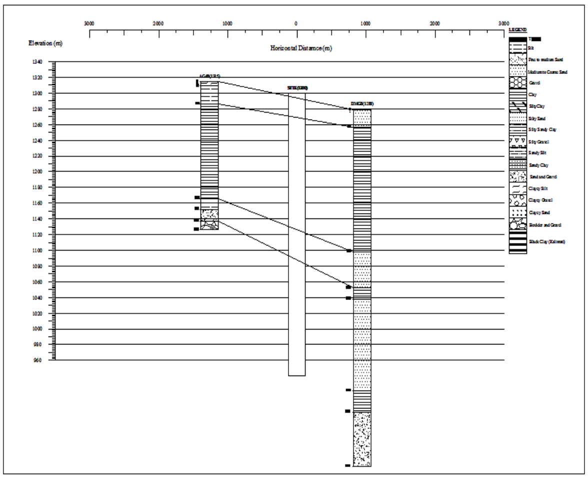


Fig.2.13 a Borehole-log columns along section 1-1

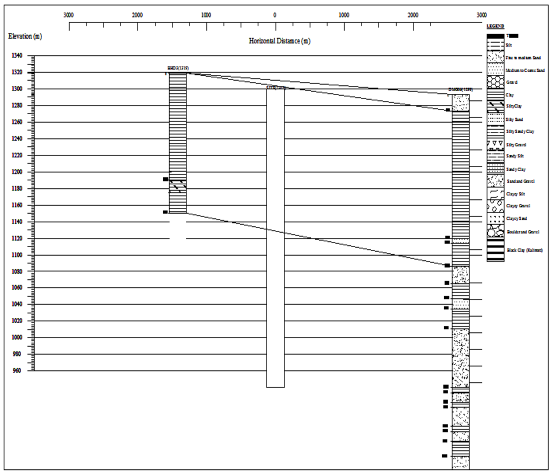


Fig.2.13 c Borehole-log columns along section 3-3

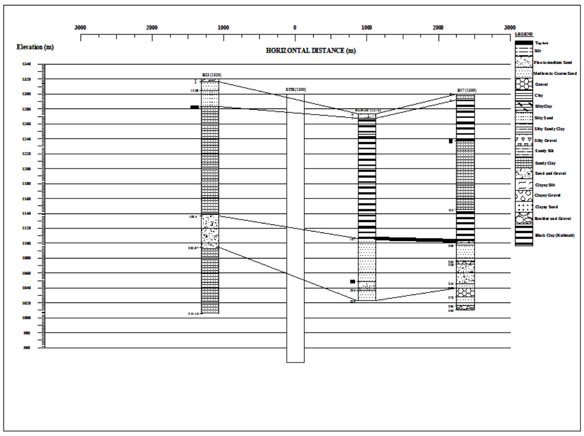


Fig. 2.13 b Borehole-log columns along section 2-2

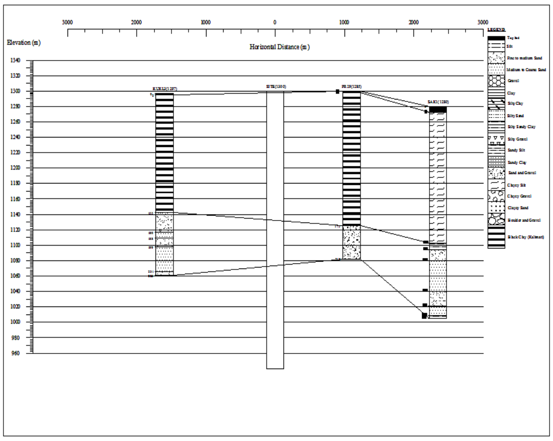


Fig.2.13 d Borehole-log columns along section 4-4

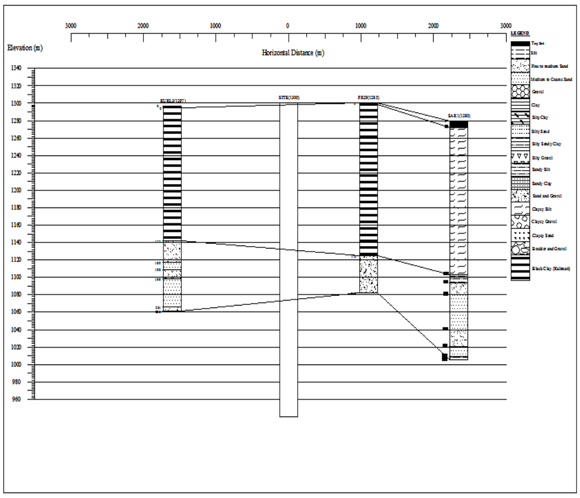


Fig. 2.13e Borehole-log columns along section 5-5

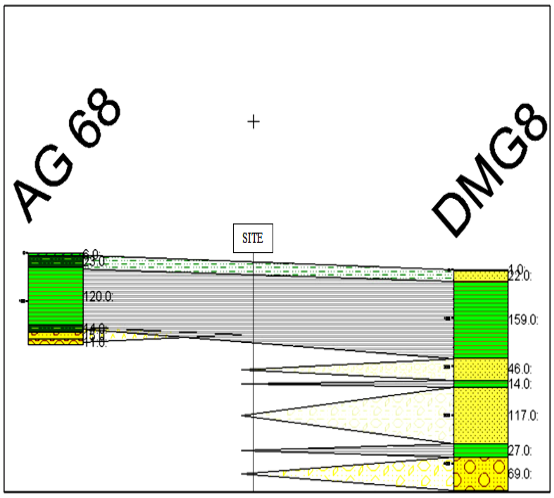


Fig. 2.14a Fence Diagram along Section 1-1

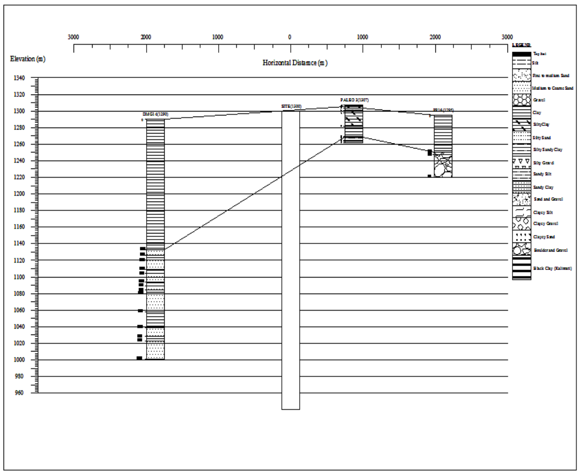


Fig. 2.13f Borehole-log columns along section 6-6

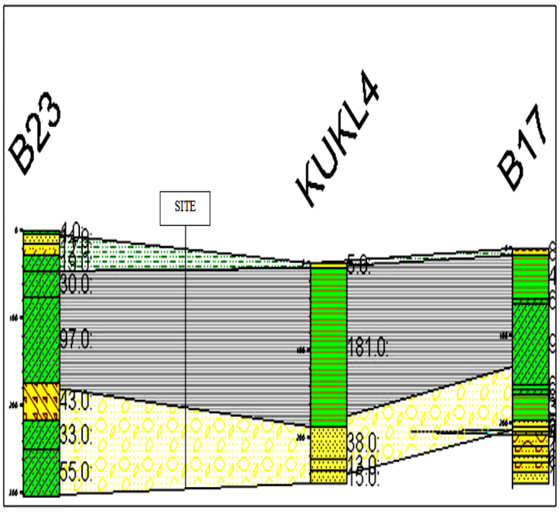


Fig. 2.14b Fence Diagram along Section 2-2

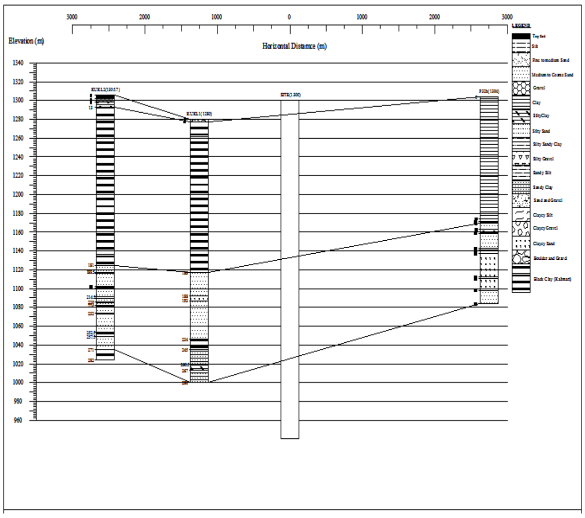


Fig.2.13g Borehole-log columns along section 7-7

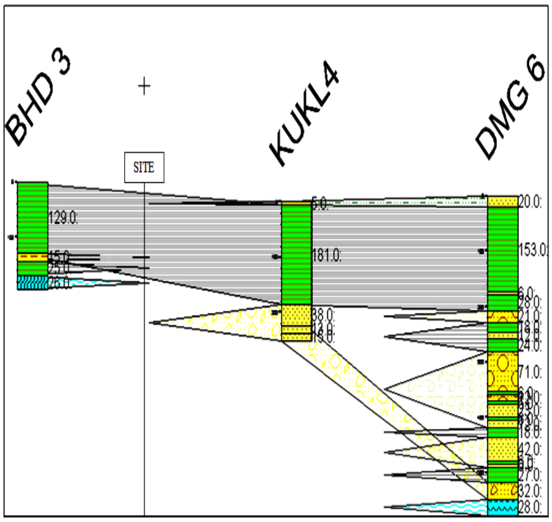


Fig. 2.14c Fence Diagram along Section 3-3

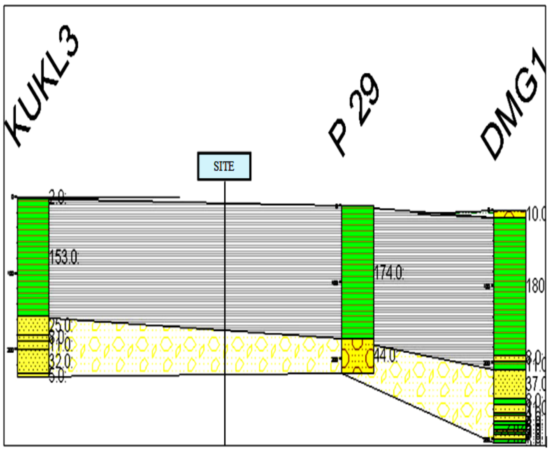


Fig. 2.14d Fence Diagram along Section 4-4

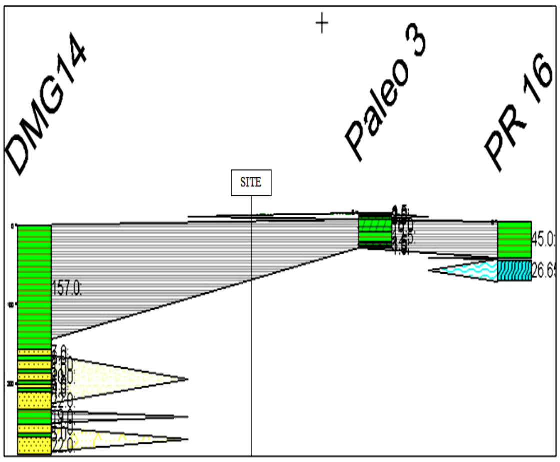


Fig. 2.14f Fence Diagram along Section 6-6

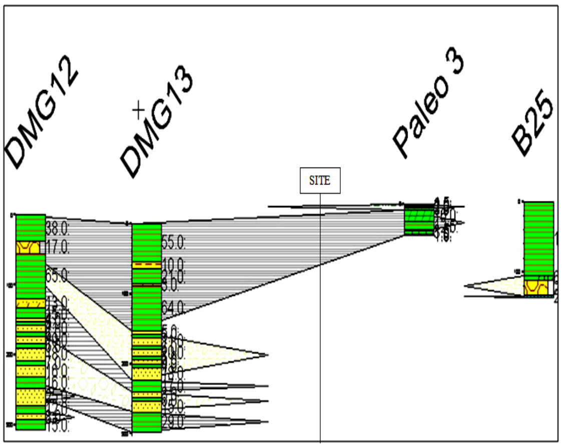


Fig. 2.14e Fence Diagram along Section 5-5

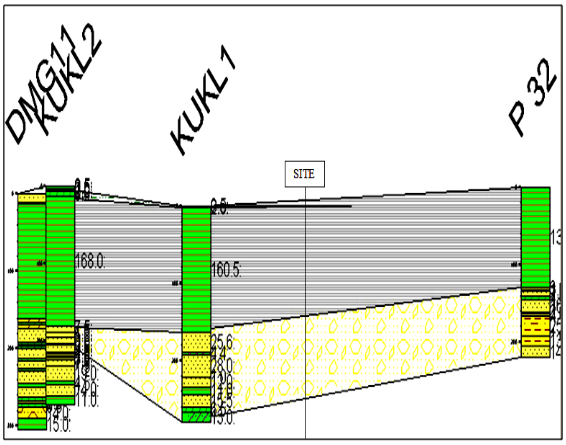


Fig. 2.14g Fence Diagram along Section 7-7

The interpolated soil profile with type and thickness of soil for the site is shown in Fig. 2.15.

After finding the soil profile for the site, the total depth of the soil sediment is divided into 96 sub-layers. The thickness is given to each sub-layer such that the characteristic site frequency (fundamental frequency) is greater than the maximum value of frequency content in the input rock motion so that the layer can propagate the motion. Thickness of sub-layer:

$$H = Vs/(4*f_{max}) \tag{2.21}$$

Where, f_{max} is the highest frequency that the layer can propagate, and

V_s is the shear wave velocity of the layer.

The shear wave velocity for each sub-layer is estimated using relations described in (1) above, and density is assigned to those sub-layers as described in (2). The properties of the soil sub layers including the shear wave values and the densities, along with the simulated seismic ground motion at the bedrock are used to carry out the ground response analysis. The shear modulus reduction curves and damping curves used for different soil types are shown in Fig. 2.16. The water table level in the analysis is taken as 3m from surface, which is the value found for the construction site in the vicinity. The Linear as well as Equivalent linear site response analysis are carried out in SHAKE 2000 using simulated earthquake motion for the bedrock of the same location taken as input motion shown in Fig. 2.17. The Fourier amplitude spectrum of the simulated ground motion acceleration time history at the bedrock is shown in Fig. 2.18. The ground motion acceleration time histories, Fourier amplitude spectra, and response spectra for each site response analysis are computed at the surface of the site.

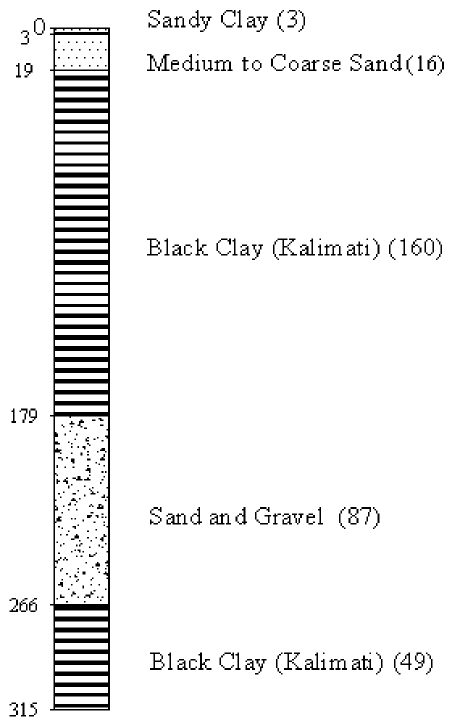


Fig. 2.15 Interpolated Borehole log at the Site

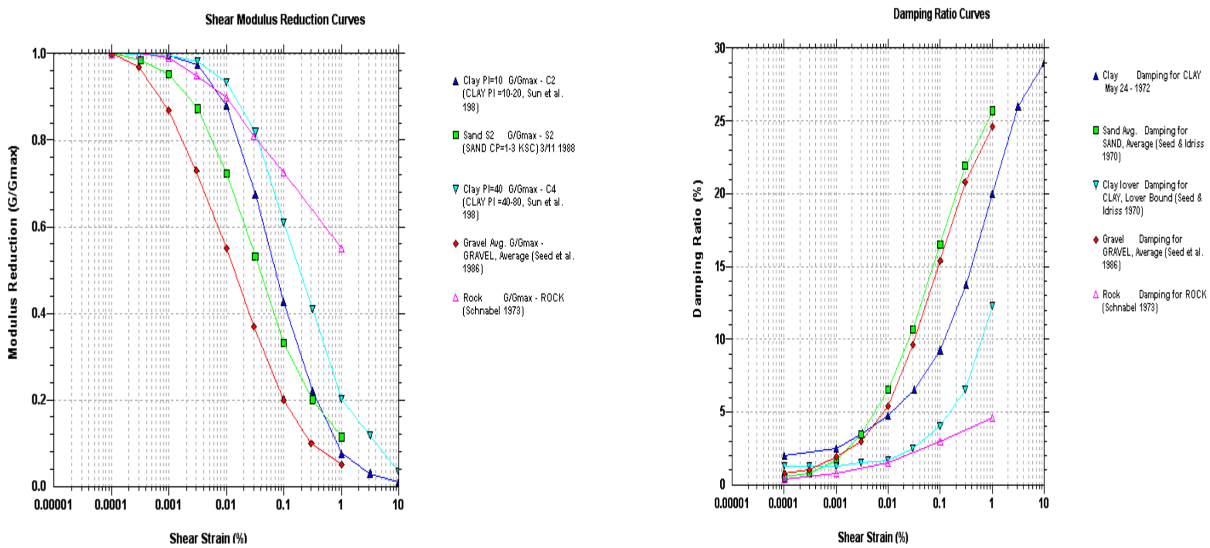


Fig. 2.16 The dynamic material properties and their relation to the shear strain

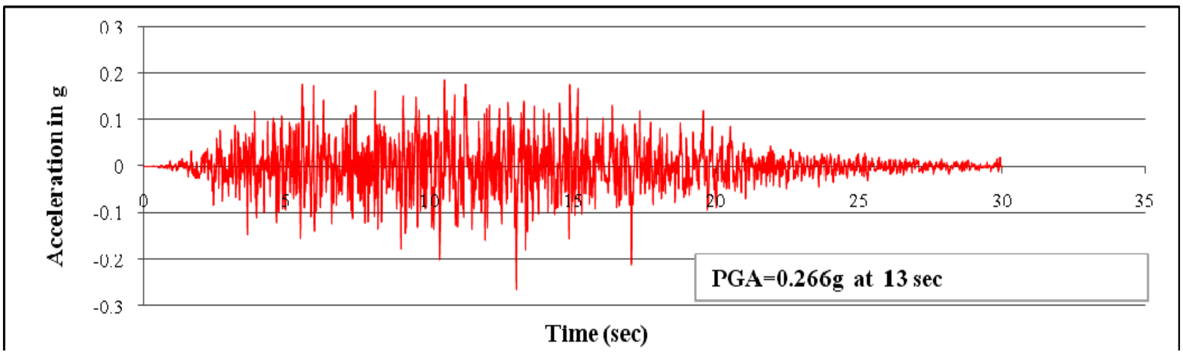


Fig. 2.17 Simulated Acceleration Time History at the Bed Rock (For 475 Years Return Period)

The time histories, Fourier spectra and spectral acceleration at the surface computed from both linear and equivalent linear site response analysis are shown in the Figs. 2.19 – 2.24.

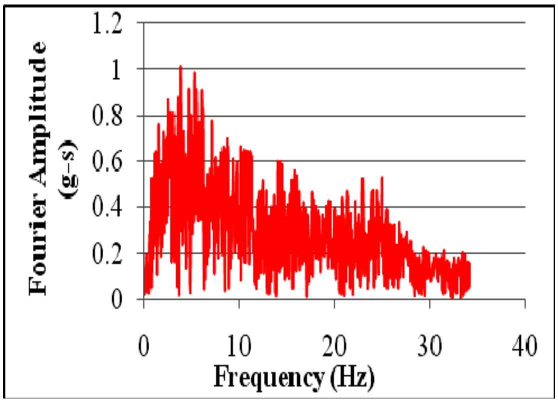


Fig. 2.18 Fourier Amplitude Spectrum of Simulated Earthquake

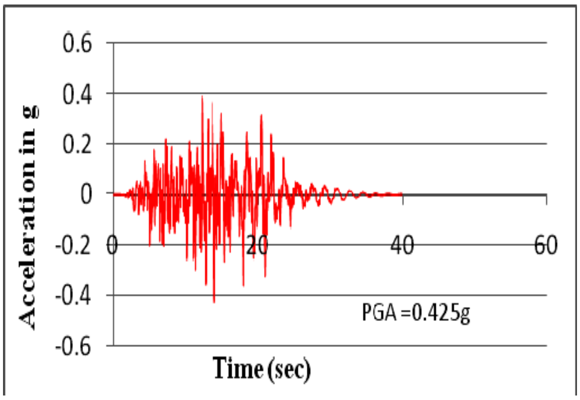


Fig. 2.19 Acceleration time history at surface (Linear site response)

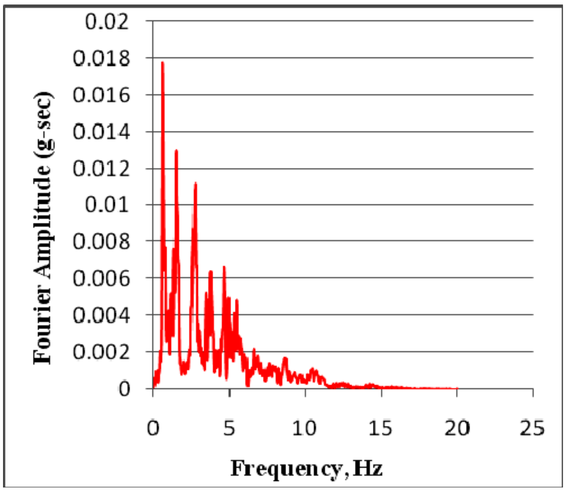


Fig. 2.20 Fourier amplitude spectrum (Linear site response)

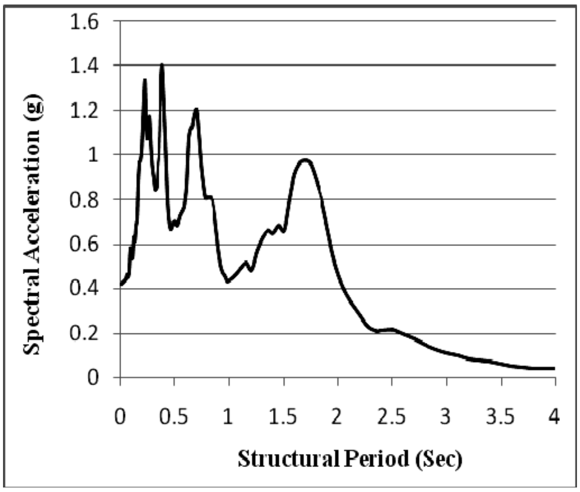


Fig. 2.21 Response Spectrum (Linear site response)

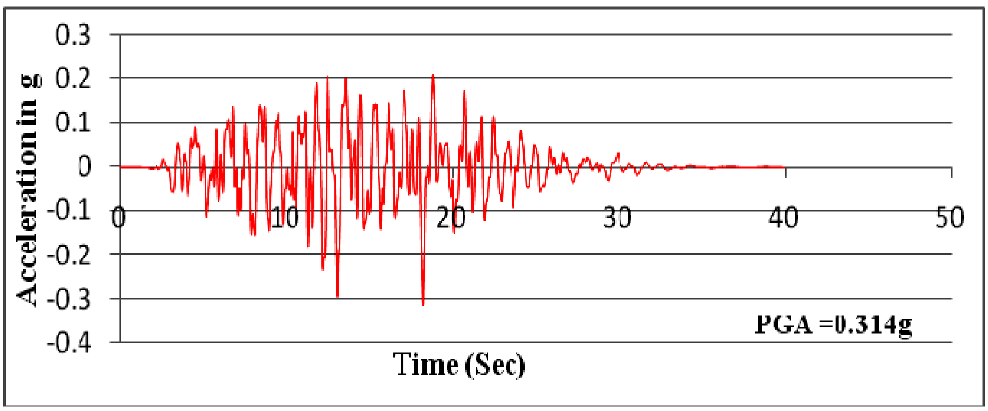


Fig. 2.22 Acceleration time history at surface obtained from Equivalent site response analysis

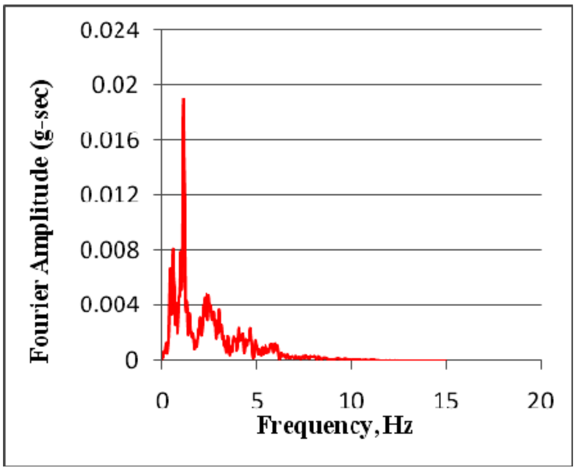


Fig. 2.23 Fourier amplitude spectrum (Equivalent linear site response analysis)

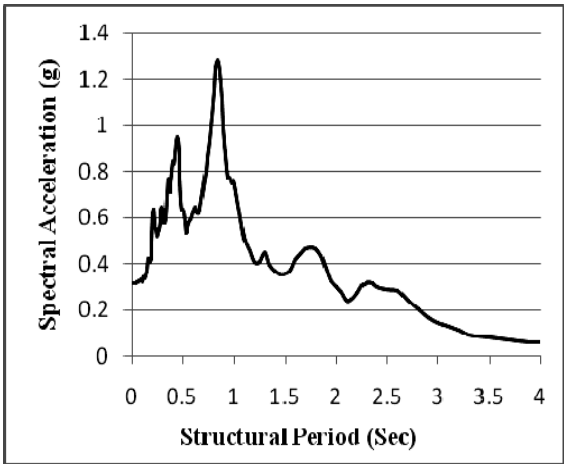


Fig. 2.24 Response Spectrum (Equivalent linear site response Analysis)

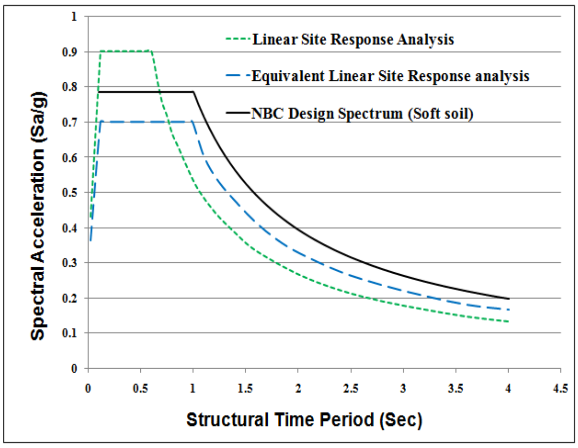


Fig. 2.25 Comparison of design response spectra (Newmark-Hall's Method) 40) with NBC 41) design spectrum (Soft soil)

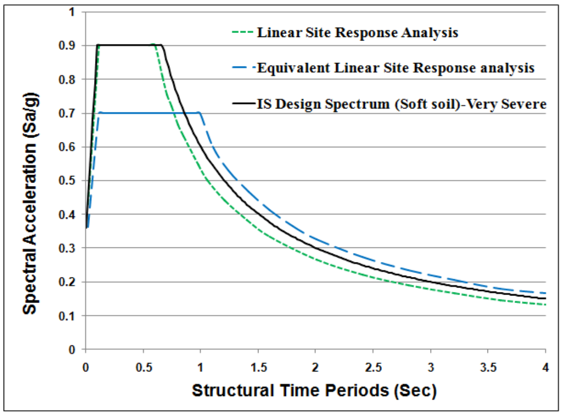


Fig. 2.26 Comparison of design response spectra (Newmark-Hall's Method) 40) with IS code 42) design spectrum (Soft soil)-Very severe

The response spectra are smoothened by Newmark- Hall's method 41) to obtain design response spectra for both type of site response analysis, and are compared with Nepal Building Code design spectrum 42) and IS code design spectrum 43) as shown in Fig.2.25 and Fig.2.26 respectively. The Newmark-Hall design spectrum is obtained by using nonlinear software. The velocity time history and displacement time history of surface motion required for the input ground motion for Nonlinear are calculated from Seismosignal software.

The acceleration time history at surface as shown in the Fig. 2.19 obtained from linear site response analysis gives peak ground acceleration 0.425g which is 1.6 times greater than the PGA at bed rock. As seen in Fig 2.20, the Fourier Amplitude spectrum at surface, the majority of energy is concentrated between the frequencies 0.5 Hz to 2.5 Hz. It shows that the linear effect of soil subjects more seismic energy to the structures having natural time period ranging from 0.4 sec to 2 sec.

The acceleration time history at the surface as shown in Fig. 2.22 from equivalent linear analysis gives peak ground acceleration 0.314g which is 1.18 times greater than the PGA at the bed rock. As seen in Fig 2.23, the Fourier amplitude spectrum at surface, the majority of energy is concentrated at frequency of about 1 Hz. Comparison of the Fourier amplitude spectrum with that at bed rock shows that the soil conditions greatly influence the frequency content of the earthquake motion. The results show that the board banded spectrum is converted into narrow banded. It shows that the nonlinear effect of soil subjects more seismic energy to the structures having natural time period of about 1 sec. The amplification value of this response is less than that of linear analysis because comparatively dissipation of energy within the soil mass is more in equivalent linear analysis.

The response spectrum presented in Fig 2.21 obtained from linear site response analysis shows that peak spectral value is about 1.54 g at period of 0.38 sec. Comparison of the response spectrum with the spectrum at the bed rock shows that the peak of the response spectrum at surface is significantly deviated from that at the bed rock. This clearly shows that the input excitation to the structures having those periods may be significantly influenced by the soil conditions. The response spectrum given in the Fig. 2.24 obtained from equivalent linear analysis shows that the peak spectral value is 1.28g at period 0.84 sec. The comparison of response spectrum with that of linear analysis shows the spectral shapes become boarder banded and the spectral values are drastically reduced near the peaks when equivalent linear analysis is taken into account.

The site response analysis results show that the average site period is about 2 sec. So, the site can be classified into soft soil site. Fig. 2.25 presents the comparison of Smooth response spectrum obtained by Newmark-Hall's method 41) at surface for both type of site response analysis with design response spectrum of NBC 42) for soft soil. The smooth response spectrum of linear site analysis shows that the maximum spectral accelerations for the period ranging from 0.13 sec to 0.62 sec are about 1.2 times greater than that are stipulated in code. And the smooth response spectrum of equivalent linear site analysis shows that the peak spectral accelerations for the period ranging from 0.15sec to 0.99 sec are about 1.1 times less than that are stipulated in code. This indicates that the structure of period between those ranges have to be faced little bit smaller seismic forces than stipulated in codes for the site response analysis. The comparison shows that the spectra are lower at longer periods greater than 0.7 seconds in the case of linear site response analysis which seems very conservative at longer periods.

Fig. 2.26 presents the comparison of Smooth response spectrum obtained by Newmark-Hall's method 41) at surface for both type of site response analysis with IS code design spectrum 43) for soft soil. The smooth response spectrum of linear site analysis shows that the maximum spectral accelerations for the period ranging from 0.13 sec to 0.62 sec is nearly equal with the value stipulated in the code. And the smooth response spectrum of equivalent linear site analysis shows that the peak spectral accelerations for the period ranging from 0.15sec to 0.99 sec are about 1.3 times less than that are stipulated in code. This indicates that the structure of period between those ranges have to be faced little bit smaller seismic forces than stipulated in codes for the equivalent site response analysis. This comparison shows the spectra for longer period are closer to each other for both site analyses. Fig. 2.19 presents the simulated acceleration time history at the surface for the return period of 475 years, and the peak ground acceleration (PGA) is about 0.4 g. Fig.2.19 also presents the simulated acceleration time history at the surface for the return period of 475 years, and it is developed through ground response analysis with the simulated ground motion at the bedrock. It is seen from Fig. 2.19 that the peak ground acceleration is about 0.425 g, which is slightly larger than the PGA in Fig. 2.9.

Some of the major conclusions of the study are:

1. The maximum PGA amplification is about 1.6 times greater than PGA at bed rock for linear ground response analysis and 1.18 for equivalent linear response analysis.
2. The soil conditions greatly influence frequency content of the earthquake motion. The broad banded spectrum is converted into narrow banded.
3. The linear effect of soil subjects more seismic energy to the structures having natural time period ranging from 0.4 sec to 2 sec and nonlinear effect of soil subjects more seismic energy for those structures having natural time period of 1 sec.
4. When the soil has a linear behavior, the input excitation to the structures having 0.38 seconds periods (with peak spectral value 1.54g) may be significantly influenced by the soil conditions. On the other hand, when the soil goes to the non linear behavior, soil may influence more excitation (with peak spectral value 1.28g) to the structures having period of 0.84 sec
5. The spectral shapes become broader banded and the spectral values are drastically reduced near the peaks when equivalent linear analysis is taken into account.

References

1) Molnar, P.: Structures and tectonics of Himalaya: constraints and implication of geophysical data, *Ann. Rev. Earth Planet Sci.* I2, pp. 489-518, 1984.
2) Avouac, J. P.: Mountain building, erosion, and the seismic cycle in the Nepal Himalaya, *Advances in Geophysics, Elsevier Inc*, 46, 2003.
3) BECA World International (New Zealand) in association with SILT Consultants (P.) Ltd. (Nepal), TAEC Consult (P.) Ltd. (Nepal), Golder Associates (Canada) and Urban Regional Research (USA): Seismic Hazard Mapping and Risk Assessment for Nepal, 1993.
4) Rana B. S. J. R.: Nepal's Great Earthquake 1934 (in Nepali, *Sahayogi Press*, Tripureshwor, Kathmandu, 1935.
5) Ambraseys, N., and Jackson, D.: A note on early earthquakes in Northern India and Southern Tibet. *Current Science ,* 84 (4), 2003.
6) Maskey, P.N., and Datta, T.K.: Risk consistent response spectrum and hazard curve for a typical location of Kathmandu valley, *13th world conference on earthquake engineering,* Vancouver, B.C., Canada, Aug.1-6 , 3124, 2004.
7) Kawashima, K., Aizawa, K. and Takahashi, K.: Acceleration of peak ground motion and absolute acceleration response spectra, *Proc., Eight World Conference on Earthquake engineering,* San Francisco , 2, pp. 257-264, 1984.
8) Japanese International Cooperation Agency (JICA): *The study on earthquake disaster mitigation in the Kathmandu valley, Kingdom of Nepal*, 2002.
9) Parajuli, H., Kiyono, J. and Scawthorn, C.: Probabilistic seismic hazard assessment of Kathmandu, *Proceedings of twentieth KKCNN symposium on civil engineering*, Jeju, Korea, Oct. 2007.
10) Parajuli, H. R.: *Seismic risk assessment of a reinforced concrete framed building,, Master's thesis.* Institute of Engineering, Department of Civil Engineering, Tribhuvan University, Pulchowk Campus, Nepal, 2002.
11) Parajuli, H., Kiyono, J., Ono, Y. and Tsutsumiuchi, T.: Design earthquake ground motions from probabilistic response: Case study of Nepal, *Journal of Japan Association for Earthquake Engineering*, Vol.8, No.4, November, 2008.

12) USGS. http://pubs.usgs.gov/gip/dynamic/understanding.html,http://earthquake.usgs.gov/regional/world/world_deaths.php.
13) Pant, M. R.: A step towards historical seismicity of Nepal. *Fransco-Nepal conference on people, environment and landscape of Himalayas*, 20 April, 2000.
14) Ambraseys, N.N., and Douglas, J.: Magnitude calibration of north Indian earthquakes, *Geophysics. J. Int. , 159*, pp. 165-206, 2004.
15) Hank, T.C., and Kanamori, H.: A moment magnitude scale, *Journal of Geophysics Res. , 84*, pp. 2348-2350, 1979.
16) Mcguire, R. K.: Seismic hazard and risk analysis, *Earthquake Engineering Research Institute, MNO-10*, 2004 .
17) Stepp, J.: Analysis of completeness of the earthquake sample in the Pudet Sound area and its effect on statistical estimates of earthquake hazard, *Proceedings of the first microzonation conference*, Seattle, WA, pp. 897-9091992.
18) Bilham, R., and Ambraseys, N.: Apparent Himlayan slip deficit from the summation of seismic moments for Himalayan earthquakes, *Current Science , 88* (10), 1658-1663, 2005.
19) Crouse, C. B.: Ground-motion attenuation equations for earthquakes on the Cascadia Subduction zone, *Earthquake Spectra,* 7, 201, 1991.
20) Fukushima, Y., and Tanaka, T.: A new attenuation relation for peak horizontal acceleration of strong earthquake ground motion in Japan, *Bulletin of Seismological Society of America , 80* (4), 757-783, 1990.
21) Molas, G.L., and Yamazaki, F.: Attenuation of earthquake ground motion in Japan including Deep focus events, *Bull. Seismol. Soc. Am. , 85*, pp. 1343-1358, 1995.
22) Youngs, R.R, Chiou, S.J, Silva, W.J and Humhrey, J.R.: Strong ground motion attenuation relationships for subduction zone earthquakes. *Seismological Research Letters , 68* (1), pp. 58-73, 1997.
23) Gregor, N. J., Silva, W. J., Wong, I. G., and Youngs, R. R.: Ground-motion attenuation relationships for Cascadia subduction zone, *Bulletin of Seismological Society of America , 92* (5), pp. 1923-1932, 2002.
24) Atkinson, G.M., and Boore, D.M.: Empirical ground-motion relations for subduction-zone earthquakes and their application to Cascadia and other regions, *Bull. Seismol. Soc. Am. , 93* (4), pp. 1703-1729, 2003.
25) Atkinson, G.M., and Boore, D.M.: Erratum to empirical ground-motion relations for subduction-zone earthquakes and their application to Cascadia and other regions. *Bull. Seismol. Soc. Am. , 98* (5), pp. 2567-2569, 2008.
26) Kanno, T., Narita, A., Morikawa, N., Fujiwara, H., and Fukushima, Y.: A new attenuation relation for strong ground motion in Japan based on recorded data, *Bulletin of Seismological Society of America , 96* (3), pp. 879-897, 2006.
27) Zhao, J. X., Zhang, J., Asano, A., Ohno, Y., Oouchi, Takahashi, T., Ogawa, H., Irikura, K., Thino, H. K., Somerville, P. G., Fukushima, Y., and Fukushima, Y.: Attenuation relations of strong ground motions in Japan using site classification based upon predominant period, *Bulletion of Seismological Society of America , 96* (3), pp. 898-913, 2006.
28) Woo, W. : Kernel estimation methods for seismic hazard area source modeling, *Bull. Seismol. Soc. Am. , 86* (2), pp.353-362, 1996.
29) Chen, Y., Liu, J., Chen, L., Chen, Q., and Chan, L.S.: Global seismic hazard assessment based on area source model and seismicity data, *Natural Hazards, Kluwer Academic Publishers , 17*, pp. 251-267, 1998.
30) Wells, D.L., and Copersmith, K.J.: New empirical relationships among magnitude, rupture length, rupture width, rupture area, and surface displacement, *Bull. Seismol. Soc. Am. , 84* (4), pp. 974-1002, 1994
31) Lave, J., Yule, D., Sapkota, S., Basnet, K., Madan, C., Attal, M., and Pandey, R.: Evidence for a great medieval earthquake (1100 A.D.) in the central Himalayas, Nepal, *Science* , 307, 2005.
32) Mcguire, R.K.: Probabilistic hazard analysis and design earthquakes: Closing the loop, *Bull. Seismol. Soc. Am*., 85, (5), pp.1275-1284, 1995.
33) Chapman, M.C.: A probabilistic approach for ground motion selection for engineering design, *Bull. Seismol. Soc. Am.* 85, pp. 937-942, 1995.
34) Bazzurro, P., Cornell, C.A.: Deaggregation of seismic hazard, *Bull. Seismol. Soc. Am.*, 89 (2), pp. 501-520, 1999.
35) Osaki , Y.: Introductory book for spectral analysis of earthquake motions", Kajima Institute of Publishing Co. Ltd, 1994.
36) Kempton, J. J. and Stewart, P. J.: Prediction equations for significant duration of earthquake ground motions considering site and near-source effects, *Earthquake spectra*, Vol. 22(4), pp. 985-1013, 2006.
37) Govinda Raju, L. Ramanna, G. V. Rao, C. Hanumantha, and Sitharam, T. G.: Site-specific ground response analysis, *Current Science*, Vol. 87, No. 10, 2004.
38) Piya, B.K: Generation of a Geological Database for the Liquefaction Hazard Analysis For Kathmandu Valley, *Thesis,* 2004.
39) Ohta, Y. and Goto, N.: Estimation of s-wave velocity in terms of characteristic indices of soil, *Butsuri-Tanko*, Vol. 29, No. 4, pp. 34-41, 1976.
40) Nippon Koei & Oye: The study on Earthquake Disaster Mitigation in the Kathmandu Valley, Kingdom of Nepal, *report*, 2001.
41) Newmark, N. M., and Hall, W. J.: Procedures and criteria for earthquake resistant design, *Building Practices for Disaster Mitigation*, Building Science Series 46, U. S. Department of Commerce, pp 209-236, and 1973.
42) NBC 105- 1994: Seismic Design of Buildings in Nepal, *Nepal National Building Code*, HMG of Nepal, Ministry of Physical Planning and Works, DUDBC, Kathmandu, Nepal, 2004.
43) IS 1893 (Part 1) 2002: General Provisions and Buildings (Fifth Revision), *Indian Standard Criteria for Earthquake Resistant Design of Structures*, Bureau of Indian Standards, New Delhi, India, 2002.

3. Structural Analysis of Traditional Brick Masonry Buildings

3-1. Present situation of Traditional Brick Masonry Buildings in Patan

Prem Nath Maskey

3-1.1 Structural Design and Construction

The study area of Jhatapol represents the prototype of settlements in Patan city which has become a heritage site in terms of traditional buildings and historical structures. The area and its neighborhood have their layout with a mixture of buildings of different periods, with very different levels of maintenance, from the 15th century onward. Recently a significant number of original buildings have been intervened with concrete framed buildings, usually with extension of floors making the buildings with 5 or more than 5 storeys. This has aggravated the seismic vulnerability situation with substantially greater height and small plan area.

The large number of private buildings in the core area of Patan city has traditional architecture and constructed with indigenous technology. Most of the buildings are three or four stories high with floor height between 1.8 to 2.4m. Generally these buildings have a simple rectangular plan with depth about 6 m and length varying from 3 to 10 meters. The foundation is usually shallow, made out of stones or brick. The superstructure is constructed with locally available burnt clay bricks and mud-mortar. The whole structure is supported by three walls, two outside walls and one spine wall at the centre. At the upper storey the spine wall is sometimes replaced by a timber frame system so as to create a larger continuous space. The floors and roof are supported by timber joists, over which wooden boards or planks with a thick layer of mud topping is applied. The roof is usually doubly pitched covered with traditional roofing tiles made of burnt clay.

The buildings are coalesced with each other forming a courtyard. At least one house in the courtyard provides access to the street through a gateway on the ground floor. This is generally due to lack of space or due to security reasons. A brief description of the most important structural components is described in the following paragraphs.

Foundations

The foundations of the traditional masonry houses, small temples, and monasteries are typically shallow and normally strip foundations constructed with brickwork at a depth of about 150 cm or more below grade level for a three story structure. These are sometimes often stepped, and the bottom most step has a width of about 75 cm and more. The base is generally formed by a few layers of large pebbles or broken stones. This is followed by the usual brickwork, which continues into the upper structure. No damp proofing was used in traditional masonry structures.

Masonry walls

The structural system of the traditional brick masonry buildings is based on the unreinforced brick masonry in mud mortar. Walls usually consist of three different vertical layers of masonry. The outer face is made of a wythe of burnt brick, locally known as *ma apa*. For the main front of important buildings or temples, a special wedge-shaped brick, *daci apa* is employed quite often to create a veneer-like façade. The taper of these bricks results in a varying thickness of the mortar joint from the front to the back. A hairline joint is created, which decreases the weathered area of the joints. These hairline joints are filled with *saldhup*, a mixture of natural black resin and mustard oil (Fig. 3-1.1).

The inner face of the wall is made of one wythe of burnt or even unburnt brickwork (*kaci apa*). Only rarely do those thick walls consist of continuous solid brickwork. More often they consist of a core of brick rubble infill set between an outer faces of brick (Fig.3-1.1). The bricks are traditionally laid in mud mortar. Commonly, all bricks of the outer face are laid as runners, so that there is no or only little bonding between the different leafs. Cross walls are often simply butt jointed with no interlocking at all. Figs. 3-1.2a and 3-1.2b presents the walls made of ordinary clay brick and walls made with the conical bricks, *daci apa*.

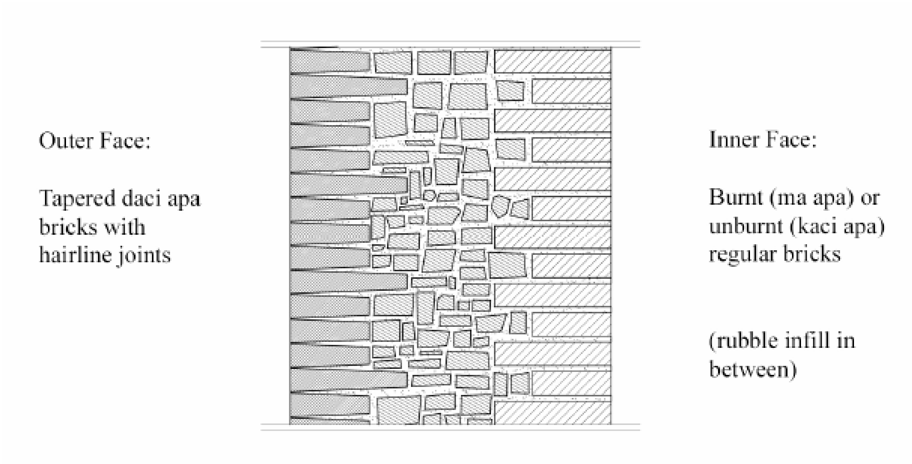


Fig. 3-1.1 Typical wall cross section

Fig. 3-1.2 (a) ordinary brickwork (b) original *dachi aapa* brickwork

Floor system

The floor of the traditional houses is constructed of timber floor. Such a floor is supported on timber joists running from party wall to party wall with variable dimensions, but usually of section 50 x 100 mm and closely spaced at a spacing of 200 to 300 mm. Floors are supported by closely spaced timber joists with a layer of terra cotta tiles or wooden planks. This subfloor is topped with a 10cm thick cover of fine clay. The joists rest on continuous wooden wall plates that are embedded in the masonry. Wooden pegs are provided at the ends of the timber joists adjacent to the supports, which are normally wall plates, to restrain the horizontal movement of the joists. The average height of the floors is about 2.20 m.

Roof system

Traditionally the buildings have the roof structure based on timber rafters, sometimes on timber trusses, with the local roofing tile (*jhingati*) and with a thick mud insulating layer present The most prominent feature of the traditional masonry buildings is the wide projecting roofs, which typically protrude more than one meter from the façade; in the case of larger buildings and temples this can be more than 2 meters. All roofs are pitched purlin roofs, with a slope of about 30 degrees. Ridge beams are typically supported by kingposts. The eave purlins are supported by often magnificently carved wooden brackets (*tunalas*). The brackets are set at an angle of approximately 45 degrees, and spaced at about 2m centers. These are supported either on a brick cornice or a wooden ledge, or, less often, on a slightly projecting beam. The rafters are connected above the ridge beam with half splice joints. A layer of horizontal wooden planks or flat terra cotta tiles is placed on top of the rafters. A thick layer of mud (5 – 20cm) is put on top of this substructure, in which the local roof tiles (*djingati*) are pressed.

Windows size and frames

The traditional masonry buildings are characterized by windows, in general, of smaller square size, and located symmetrically in the walls. The windows are built with a wooden double frame, and with wooden lintels extending in multi layers to the surrounding walls. The window opening shape, size and layout in the traditional buildings in later periods are different and are generally altered in style. The most prominent feature of the traditional buildings is the latticed window (*San Jhya*) peculiar to the Newar house, a richly decorated window that takes most of the façade at the third storey level, and with seating framed within it. The short opening ratios for the windows in the traditional houses allow a larger shear area near the openings, and stops decrement of the horizontal in plane stiffness of the walls, which are crucial in the seismic resilience of the unreinforced masonry buildings.

Timber framed structure of the ground floor (*dalan*)

The *dalan* is one of the prominent features of many traditional buildings in the locality. It is normally a ground floor space with a timber frame made of twin columns, surmounted by a capital on which sits a double beam (Fig. 3-1.3). The two adjacent timber frames are usually connected only at the level of the beam. This space is most commonly found at the ground floor of the main façade of buildings in which the front room is used as shop or workshop.

Fig. 3-1.3 *dalan* structure at ground floor of a courtyard

A similar timber construction is also common in upper storeys as an internal structure in place of the spine wall. The columns usually have square cross section of about 100 x 100 mm to 150 x 150 mm, pinned to the ground or floor, and at a spacing of 100 to 150 cm apart. The capital and the beam are also connected to the column by timber pins and the joists of the floor above sit directly on the beam, connected to this in some cases by timber pegs. As is seen, the first floor joists directly support the façade of the upper storeys. The *dalan* structure, somehow imparts a soft storey effect in seismic terms associating with the failure mechanism, as all connections are simply pinned; this situation may increase the seismic vulnerability factor of the traditional masonry buildings.

Earthquake resistant measures

The traditional masonry buildings of Patan have the following earthquake resistant measures:

1. **Configuration**: Most of the buildings are rectangular. These simple and Symmetric configurations in plan make the building more stable. This causes no excessive torsion because the centre of mass and center of rigidity coincides in the building.
2. **Length to breadth ratio**: In most of the buildings the length to breadth ratio is 2or less.
3. **Openings**: Openings are relatively small and symmetrically located. The small openings increase the length of the facade and substantially increase the stiffness of the building.
4. **Double framing of opening**: Buildings have two complete frame of timber around the openings to strengthen it against lateral force.
5. **Wall thickness**: Wall thickness at the ground floor is normally 45 cm or more. Thickness decreases with succeeding upper floors. As the horizontal thrust at the ground level develops highest at the time of earthquake in the building, the greater thickness reduces the shear failure at that time.
6. **Floor height**: In all cases the storey height is less than 2.5m
7. **Number of stories**: Most of the buildings are three stories.
8. **Wooden bands**: Wooden bands around the building at sill level, lintel level and at the floor level can be found curved as "Naga". These bands protect the walls from out of plane failures as well as provide integrity between different structural elements by connecting orthogonal walls. Also building act as monolithic, so that earthquake force is resisted by the building as complete unit rather than by individual part.
9. **Vertical post at corners**: These vertical posts at corners act as vertical tensile reinforcement. This protects the building from damage due to tensile cracks in the building. In some cases they provide some redundancy in the system which is very useful to withstand earthquake force.
10. **Struts**: Struts are long wooden planks which support the overhanging roof of the temple. It transfers the load of the roof to the vertical load bearing wall.
11. **Wooden corner stitch**: In addition to wooden bands, corner stitch can be found which connects orthogonal walls and protects from separation at corners.
12. **Wooden pegs**: Proper connection of all wooden elements by wooden pegs can be seen in traditional buildings, which helps for proper connection of roof and floor with wall as well as the different elements of roof or floor.
13. **Boxing of opening**: Boxing of opening by wooden frames, either all around or along both edges of the masonry wall provided strength around the opening.
14. **Reducing load consecutively in upper floors**: There are the reduction of the wall thickness in upper floors due to lesser load carrying requirement and the use of light partition walls, second and third floor central walls are often total timber frames. This reduces dead load of upper floor and gives more shear strength to the spine.

15. **Plinth**: Mostly on temples, foundation design for tall temples was changed into massive multiple plinths. These improve response against wave amplification and avoid resonance with ground.

Deficiencies increasing the vulnerability of the traditional masonry buildings

The deficiency elements for earthquake resistant measures in traditional masonry buildings are as listed below:

1. **Wall**: Wall structure was always built in mud mortar with three vertical layers. Outer and inner face layers were not well connected with the middle core wall. Normally the middle core was filled with rubbles and mud, which makes wall very poor to hold the heavy load from the main structure. The quality of the bricks was always good on the exterior surface, but normally for interior walls inferior bricks were used. In many places, fired bricks were used only for the exterior while for the interior and middle fill, simple sun-dried bricks and rubbles were used.
2. **Shape**: Most of the buildings are simple and symmetric in plan. However, they can be slender and thus vulnerable to damage.
3. **Pounding effect**: In urban most of the buildings are joined together to form blocks, Hence during severe earthquake, when all buildings shake according to their own natural period of vibration, pounding action between adjoining houses can take place.
4. **Material**: Sun dried bricks used for walls in buildings are brittle in nature and cannot take tensile stresses incurred during earthquake, thus resulting in large cracks or collapsing of walls. Also the quality of mortar is poor.
5. **Heavy roof and flooring**: In many buildings roof is held relatively tightly to wall by use of wedges and tie members. However, roof is heavy due to use of thick layer of mud and heavy tiles that can create problems as bigger the reactive masses of the building, the bigger is the earthquake forces.
6. **Workmanship and maintenance**: The construction workmanship is entirely depends on the skill and experience of the craftsmen. The climate with heavy monsoon rains places several demands on materials. In some cases, no measures are taken until the building finally falls down.

3-1.2 Method of Repair

The existing stock of traditional brick masonry buildings of Patan constructed in different periods of history is in various states of condition. The aging of the materials, lack of regular maintenance and repair, modification of the buildings as per the social requirements have substantially reduced their original strength and serviceability. It is well recognized that the brick masonry buildings have a limited capacity to withstand the earthquake action even when these are new. In order to prepare the traditional buildings to properly perfom their intended functions, and more over to withstand safely the seismic action in future earthquakes, repair, restoration and strengthening of these buildings become essential. The need to improve the ability of these existing buildings to withstand seismic forces arises generally from the evidence of damages of masonry buildings in past earthquakes.

The need as well as the degree of strengthening must be based on the structural assessment of the buildings in reference with the possible seismic hazard in the locality. The establishment of actual strength and capacity is encountered with considerable uncertainties related with material properties and with the amount of strength deterioration due to age. However, the decisions are generally based on gross conservative assumptions about actual strength.

The method of repair or restoration and strengthening largely depend upon the structural system, materials of construction, and technology to be adopted. Moreover, there may be a restriction on use of new materials from the Department of Archaeology, if the traditional buildings are labeled as historical structures, and are located in the monumental zones. In most of the cases, the materials to be used in the process of repairs, restoration and strengthening are confined to bricks, mud mortar, timber and stones.

(1) Concept of Repair

The main purpose of repairs is to reinstate the original character of the building so that the building will function as originally intended including all its services. However, repairs are not intended to improve the structural capacity, and hence may not meet the structural requirements to undertake the seismic action in future earthquakes. The repair works in the traditional brick masonry buildings of Patan include the actions like,

i. Patching up of defects such as cracks and fall of plaster;
ii. Repairing doors, windows, replacement of glass panes;
iii. Re-building non-structural walls etc;
iv. Re-plastering of walls as required;
v. Rearranging and replacement of disturbed/broken roofing tiles(jhingati);
vi. Relaying cracked flooring at ground level;
vii. Relaying mud topping of the timber floor;
viii. Replacement of planks/boards of the timber floor;
ix. Treatment or replacement of timber elements such as joists, rafters, wooden stairs and wall plates etc.;
x. Treatment for efflorescence and dampness;
xi. Repainting/ relaying preservative on wooden surfaces;
xii. Redecoration — whitewashing, painting, etc;
xiii. Repairing services such as electric wiring, gas pipes, water pipes and plumbing services.

(2) Concept of Restoration

The main purpose of restoration is to carry out repairs to structural elements of the buildings, and hence restoration is the reinstatement of the strength the building had before the damage occurred. This type of action is generally is undertaken when there is evidence that the structural damage can be attributed to exceptional phenomena that are not likely to happen again and that the original strength provides an adequate level of safety. The restoration process includes the actions like rebuilding portions of structural elements or addition of more material with an aim to bring back the original strength. The restoration process may involve inserting temporary supports, underpinning, etc.

The restoration works in the traditional brick masonry buildings of Patan may include:

i. Removal of portions of cracked masonry walls and piers and rebuilding them in richer mortar. Use of no shrinking mortar will be preferable.
ii. Addition of reinforcing mesh on both -faces of the cracked wall, holding it to the wall through spikes or bolts and then covering it suitably. Several alternatives have been used.
iii. Injecting epoxy like material, which is strong in tension, into the cracks in walls, columns, beams, etc.

(3) Strengthening Concept

The main purpose of strengthening of the traditional brick masonry buildings is to increase its capacity of withstand the seismic action in future earthquakes. The seismic performance of the existing traditional brick masonry buildings is affected by their original structural inadequacies, material degradation due to time, and alterations influencing the behavior of the structural system. Seismic strengthening of these buildings is essential, as their evaluation reveal that the original strength was not adequate to combat the seismic hazard expected in the area.

The traditional brick masonry buildings are located in the monumental zone and most of these carry heritage value, hence complete replacement of the buildings by new construction with earthquake resistant features adopting modern materials is not possible to retain the social harmony and cultural heritage. This situation calls for an essential need for improvement over the original seismic capacity of the buildings as a primary step towards disaster risk mitigation of the area.

The determination of the degree of strengthening depends upon the structural and seismic assessment of the buildings. The process should not be limited to improving the strength of structural elements in isolation, but should consider the overall structural behavior of the structural system of the buildings.

In reference with the existing traditional brick masonry buildings of Patan, the general principle of the seismic strengthening procedures is focused on:

i. Increasing the lateral strength and stiffness in one or both directions, by increasing wall areas or the number of walls and columns.
ii. Providing proper connections between the structural elements: walls, floors, roofs, foundations etc so that the whole structure acts as a unit. It should have a clear load path capable of transmitting the gravity as well as inertia force due to earthquake action.
iii. Removing the features, which are the sources of weakness or which produce stress concentrations in some structural members. Mass and stiffness irregularity in plan and in elevation and large openings in the walls are the intrinsic reason for such defects.
iv. Providing the traditional detailing of connection of timber joists with the wall plates or walls with timber pegs (shear lock); providing timber ties between the door and window frames at strategic levels, upright post at corners and also adjacent to openings to give the framing action.
v. Improving in-plane stiffness of the timber floors by providing extra layer of wooden planks or boards nailed perpendicular to the existing layer of planks or boards over joists.
vi. The methods of repair, restoration and seismic strengthening of unreinforced brick masonry buildings are highlighted by 1), whereas the traditional methods of them applicable to traditional brick masonry buildings of Patan are elaborated in section 2).

3-1.3 Present Situation of Buildings

The old city of Patan is a dense urban area within the Kathmandu Valley. It is predominantly inhabited by indigenous `Newar` people on housing clusters laid out around open courtyards. Similar to other Newar settlements, it is common to observe regular cultural function like ritual in the open space. The urban settlement in Patan is organized into various neighborhood units and is linked by a hierarchy of streets and open spaces, used for various rituals. The traditional neighborhood units, in terms of spatial extent, are defined prominently by a series of houses joined wall to wall along the street or by a built form enclosing an open area 3). As a typical characteristic of the traditional settlement, the sequence of open spaces are very much part of the life of the people for carrying out daily activities and rituals. The open space and the community structures like sattals and patis may serve as the excellent emergency escape and shelters respectively in the event of disasters such as earthquake, epidemy or fire. The inhabitants in Patan city, as in other old cities of the Kathmandu Valley, are exposed to the earthquake hazard.

It is evident from various characteristics of the existing traditional buildings that the inhabitants had a fail deal of knowledge and practice of planning the space, and constructing the buildings to combat with the earthquake hazards. Within the limits of the availability of the indigenous materials and technology, the builders practiced to achieve the most possible earthquake resistant buildings. The regular configuration with symmetrical plans, number of storeys limited to three, symmetrical location of openings with minimum sizes is planning features appropriate for the earthquake resistance characteristic of the structure. Several details of connections, such as the shear locks in the joists, vertical upright posts, provision of timber ties at strategic levels of walls, provision of double framing in the door and window openings are some of the examples of many design methods and technology adopted, which considerably help in overcoming the intrinsic weaknesses of the unreinforced brick masonry.

The old city of Patan has been able to hold its urban fabric including the traditional brick masonry buildings. However, there has been a recent trend of modernizing the buildings in the area basically due to change in socio-economic condition, and the new requirements according to the changing living styles.

The present situation of the buildings in the area can be described as the conglomeration of three broad categories: (i) traditional brick masonry buildings in original form, and (ii) traditional brick masonry buildings with modifications, and (iii) buildings with modern features like inclusion of concrete frames. The structural condition and seismic resilience of these types of buildings are different. The salient features of these categories are briefly described as follows:

(1) Traditional brick masonry buildings in original form

The structural configuration and the construction method have been retained as original with a little or no change (Fig. 3-1.4 and 3-1.5). These are the traditional buildings constructed during the Malla Period, or Shaha Period. However, many of these buildings at present are not in a good shape due to various blemishes, basically due to deterioration of materials in age and lack of repair and maintenance. Most of these buildings still exhibit the grandeur of the architecture and excellent configuration appropriate for the structural response against the possible seismic action, but these are in urgent need of restoration so that nor only the strength of the structural components are reinstated but also the building could perform with adequate strength in an integrated manner.

(2) Traditional brick masonry buildings with modifications

These are the traditional brick masonry houses modified considerably in terms of configuration and characterized by horizontal as well as vertical irregularity. Addition of extra stories with larger openings and replacement of traditional roofs by corrugated galvanized iron sheets are the main deviation from the original form (Fig. 3-1.5). Addition of balconies with a provision of concrete slab from the masonry wall without proper anchorage (Fig. 3-1.6) further increases the vulnerability of the building. Such type of modifications gravely affects the originally planned lateral load resisting nature in terms of stiffness and strength. In some cases, the buildings may not be adequate even for gravity loads. Evidently such buildings are not assessed for their structural adequacy and the existing foundations are not checked for the addition of more floors.

Fig. 3-1.4 a)Traditional brick masonrybuilding in original form, Jhatapol study area, Patan

Fig.3-1.4 b) Traditional brick masonry building in original form(a mixture of traditional and 19th century imported architecture) Dhaubaha, Patan

Fig. 3-1.5 Traditional brick masonry building with addition of extra floors, Jhapapol study area, Patan

Fig.3-1.6 Traditional brick masonry building with addition of extra floors and balconies without proper anchorage at Patan

(3) Buildings with modern features

These are the buildings in the locality which must have been constructed as a substitute for the repair or restoration of the traditional brick masonry buildings. The structural system in general of such buildings is reinforced concrete frames. However, most of such buildings do not comply with the code provisions related with concrete structures and the seismic resistant structures. Although the provision of reinforced concrete slabs substantially impart the in- plane stiffness of the floors in the buildings, and the concrete frames are better than masonry load bearing walls, the construction of such buildings without proper design and detailing may not yield the expected performance of the building. For the regular buildings with the provision of infill walls of adequate thickness, the provision of concrete frames with inadequate frame sections may act as the framing of the walls, and hence enhancing the brick masonry structural system; this way such buildings could be treated structurally as the wall systems framed by concrete frames. Deficiencies such as inadequate size of the concrete columns and the absence of load path (Fig. 3-1.7) substantially aggravate the performance of the structures. The popular trend of providing masonry walls over the projected beams to increase the room space substantially increases the seismic vulnerability of the buildings (Fig. 3-1.8). This type of buildings are usually with more floors like 5 or more, with increased floor height of about 2.4 to 3 m. Plan and elevation irregularity are common in these buildings, with provision of larger openings for windows, and with projecting overhangs with walls. The external walls are either brick exposed or covered with plaster and painted. The roof is constructed either with corrugated galvanized iron sheet or reinforced cement concrete, which is either flat with a terrace or gently sloped.

Fig. 3.7 Reinforced concrete frame building with unclear load path, Jhatapol study area.

Fig. 3.8 Reinforced concrete frame building with walls on overhang beams, Jhatapol study

References

1) International Association for Earthquake Engineering (IAEE): Repair, restoration and strengthening, *Guidelines for Earthquake Resistant Non-Engineered Construction*, pp. 1-23, 1986.
2) Ranjitkar, R.K.,: Advice for Maintenance of Historic Houses in the Kathmandu Valley, Kathmandu Valley World Heritage Site, Nepal, *Heritage Homeowner's Preservation Manual*, UNESCO Kathmandu 2006
3) Bhandari, R. B. and Okada, N.: Formulating Time Tested Knowledge for Sustainable Disaster Risk Reduction; A Case Study of Kathmandu, *Annuals of Disas. Prev. Res. Inst.*, Kyoto Univ., No. 52 B, 2009.

3-2. Structural Analysis of Traditional Brick Masonry Buildings (Case study)

3-2.1 Structural Survey of Traditional and Historical Buildings

Hari Ram Parajuli

The selected target area for the study is the Jhatapol, typical urban areas in Patan of the Kathmandu Valley. The Jhatapol area includes a part of the boundaries of the World Heritage Monument zone and its buffer zone. The research area with respect to the World Heritage Monument Zone is presented in Fig. 3-2.1.

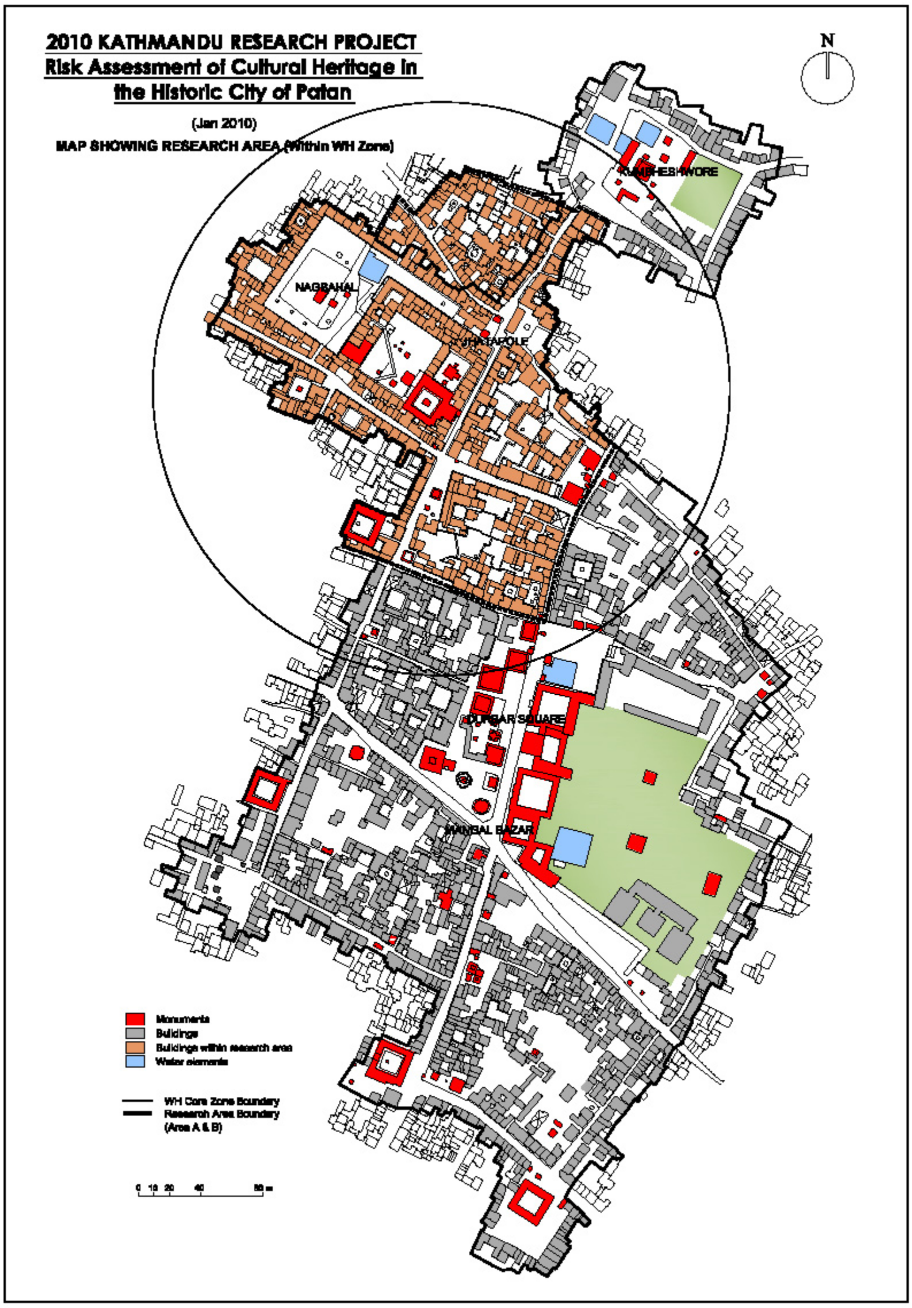


Fig. 3-2.1 The World Heritage Monument Zone and the research area

For the purpose of the building survey for the seismic vulnerability assessment, the study area of Jhatapol is divided into three clusters, namely Block A, B and C (Fig. 3-2.2). There are altogether 218 numbers of buildings within the three blocks. Before undertaking the detailed structural survey of the buildings, the buildings are indexed in the base map using a specific nomenclature, such as A1, A2, .., B1, B2,… and C1, C2,… , the first letter indicating the cluster block and the numeric value indicating the building number (Fig. 3-2.2 and 3-2.3).

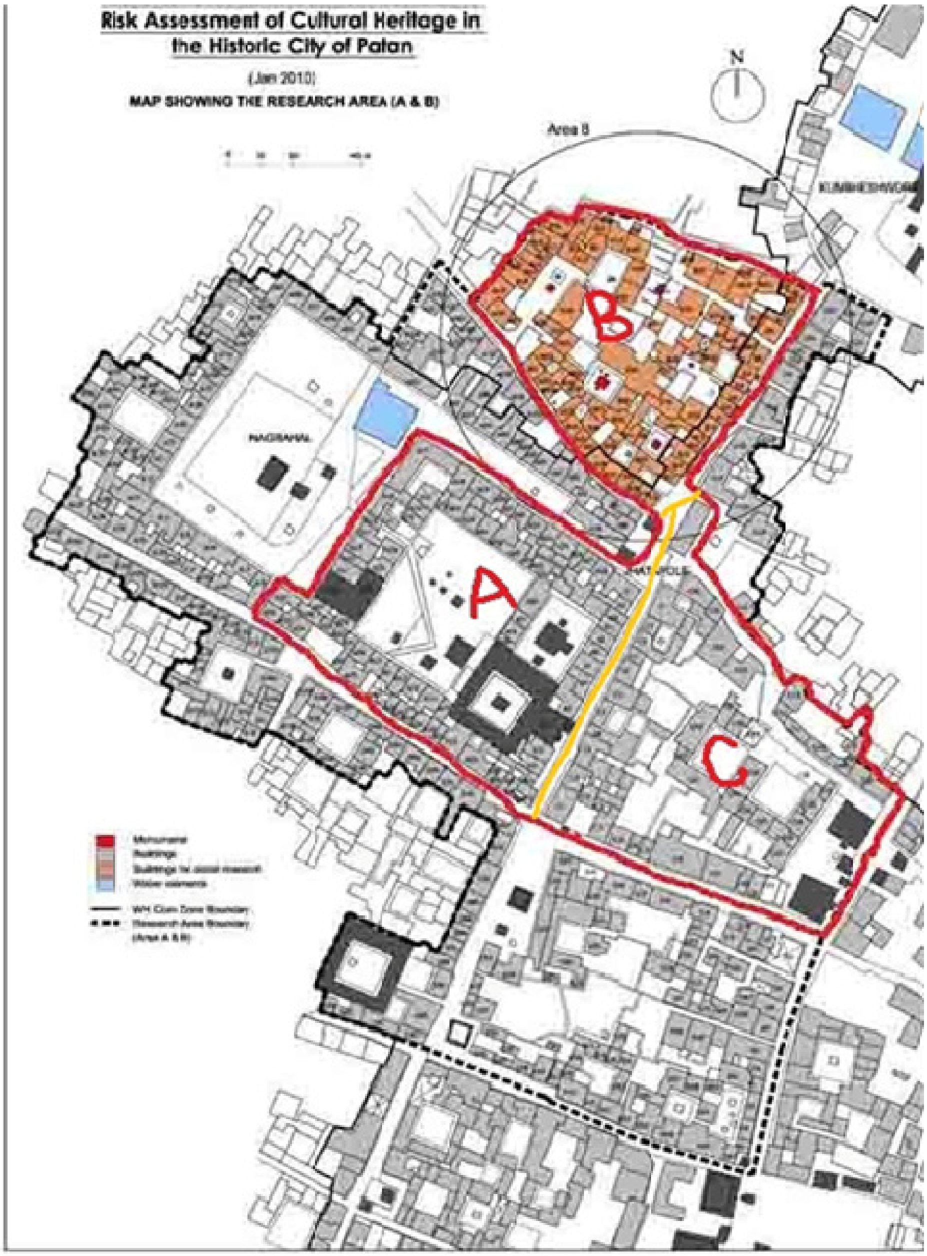


Fig. 3-2.2 Jhatapol study area with building stocks in 3 subzones: Block A, B and C

The main purpose of the structural survey of the buildings is to record the structural condition and the construction history of the buildings. In this connection, for each of the buildings the information collected are on:

- (i) Building No. in the base map
- (ii) Uses
- (iii) Building type

(iv) Construction pattern
(v) Number of storeys
(vi) Modifications
(vii) Inclination of building
(viii) Damages of foundation
(ix) Structural condition of walls
(x) Material deterioration in walls

All the above information are recorded in the building survey forms, a sample of which is given in Appedix A. The results of the visual screening of the buildings are presented in Table 3-2.1.

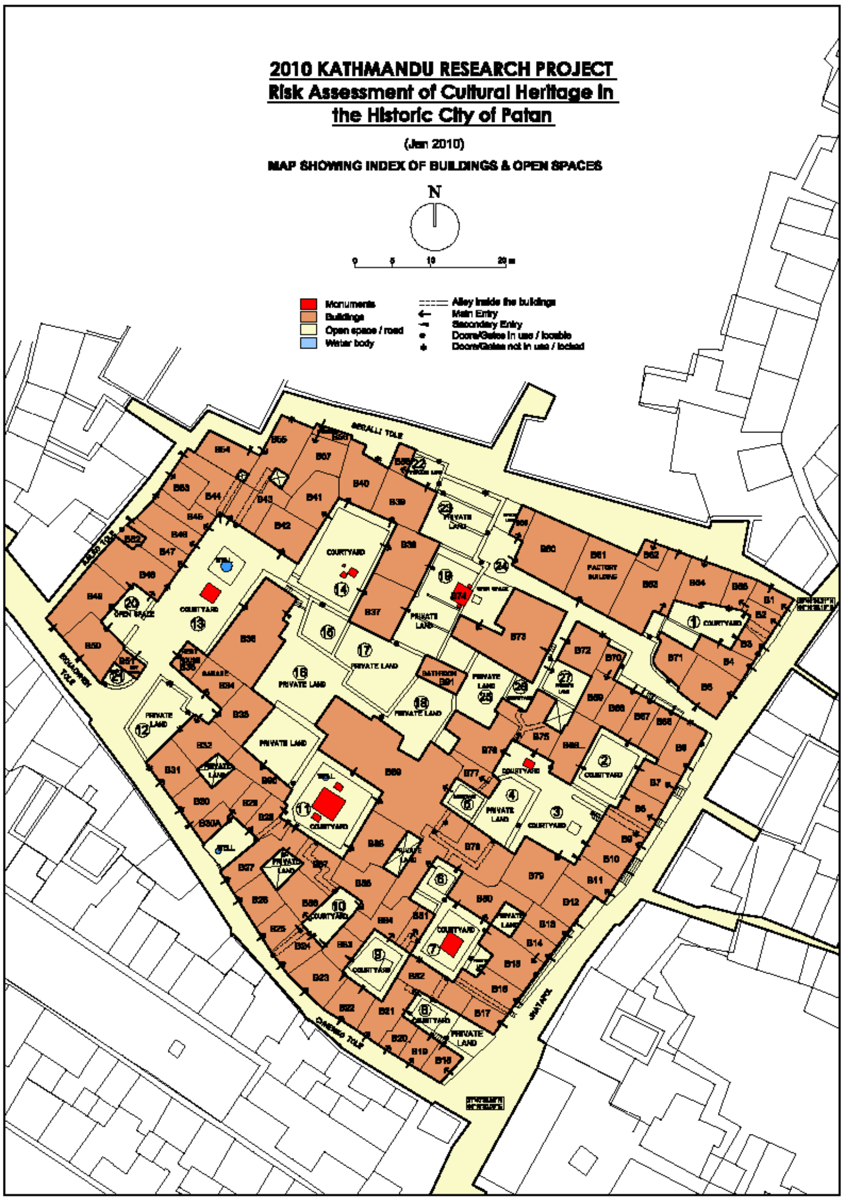


Fig. 3-2.3 Jhatapol study area with building indexes

Analyzing the structural survey of the buildings the following points were observed.

1. From the utility aspect, the buildings are categorized into two groups-residential and commercial. About half of the total buildings surveyed have been using for dual purpose,

residential as well as commercial. The owners in general, rent out the ground floor of the building for shops or other purposes.

2. The buildings are constructed either in brick masonry or in reinforced concrete. About three quarter of the total buildings is made in brick masonry.
3. The number of storeys of the surveyed buildings varies from one to six storeys. Majority of buildings are of 3-5 storeys with large window openings.
4. All buildings are categorized into damaged and undamaged. A building is categorized as damaged if it has cracks or inclinations at any locations such as near the openings (windows, doors), walls and foundations. Out of 218 buildings, 100 buildings, which is about 45% of the total buildings surveyed, are found in the class of damaged already because of sustained vertical loads, environmental deterioration and aging. It clearly shows that there will be severe damages in the buildings if even a moderate magnitude earthquake hits the city.
5. A building is categorized modified if any kind of repair, maintenance works has been made on the building. It also includes the addition of floors. A maximum number of modification and damages are observed in the 4 storeyed and the 5th storeyed buildings. This is basically due to the floor additions which have increased the vulnerability even in vertical loads.

Table 3-2.1 Construction type and structural condition of the existing buildings

No. of storeys	No. of builds.	Building type			Damage	Uses			Modifications
		RCC	Brick Masonry	Others		Res.	Res. & Comm.	Others	
1	13	2	10	1	6	0	1	12	0
2	2	0	2	0	0	0	0	2	0
3	35	5	30	0	18	16	14	5	3
4	90	18	72	0	43	46	41	3	35
5	71	18	52	1	32	35	32	4	39
6	7	3	4	0	1	3	4	0	4
Total	218	46	170	2	100	100	92	26	81

The classification of the surveyed buildings by number of storeys, type of the construction and utility is presented in Figs. 3-2.4 to -3-2.6. The status of the buildings in terms of modification and damage state are shown in Figs. 3-2.7 and 3-2.8.

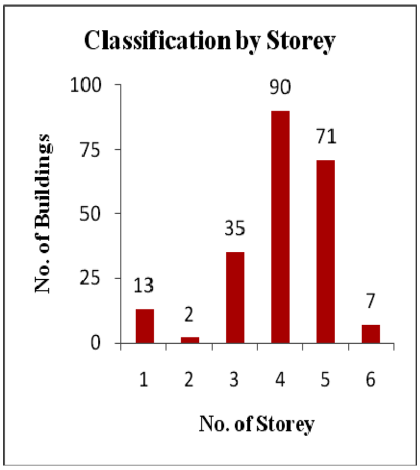


Fig. 3-2.4 Buildings by no. of storeys

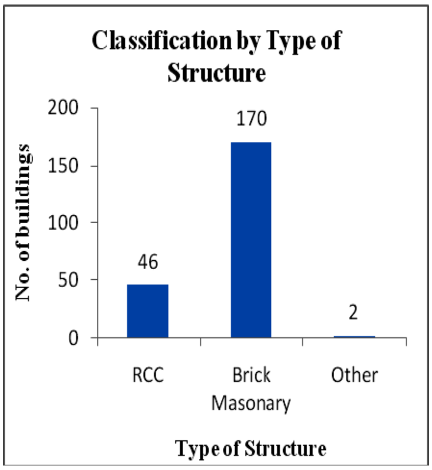


Fig. 3-2.5 Buildings by construction type

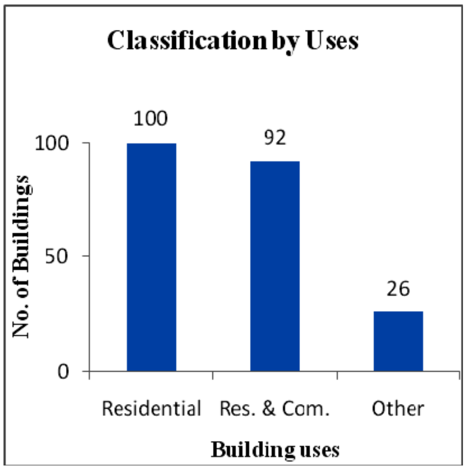


Fig. 3-2.6 Buildings by usage

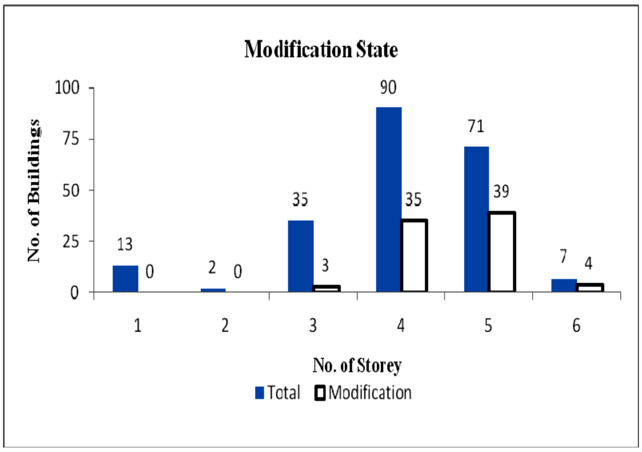


Fig. 3-2.7 Buildings by modification state

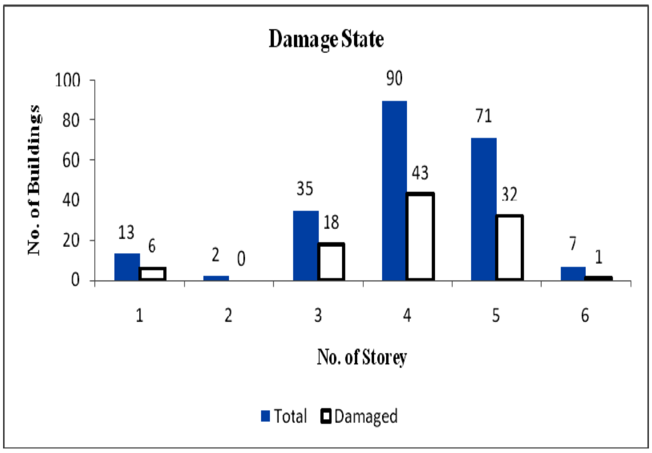


Fig. 3-2.8 Buildings by damage state

Visual inspection of the buildings from structural survey show that three quarter of the buildings are made of traditional bricks have sustained stresses and cracks due ageing. Forty five percentages buildings are already sustained some kinds of damages even in vertical and environmental attacks. Then question arises what would happen if an earthquake hit the buildings.

3-2.2 Experimental verification for Element of Reinforced Technology

Prem Nath Maskey

In order to investigate the seismic capacity of the buildings with reinforced concrete frames in the study area of Patan it is necessary to establish the material properties of the existing buildings. Mainly the compressive strength of the hardened concrete has to be established for the structures. Since any destructive testing is not possible to conduct, the testing of concrete by cutting the core from the existing structure is ruled out, and the test is confined to non-destructive tests only. The determination of the compressive strength of the concrete by Schmidt (Rebound) Hammer test and by Ultra-sonic Pulse Velocity are described.

(1) Non-Destructive Tests (NDT)

The Non Destructive Testing (NDT) of concrete has a great technical and useful importance especially in the case of construction quality assessment. The main advantage of non-destructive testing method is to avoid the concrete damage or the performance of building structural components. Additionally, their usage is simple and quick. Test results can be made available at the site and the problem of core drilling is overcome. The Schmidt rebound hammer (SRH) and the ultrasonic pulse velocity (UPV) tests, are two simple and useful non- destructive tests, which are useful when a correlation can be developed between hammer/ultrasonic pulse velocity readings and the strength of the same concrete. This non-destructive measurement method has proved to be of real importance in all constructions serving the purpose of testing and as an effective tool for inspection of concrete quality in concrete structures

(2) Schmidt Rebound Hammer Test

Schmidt Rebound Concrete Hammer test is carried out to assess the concrete and strength characteristics. It is a non-destructive test in which concrete can be tested in situ condition. This instrument enables engineers to control concrete quality and to detect weak spot. As the test can be done in the site condition, the strength of concrete can be evaluated in different time interval. The main advantage of this test is that it is easy to handle and it saves time. A typical Schmidt rebound Hammer is shown in Fig. 3-2.9.

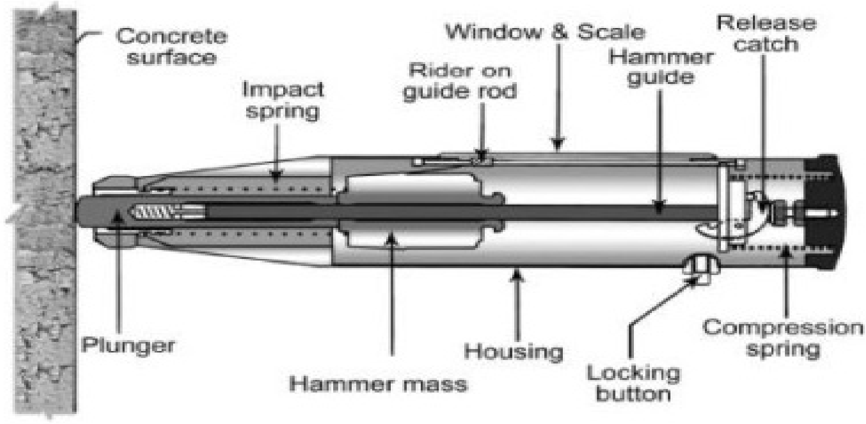


Fig. 3-2.9 Typical rebound hammer

The Schmidt rebound hammer is principally a surface hardness tester. It works on the principle that the rebound of an elastic mass depends on the hardness of the surface against which the mass impinges. There is little apparent theoretical relationship between the strength of concrete and the rebound number of the hammer. However, within limits, empirical correlations have been established between strength properties and the rebound number. The method of using the hammer is described in reference with Fig. 3-2.10. With the hammer pushed hard against the concrete, the body is allowed to move away from the concrete until the latch connects the hammer mass to the plunger. The plunger is then held perpendicular to the concrete surface and the body pushed towards the concrete. This movement extends the spring holding the mass to the body. When the maximum extension of the spring is reached, the latch releases and the mass is pulled towards the surface by the spring. The mass hits the shoulder of the plunger rod and rebounds because the rod is pushed hard against the concrete. During rebound the slide indicator travels with the hammer mass and stops at the maximum distance the mass reaches after rebounding. A button on the side of the body is pushed to lock the plunger into the retracted position and the rebound number is read from a scale on the body.

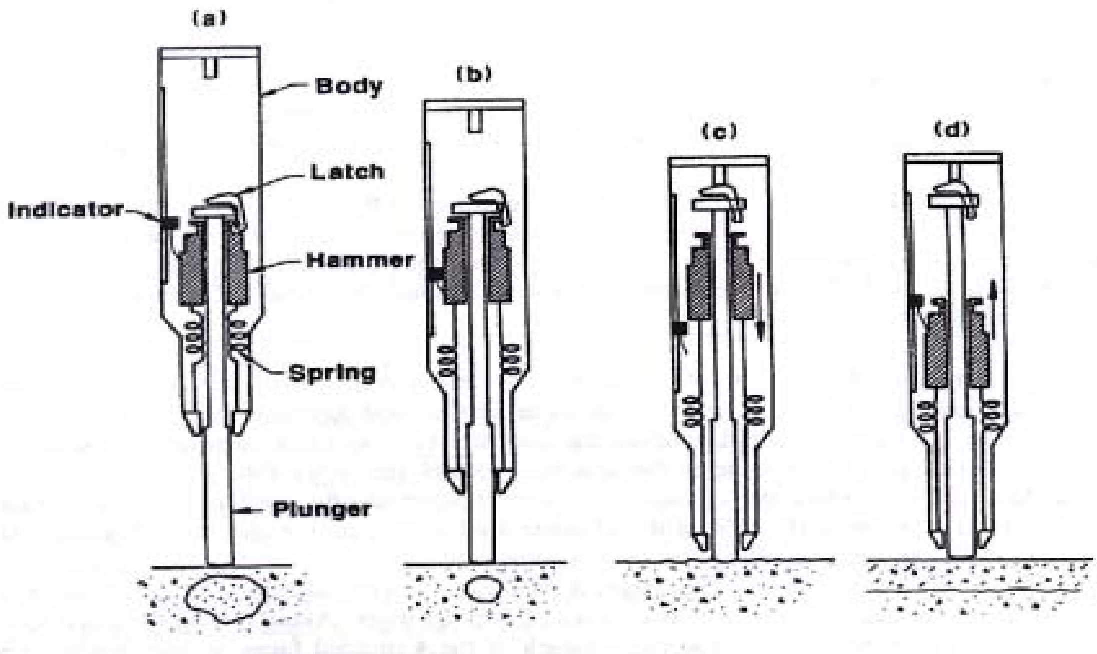


Fig. 3-2.10 working of Schmidt Rebound Hammer Test

The hammer can be used in the horizontal, vertically overhead or vertically downward positions as well as at any intermediate angle, provided the hammer is perpendicular to the surface under test. The position of the mass relative to the vertical, however, affects the rebound number due to the action of gravity on the mass in the hammer. Thus the rebound number of a floor would be expected to be smaller than that of a soffit and inclined and vertical surfaces would yield intermediate results. Although a high rebound number represents concrete with a higher compressive strength than concrete with a low rebound number.

To carry out the test firstly the site for the site is selected. The ceiling, columns and beams of a framed structured building are selected for the test. For testing of concrete slab, a grid of 1m x 1m is marked where the tests have to be performed. The points for testing on beam are selected at a distance of 1m. Three numbers of columns are selected with 3 points for tests on each of them. The plaster or coating on the concrete surface is removed, and the cement slurry present in top layer also is removed.

The rebound values obtained for each testing point in the grid are tabulated and the mean value is calculated for each row of the points and tabulated. These mean values are corrected using correction factors for inclination of 900 downward. The corresponding average values of the cube strengths for every row point are found with the help of correlation chart developed from the test conducted on the cubes. In order to develop correlation chart, standard cube specimen are prepared and cured as per standard specification. The Schmidt rebound hammer test is carried out on each cube and the compressive strength of these cubes is determined by testing on compressive testing machine. Then a chart is developed using R – value and the corresponding compressive strength. The correlation between Rebound number and uniaxial compressive strength is presented in Table 3-2.2.The correlation chart developed is shown in Fig. 3-2.11. The Rebound Value Correction for inclination (90° downward) is given in Table 3-2.3. The test results with rebound number and compressive strength for slab, beam and columns respectively are presented in Table 3-2.4 –3-2.6

Table 3-2.2 Correlation between Rebound Number and Uniaxial Compressive Strength:

S.N	Specimen type	Weight (Kg)	Rebound No.					Avg. Rebound No.	Failure load (KN)	Compressive Strength(Mpa)
			1	2	3	4	5			
1	Cube	2.460	26	30	24	24	20	24.8	210	21.0
	(100 x 100 mm)	2.440	26	24	18	36	28	26.4	235	23.5
		2.500	26	28	26	22	28	26.0	160	16.0
		2.646	42	40	50	52	36	44.0	288	28.8
		2.566	28	36	34	28	34	32.0	282	28.2
		2.575	32	28	38	42	38	35.6	268	26.8
		2.563	30	30	32	32	40	32.8	258	25.8
		2.568	21	21	20	22	22	21.2	208	20.8
		2.582	23	23	39	49	50	36.8	260	26.0
2	Cylinder	3.710	20	24	20	30	22	23.2	80	10.2
	(10 mm dia)	3.700	20	20	26	24	20	22.0	90	11.5
		3.740	18	24	28	26	30	25.2	100	12.7
		3.760	26	28	28	26	28	27.2	132	16.8
		3.779	20	26	34	36	38	30.8	125	15.9
		3.660	22	22	26	26	27	24.6	121	15.4
		3.710	22	21	20	23	22	21.6	141	18.0
		3.788	28	25	32	30	30	29.0	140	17.8
		3.737	24	25	43	38	52	36.4	154	19.6
3	Prism	4.670	28	30	26	20	22	25.2	84	14.9
	(75 x 75 mm)	4.270	31	24	33	20	20	25.6	74	13.2
		4.342	25	25	26	30	30	27.2	95	16.9
		4.370	39	34	32	27	26	31.6	97	17.2
		4.403	32	33	34	42	28	33.8	151	26.8
		4.606	32	38	32	36	32	34.0	141	25.1

Table 3-3.3 Rebound Value Correction for inclination (90° downward)

Rebound Value	Correction in rebound value
10	+3.2
20	+3.4
30	+3.1
40	+2.7
50	+2.2
60	+1.7

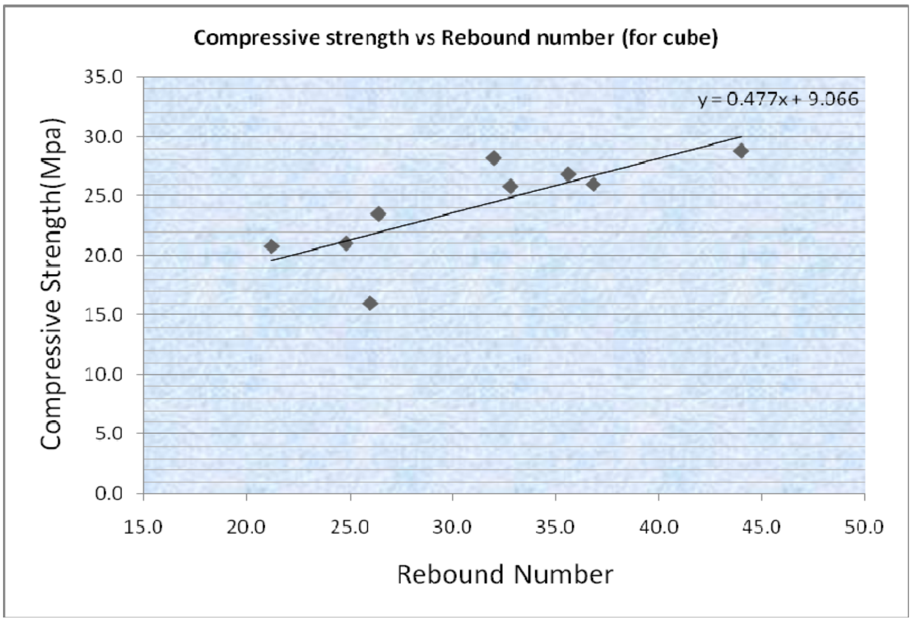


Fig. 3-2.11 Correlation chart between rebound number and uniaxial compressive strength

Table 3-2.4: Rebound number and compressive strength of slab

S.N.	Position	Rebound Number					Avg. Rebound	Correction	Corrected Value	Compressive Strength (Mpa)
		1	2	3	4	5				
1	Slab-1	21	20	25	24	28	23.60	3.508	27.108	22.00
2	Slab-2	17	16	20	18	19	18.00	3.360	21.360	19.25
3	Slab-3	29	27	30	29	33	29.60	3.688	33.288	24.94
4	Slab-4	18	22	24	18	26	21.60	3.448	25.048	21.01
5	Slab-5	28	26	32	30	26	28.40	3.652	32.052	24.35
6	Slab-6	20	21	18	24	24	21.40	3.442	24.842	20.92
7	Slab-7	24	29	24	25	26	25.60	3.568	29.168	22.98
8	Slab-8	24	26	26	24	28	25.60	3.568	29.168	22.98
9	Slab-9	18	22	18	24	24	21.20	3.436	24.636	20.82
10	Slab-10	22	19	22	24	22	21.80	3.454	25.254	21.11
11	Slab-11	29	24	22	22	20	23.40	3.502	26.902	21.90
12	Slab-12	20	20	20	24	24	21.60	3.448	25.048	21.01
13	Slab-13	22	18	24	26	22	22.40	3.472	25.872	21.41
14	Slab-14	20	23	23	24	19	21.80	3.454	25.254	21.11
15	Slab-15	29	22	18	24	22	23.00	3.490	26.490	21.70
16	Slab-16	22	25	22	20	22	22.20	3.466	25.666	21.31
17	Slab-17	19	20	24	24	22	21.80	3.454	25.254	21.11
18	Slab-18	27	24	22	22	20	23.00	3.490	26.490	21.70
19	Slab-19	19	20	22	29	24	22.80	3.484	26.284	21.60
20	Slab-20	20	22	24	28	28	24.40	3.532	27.932	22.39
									Avg.=	21.78
									S.D.=	1.28

Table 3-2.5: Rebound number and compressive strength of beam

S.N.	Beam Position	Rebound Number					Avg. Rebound	Compressive Strength (Mpa)
		1	2	3	4	5		
1	position-1	20	28	28	20	30	25.20	21.09
2	position-2	22	22	28	29	30	26.20	21.56
3	position-3	16	22	26	28	28	24.00	20.51
4	position-4	24	28	31	34	36	30.60	23.66
5	position-5	24	30	32	32	32	30.00	23.38
6	position-6	20	20	20	22	22	20.80	18.99
7	position-7	24	20	20	32	32	25.60	21.28
8	position-8	22	16	18	20	18	18.80	18.03
							Avg.=	21.06

Table 3-2.6: Rebound number and compressive strength of column

S.N.	Position	Rebound Number					Avg. Rebound	Compressive Strength (Mpa)
		1	2	3	4	5		
1	Column-1							
	Bottom	39	40	40	44	42	41.00	28.62
	Middle	40	42	46	43	38	41.80	29.00
	Top	44	40	44	42	44	42.80	29.48
2	Column-2							
	Bottom	40	34	40	40	42	39.20	27.76
	Middle	30	33	41	34	38	35.20	25.86
	Top	44	42	40	48	42	43.20	29.67
3	Column-3							
	Bottom	42	41	39	38	42	40.40	28.34
	Middle	41	39	43	40	40	40.60	28.43
	Top	42	46	38	44	45	43.00	29.58
							Avg.=	28.53

Strength at 95% confidence level = 21.78 – 1.64 x 1.28 = 19.68

Strength at 90% confidence level = 21.78 – 1.28 x 1.28 = 20.14

Strength at 95% confidence level = 21.78 – 0.84 x 1.28 = 20.71

(3) Ultrasonic Pulse Velocity Test

This type of non-destructive test used to determine the longitudinal ultrasonic pulse velocity in concrete structure through which the wave is propagated. There is no unique relationship between the wave velocity and the strength of the concrete; however, under specified conditions these two quantities are interrelated. The common factor is the density of concrete; a change in the density results in a change in the pulse velocity. If the velocity of a pulse of longitudinal waves through a medium can be determined, and if the density and Poisson's ratio of the medium is known, then the dynamic modulus of elasticity can be computed.

The equipment for the test consists essentially of an electrical pulse generator, a pair of transducers, an amplifier and an electronic timing device for measuring the time interval between the initiation of a pulse generated at the transmitting transducer and its arrival at the receiving transducer (Fig. 3-2.12). Two forms of electronic timing apparatus and display are available, one of which uses a cathode ray tube on which the received pulse is displayed in relation to a suitable time scale, the other uses an interval timer with a direct reading digital display.

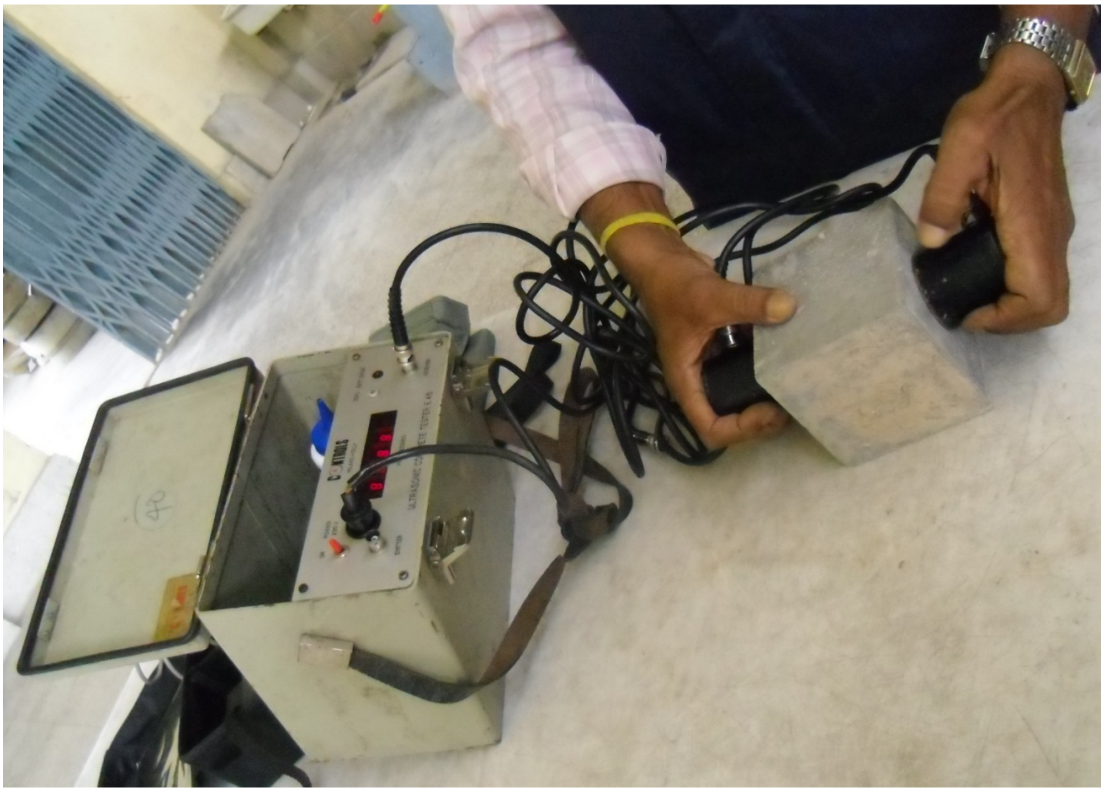


Fig. 3-2.12 Ultrasonic pulse velocity test equipment

A pulse of longitudinal vibrations is produced by an electro-acoustical transducer, which is held in contact with one surface of the concrete under test. When the pulse generated is transmitted into the concrete from the transducer using a liquid coupling material such as grease or cellulose paste, it undergoes multiple reflections at the boundaries of the different material phases within the concrete. A complex system of stress waves develops, which include longitudinal, shear and surface waves, and propagates through the concrete. The first waves to reach the receiving transducer are the longitudinal waves, which are converted into an electrical signal by a second transducer. Electronic timing circuits enable the transit time T of the pulse to be measured.
Longitudinal pulse velocity (in km/s or m/s) is given by:

$$V = \frac{L}{T} \tag{3-2.1}$$

where
V is the longitudinal pulse velocity,
L is the path length,
T is the time taken by the pulse to traverse that length.
If the density (ρ) and Poisson's ratio (υ) are known, then the modulus of elasticity of concrete (E) can be calculated using the equations 3-2.2 (for one dimensional body), 3-2.3 (for two dimensional body) and 3-2.3 (for three dimensional body).

$$V_P = \sqrt{\frac{E}{\rho}} \tag{3-2.2}$$

$$V_P = \sqrt{\frac{E(1-\nu)}{\rho}} \tag{3-2.3}$$

$$V_P = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)}} \tag{3-2.4}$$

Where, V_p = P – wave velocity

After obtaining the value of E from above equations, the characteristic strength (f_{ck}) and modulus of rupture (f_{cr}) can be calculated by using the IS456-2000 (equations 3-2.5-3-2.6).

$$E_c = 5000\sqrt{f_{ck}} \tag{3-2.5}$$

$$f_{cr} = 0.7\sqrt{f_{ck}} \tag{3-2.6}$$

where, E_c = modulus of elasticity of concrete

f_{ck} = characteristic strength of concrete

f_{cr} = modulus of rupture of concrete

Although the ultrasonic pulse velocity test can be applied for many purposes, here the use is basically aimed at determination of correlation of pulse velocity and strength as a measure of concrete quality, and also for determination of the modulus of elasticity and dynamic Poisson's ratio of the concrete.

The transducer for the test may be arranged in different ways. It is possible to make measurements of pulse velocity by placing the two transducers on either: opposite faces (direct transmission), adjacent faces (semi-direct transmission) (Fig. 3-2.13) or the same face (indirect or surface transmission) (Fig. 3-2.14).

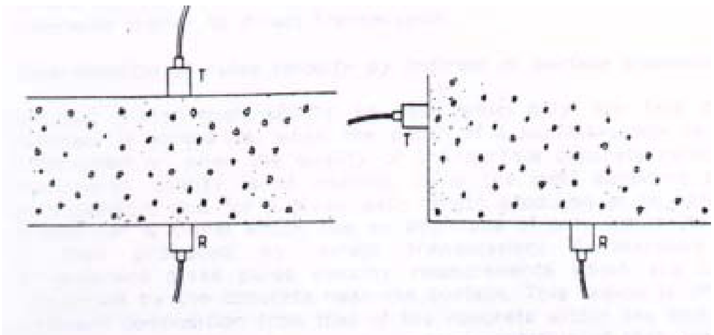


Fig. 3-2.13 Direct transmission and semi-direct transmission

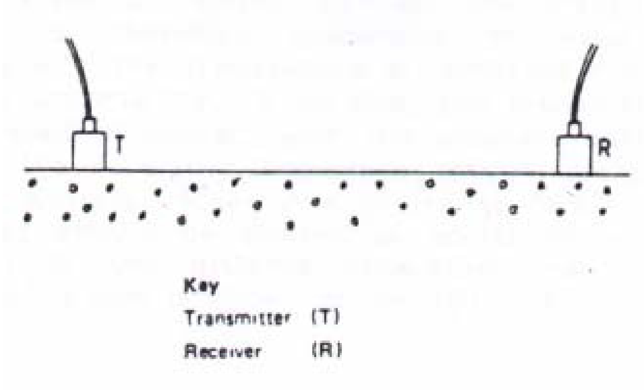


Fig. 3-2.14 Indirect or surface transmission

For the ultrasonic pulse velocity test, 6 points in each of the members (beam and column) of the framed building are selected. These points are selected on different faces of the members, and at the bottom and top of column height, and at the left and right support of the beam. Table 3-2.7 and 3-2.8 presents the test results of the column and the beam respectively.

Table 3-2.7 Ultrasonic pulse velocity and compressive strength of column no. C4

	Location points					
	1	2	3	4	5	6
Velocity of ultrasonic pulse	3.623	3.508	3.581	3. 882	3.421	3.711
Correction factor	1.0	1.0	1.0	1.0	1.0	1.0
Final value of velocity	3.62	3.51	3.58	3.88	3.42	3.71
η	1.0	1.0	1.0	1.0	1.0	1.0
Equivalent value of compressive strength of concrete	21.87	20.16	21.25	25.91	18.76	23.27
Mean value of compressive strength (N/mm^2)	**21.87**					

Table 3-2.8 Ultrasonic pulse velocity and compressive strength of beam along grid B

	Location points					
	1	2	3	4	5	6
Velocity of ultrasonic pulse	3.372	3.404	3.318	3.188	3.212	3.256
Correction factor	1.0	1.0	1.0	1.0	1.0	1.0
Final value of velocity	3.37	3.40	3.32	3.19	3.21	3.26
η	1.0	1.0	1.0	1.0	1.0	1.0
Equivalent value of compressive strength of concrete	17.99	18.45	17.21	15.19	15.50	16.28
Mean value of compressive strength (N/mm^2)	**16.77**					

3-2.3 Experimental investigation and Analyses of Traditional Brick Masonry Buildings

Hari Ram Parajuli

In connection with the research on the disaster risk management of the cultural heritage site of the historic city, the historical site of Jhatapol was selected as the focal area. The public structure of Lalitapura Pati, the land mark of the area was chosen as a case study for determination of its seismic vulnerability for the given seismic hazard, and, in its turn, to identify the building's seismic risk. The Lalitapura Pati (Fig 3-2.15.) is a two-storeyed masonry building, constructed about 300 years ago and represents the traditional construction. The building has a length of 16.5 m and a width of 5.6 m. The structural system of the building is based on unreinforced brick masonry walls constructed in mud mortar, with timber floor and sloped timber roof. The walls are made of traditional brick work with thickness 60 cm at the ground floor and 50 cm at the upper floor. In the ground floor, on the front side towards the main road, it has a series of timber colonnades instead of walls, and hence giving the traditional *dalan* appearance. Over these colonnades on the first floor the building has an elaborate latticed window with traditional wood carvings. The building has been repaired many times following the local damages mostly due to leakages and material deterioration. Recently, its sloped roof originally with local roofing tiles (*jhingati*) has been replaced by corrugated galvanized iron sheet, and the internal sides of the walls have been plastered with cement sand mortar. The current condition of the building appears as different from its original construction by look, possibly due to changes in the usage in time of the building. In order to carry out the seismic vulnerability of the building, the material properties of the walls are required to be assessed. The non destructive tests carried out to obtain the properties are described hereunder. As is obvious, any kind of destructive techniques in heritage structures are not allowed. The investigation is carried out by elastic wave measurement.


Fig. 3-2.15 Lalitpura pati, the model building; south-front (left) and west-side view (right)

(1) Material properties of historical masonry building

a) Elastic wave tomography

Elastic waves produced by a sudden redistribution of stress in a material due to external forces such as pressure, load, temperature etc., releases energy in the form of stress waves and propagates through the surfaces and recorded by sensors. The back side wall of the house shown in Fig. 3-2.15 was selected for the elastic wave experiment with tomography. An area of 1.5 m X 1.5 m was taken and 16 sensors were attached with wax at equal distances (Fig. 3-2.16). In case that the measured point was found at joints, the sensors were moved to the nearest brick surface. Then three set of measurements - two on each surface, and the other on cross sections were taken. Sensor arrangements at surfaces and at cross sections are shown in Fig. 3-2.16. Then impact on wall was given by a steel hammer having a small spherical ball at its edge near one sensor and the stress waves obtained at all sensors were recorded. Similarly, impact was given near to each sensor turn by turn and measurements at other sensors were recorded.

Fig. 3-2.16 Arrangement of sensors on two sides of the wall

Based on the first arrival time of P waves at various sensors, stress wave velocities in divided cells were evaluated aided by tomography algorithm as given in Figs. 3-2.17 – 3-2.18. The left figure is the distribution of P-wave, namely tomogram at outer surface (brick exposed surface Fig 2) and the right one is for inner surface tomogram when the wall was hit by a 8 mm diameter hammer. The figures show that P wave velocity varies from 500 to 1000 meter per sec. In the Fig.3-2.18, tomography has been shown in three dimension and cross section when impact was given to the wall by 25 mm diameter hammer. It shows that interior of the wall is stronger than towards the surface.

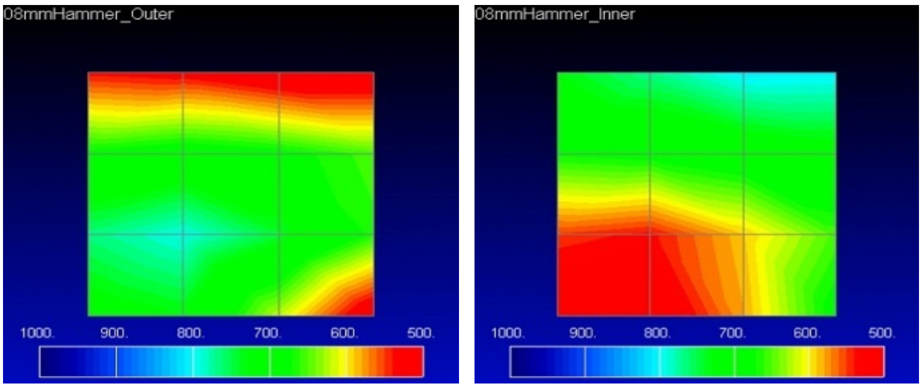


Fig. 3-2.17 Elastic wave tomography in 2D

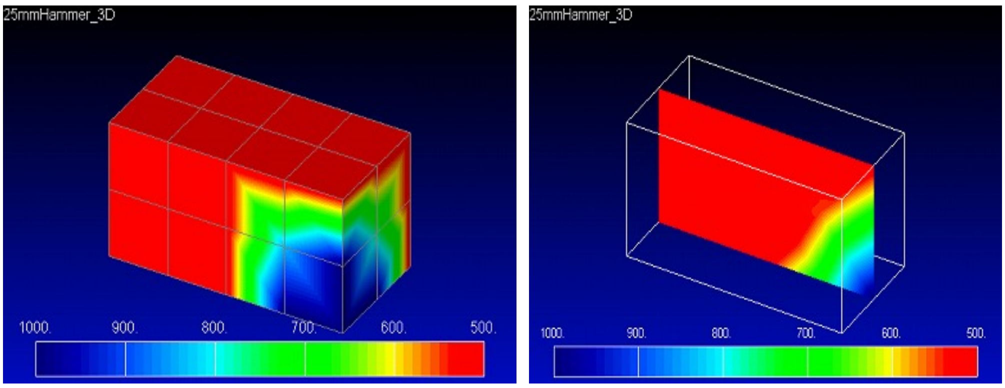


Fig. 3-2.18 Elastic wave tomography in 3D

b) Measurement by Pocket AE

The Pocket AE is a handheld instrument for acoustic emission testing and performs advanced wave-form based signal acquisition and processing. The elastic wave velocities estimated by the tomography give a wide range of values. The Figs. 3-2.17-3-2.18 show that these range from 500 to 1000 m/sec. For the finite element method (FEM) analysis, the specific values are required rather than the range. Thus, using pocket AE, a series of measurements were taken at various locations of different walls (Fig. 3-2.19). Noting the first arrival time of elastic waves, time differences of successive peak amplitudes at the near and the far end sensor channels were calculated. Then, velocity was calculated by thickness through which wave passes, divided by the time difference between two sensors. Then from the primary wave velocity, with the measured unit weight ($19KN/m^3$), and the assumed Poison's ratio (0.2), the modulus of elasticity (E) and the shear wave velocity (V_s) of the material are calculated, and shown in Table 3-2.9 using Kramer 1) relations.

Fig. 3-2.19 Instrumentation of two channel Pocket AE

Table 3-2.9 Obtained parameters

Types of walls	V_p (m/sec.)	G (N/mm^2)	V_s (m/sec.)	E (N/mm^2)
Single brick	2499	4537	1531	10889
11 cm wall	2576	4820	1677	11567
23 cm wall	1312	1250	803	3000
44 cm wall	670	326	410	782

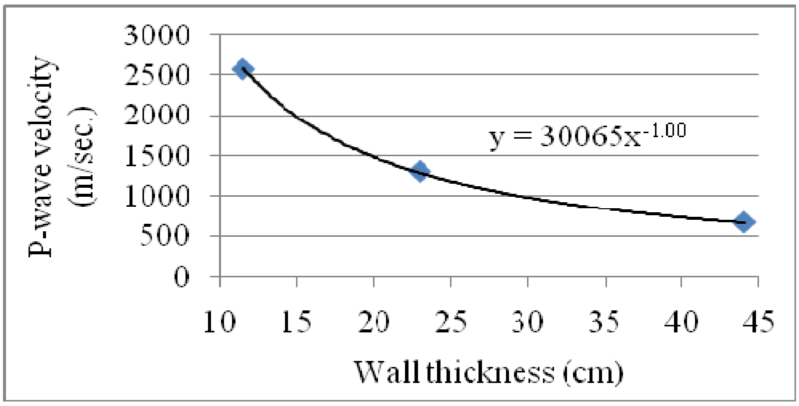


Fig. 3-2.20 Plotting of P-wave velocity with wall thickness

The experiment results show that as the wall thickness increases its P wave velocity decreases. The joints and voids inside the wall sharply decrease its strength, as a result P wave velocity is found decreasing. If the P wave velocity from the equation shown in Fig. 3-2.20 is projected for 55cm wall, it becomes 547m/sec. which lies in the ranges shown in Fig. 3-2.17-3-2.18 and seems reasonable. Though, interpolation and extrapolations would not be the case always, rather vary wall to wall depending upon its own properties. However, the trend of curve in the Fig. 3-2.20 shows, elasticity decreases with the increase of joints and voids in thicker wall.

c) Micro tremor measurement

The second experiment was ambient vibration measurements of the building and ground. Ambient vibrations of the historical model building at various locations and at three other locations in nearby areas were measured by three seismometers. The instruments measuring microtremors on ground and the buildings are shown in Figs. 3-2.21 and 3-2.22 respectively. The predominant frequency of ground was estimated from H/V spectrum. Various natural frequencies of the building were estimated from the Fourier spectrum, and damping of the masonry was estimated from the random decrement and transfer function between ground floor, first and second floors 2). In order to determine the damping from free vibration, the building was pushed off (Fig. 3-2.23). The push off was followed by recording of the free vibration history, and the damping value was calculated. The predominant frequency of the ground is found to be 2.1Hz, whereas the fundamental frequencies of the building are found as 4.3Hz, 4.7Hz, 6.8Hz and 7.7Hz along transverse and longitudinal directions respectively. The damping of the building is 6.4%.

Fig. 3-2.21 Measurements on ground

Fig. 3-2.22 Measurements on building

Fig. 3-2.23 Push off on building

The numerical models used for dynamic analysis of structures are idealizations of real structures to represent the response of real structures. The validity of the models can be verified by ambient vibration experiments or forced vibration experiments. Both methods have their own merits and demerits, however, for the heritage structures where any kind of destructive experiments are not allowed, non destructive experiments such as micro-tremor ambient vibration measurements is the suitable. Since the amplitudes are very small, only linear response can be investigated.

Seismic resistant capacity evaluation requires materials properties such as modulus of elasticity, Poisson's ratio, strength of walls and elements such as bricks, mortar timber etc., in tension, compression and shear as well as dynamic properties such as fundamental natural frequencies, mode shapes and damping. Civil engineering structures such as buildings, bridges, electric towers etc., are always vibrating due to ambient vibrations produced by wind, ocean waves, and atmospheric conditions. These ambient vibrations are measured by seismometers and their responses are studied in frequency domain. Since, the amplitudes of ambient vibration are very small, they describe only linear behavior of the structures. Ambient vibration signal get amplified at the natural frequency of ground and structures. Taking advantages of those characteristics of response natural frequencies are obtained. Thus as a part of efforts, micro tremor measurement on a historical house located at Patan City has been carried out and the dynamic properties such as natural periods, mode shapes and damping ratios are obtained and discussed.

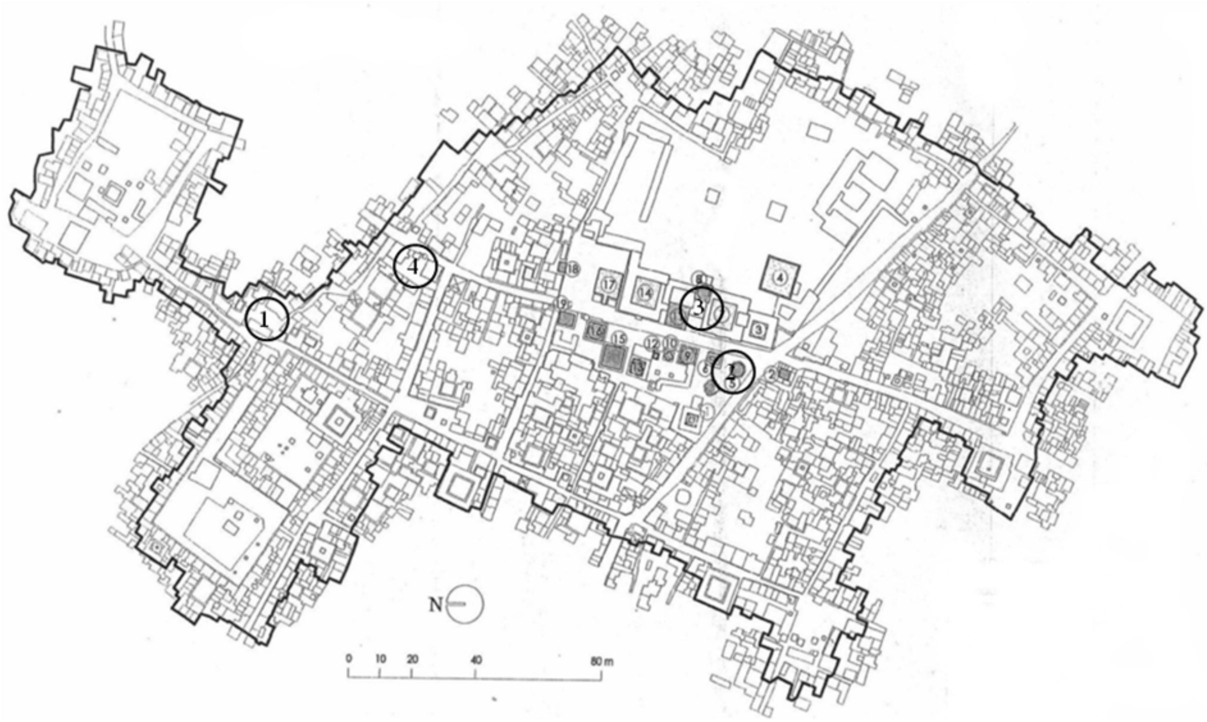


Fig. 3-2.24 Location of seismometers in World heritage site Patan Durbar Square area

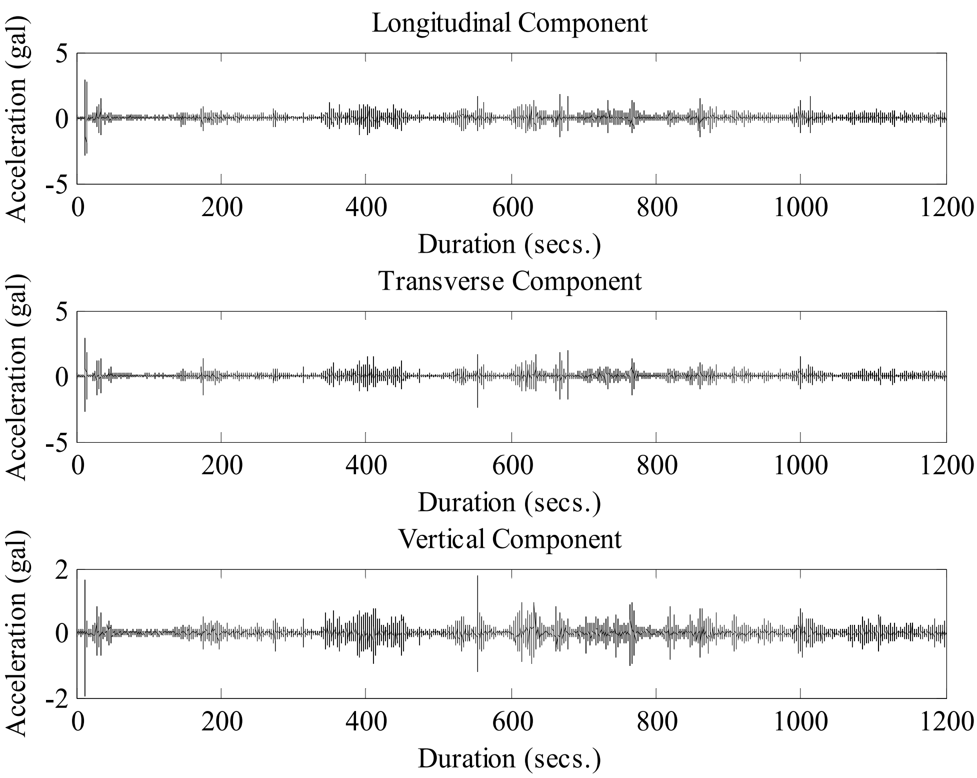


Fig. 3-2.25 Sample micro-tremor record at location 1

d) H/V Spectrum

Micro tremor measurement has long been recognized for in- situ evaluation and micro-zonation, which are widely used by engineers and planners. The most commonly used way to interpret the records is horizontal to vertical ratio namely H/V spectrum. This technique was originally introduced by Nakamura 3) to analyze Rayleigh waves in micro-tremor records. Now, it has been the method to estimate the site characteristics such as predominant frequency of site, amplification and vulnerability assessment. However, in this study, only predominant frequency of the ground is evaluated. For this purpose, seismometers were placed at four locations of the city as shown in Fig. 3-2.24, and three components of micro-tremors were recorded for 12 minutes. Sample micro-tremor records of site 1 are shown in Fig. 3-2.25.

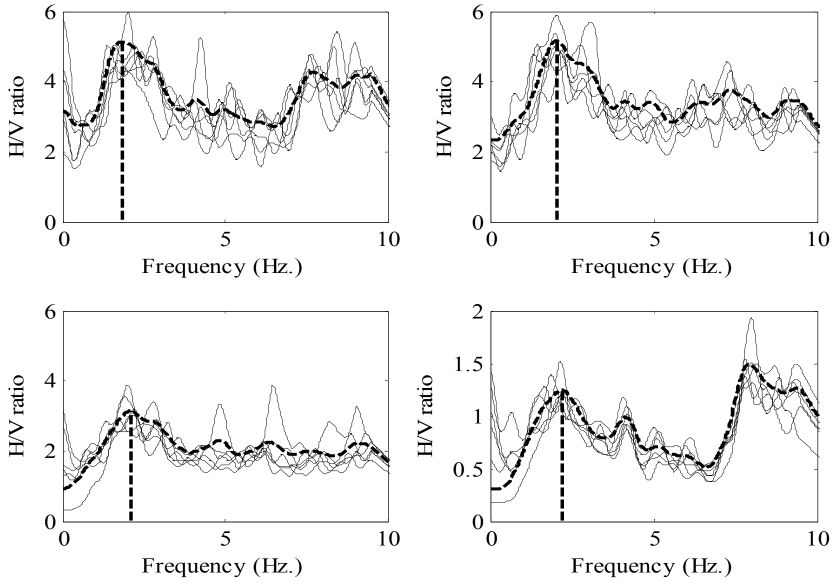


Fig. 3-2.26 Sample micro-tremor record at location 1

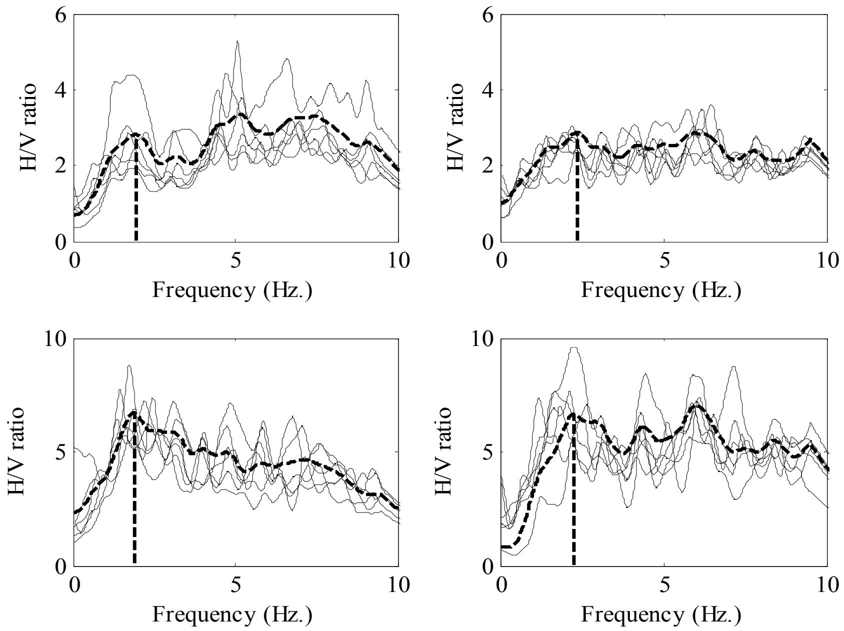


Fig. 3-2.27 Sample micro-tremor record at location 2

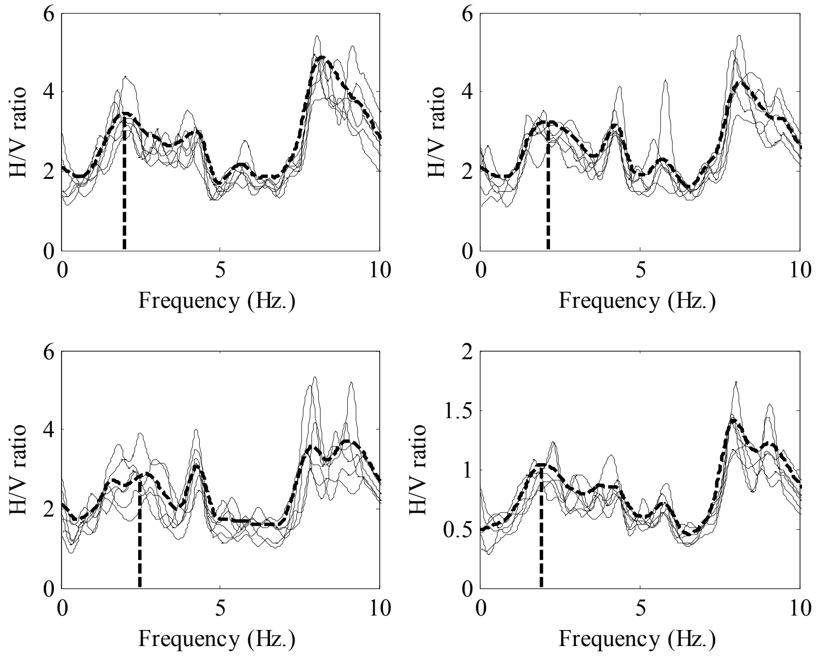


Fig. 3-2.28 Sample micro-tremor record at location 3

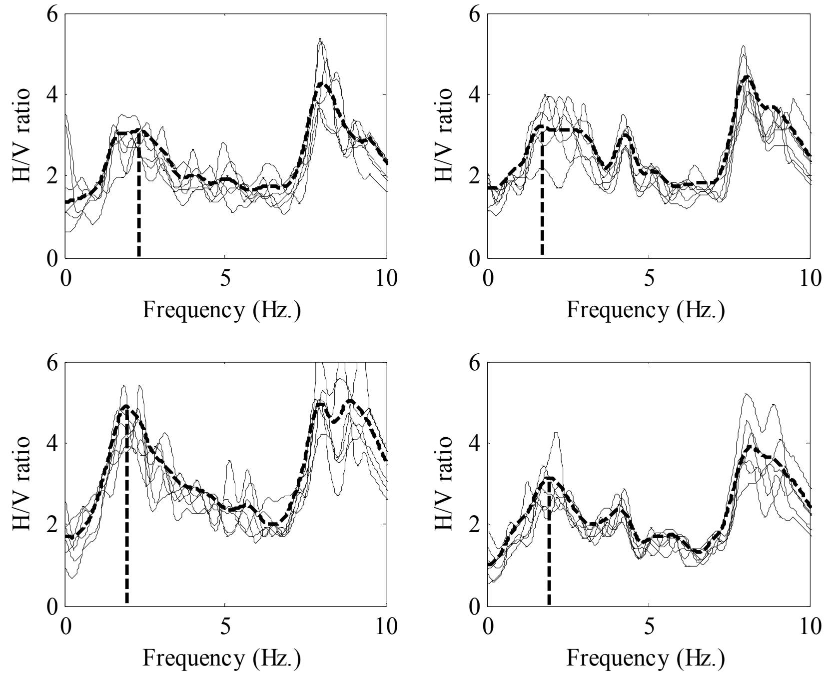


Fig. 3-2.29 Sample micro-tremor record at location 4

Considering the records are stationary, the whole record is cut into various segments having number of data in each segment equivalent to 2N which is essential to do Fast Fourier Transform (FFT). Then Fourier spectrum of each segment was plotted which are shown in Figs. 3-2.26 to 3-2.29 and obtained natural frequencies at are summarized in Table 3-2.10. From the calculation predominant frequency of the ground is found 2.07Hz.

Table 3-2.10 Frequencies from H/V spectrum

S.N	Location	Figs.	Frequency (Hz.)
1	site 1	3-2.26(a)	2.00
2	site 1	3-2.26 (b)	2.15
3	site 1	3-2.26 (c)	2.49
4	site 1	3-2.27 (d)	1.95
5	site 2	3-2.27 (a)	2.39
6	site 2	3-2.27 (b)	1.71
7	site 2	3-2.27 (c)	1.93
8	site 2	3-2.27 (d)	1.95
9	site 3	3-2.28 (a)	2.20
10	site 3	3-2.28 (b)	1.83
11	site 3	3-2.28 (c)	2.03
12	site 3	3-2.28 (d)	2.10
13	site 4	3-2.29 (a)	1.95
14	site 4	3-2.29 (b)	2.37
15	site 4	3-2.29 (c)	1.88
16	site 4	3-2.29 (d)	2.27
Average			**2.07**

e) Natural frequency of building

The building has been partitioned into two parts along longitudinal direction. The front side has big opening. Series of micro tremor ambient vibration measurements were taken at various locations of the building in the ground, first and second floors respectively. The measurements were recorded at seven different locations of the building as shown in the plan of the building in Fig. 3-2.30.

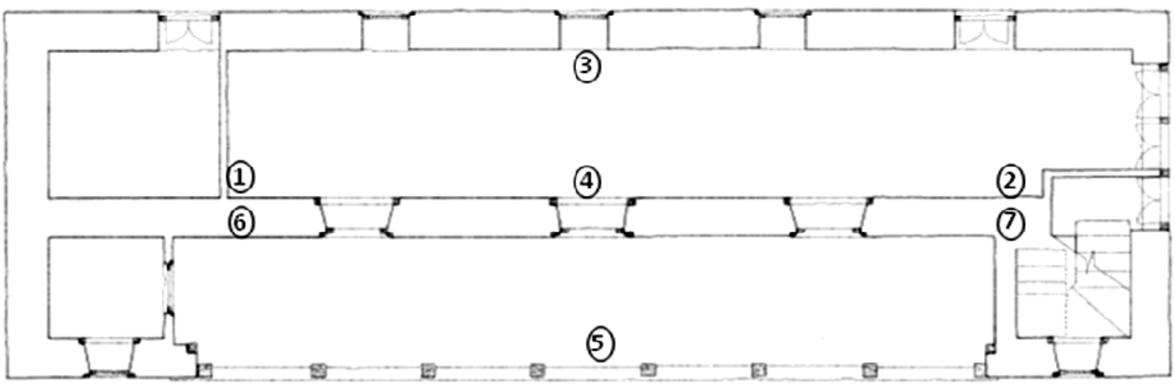


Fig. 3-2.30 Plan of the building and location of micro-tremor measurements

As in the H/V ratio calculation, twelve minutes long records are cut into six segments making 4096 data in each segment to make compatible for FFT excluding the possible noise caused by external factors such as pedestrians and vehicles etc. Fourier spectra of all the records obtained along both longitudinal and transverse directions at various locations of the building were calculated and plotted through Figs. 3-2.31 to 3-2.38 and summarized in Table 3-2.11 for. From the calculation, most dominant natural frequencies are 4.30Hz and 6.52Hz along transverse direction and 6.80Hz and 9.39Hz along longitudinal direction.

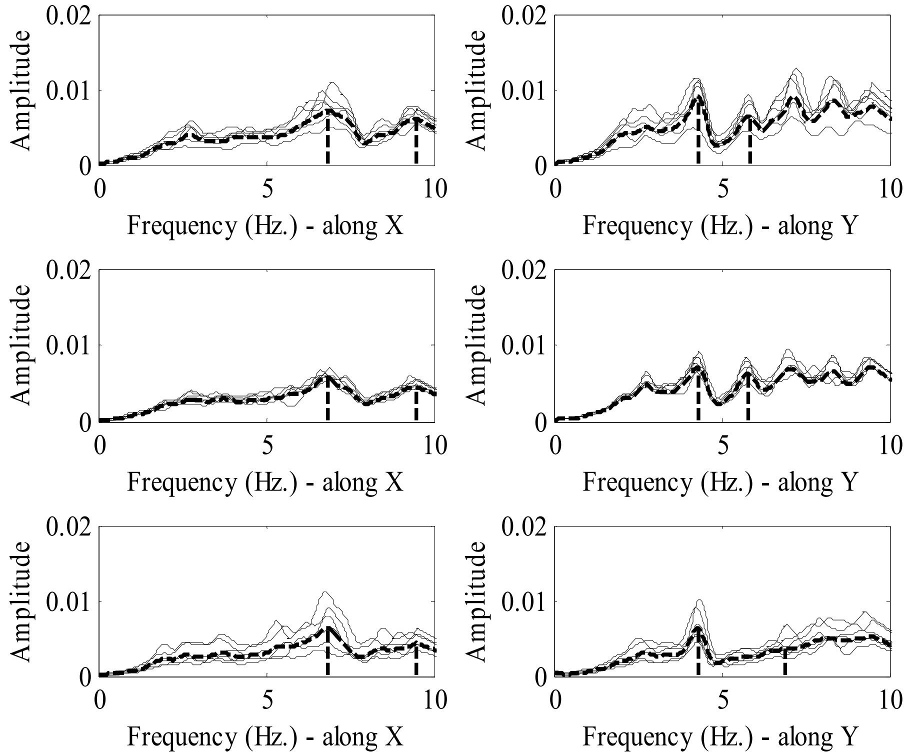


Fig. 3-2.31 Fourier spectrums

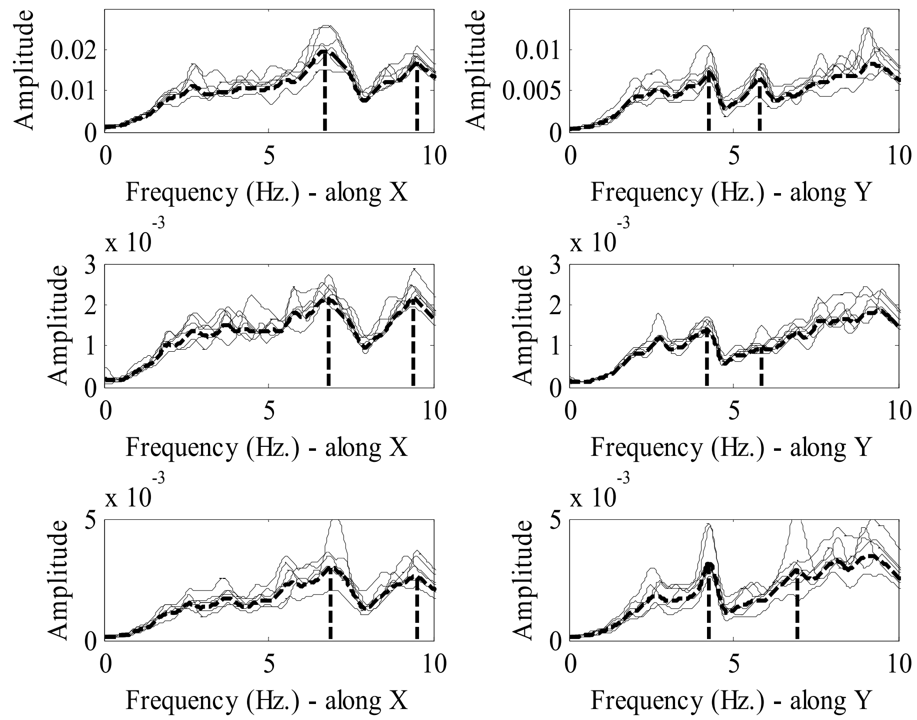


Fig. 3-2.32 Fourier spectrums

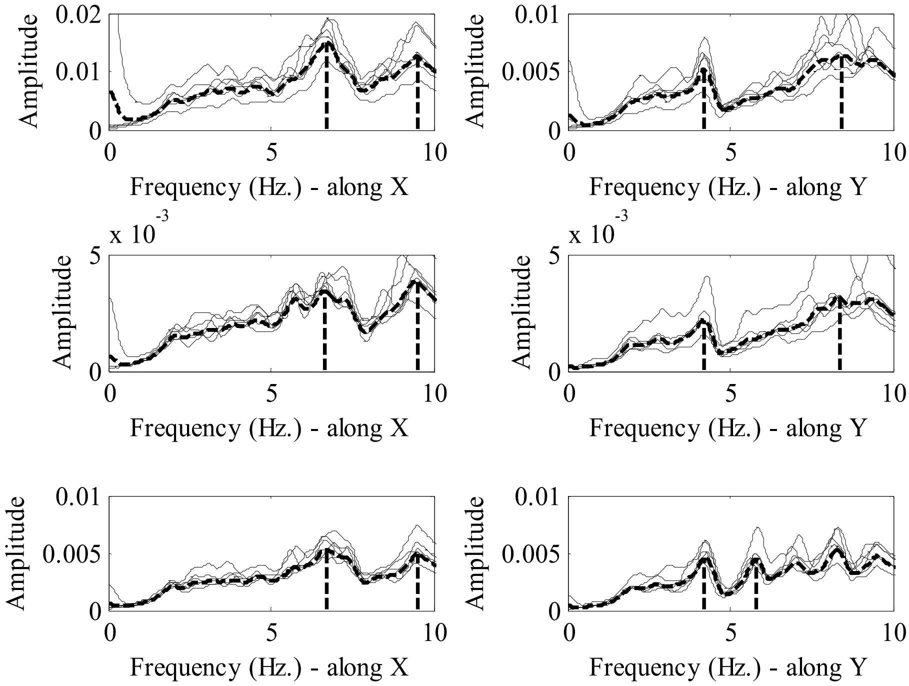


Fig. 3-2.33 Fourier spectrums

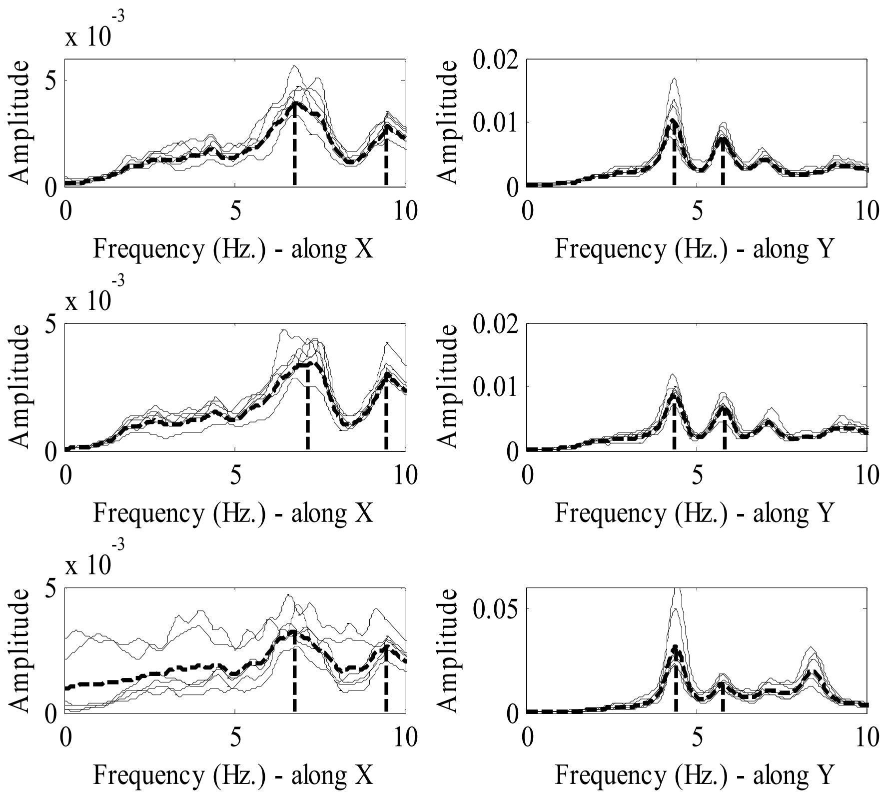


Fig. 3-2.34 Fourier spectrums

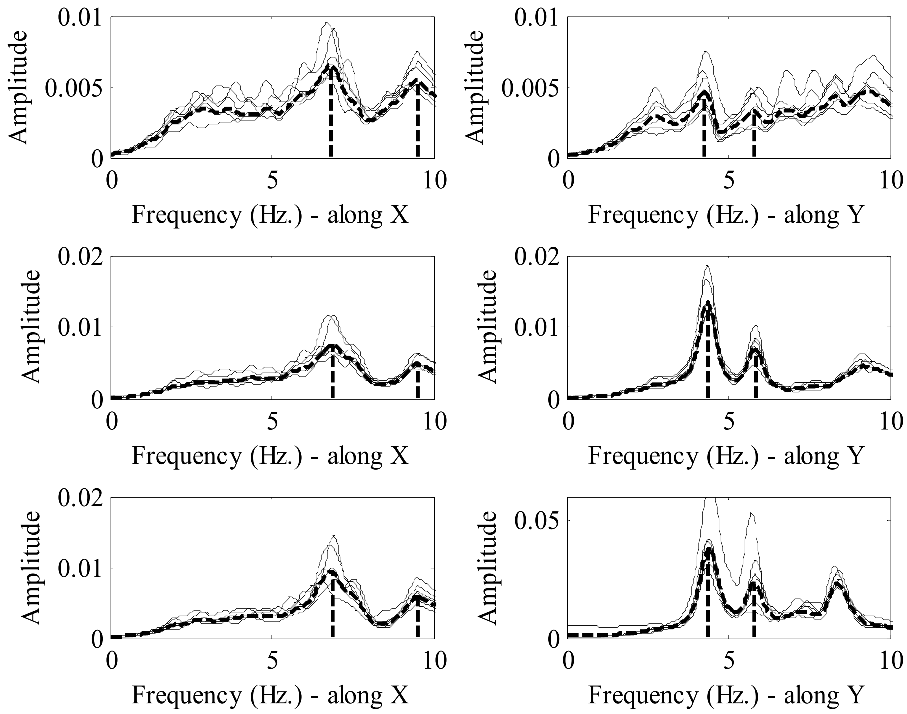


Fig. 3-2.35 Fourier spectrums

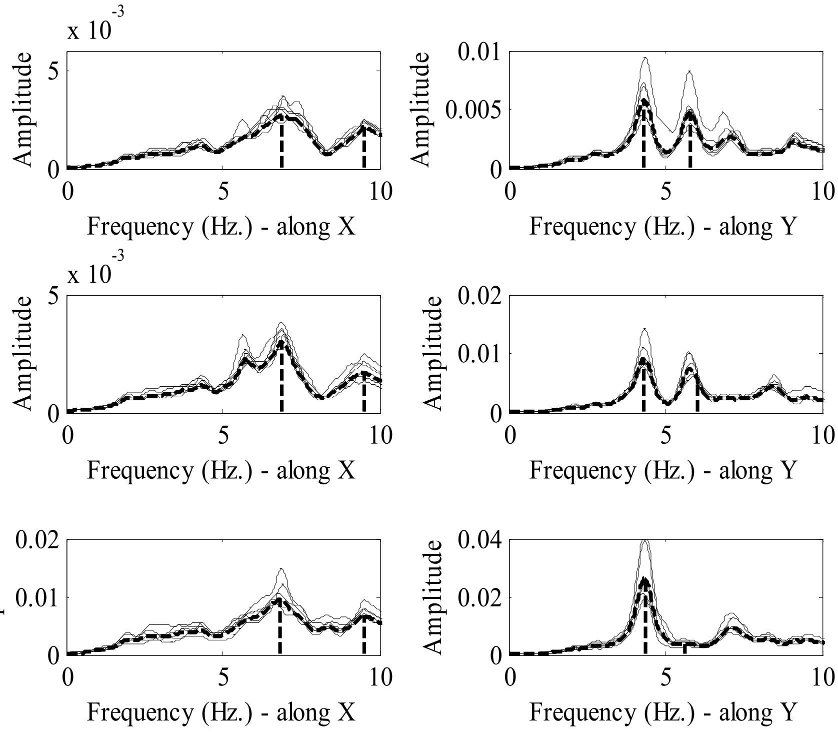


Fig. 3-2.36 Fourier spectrums

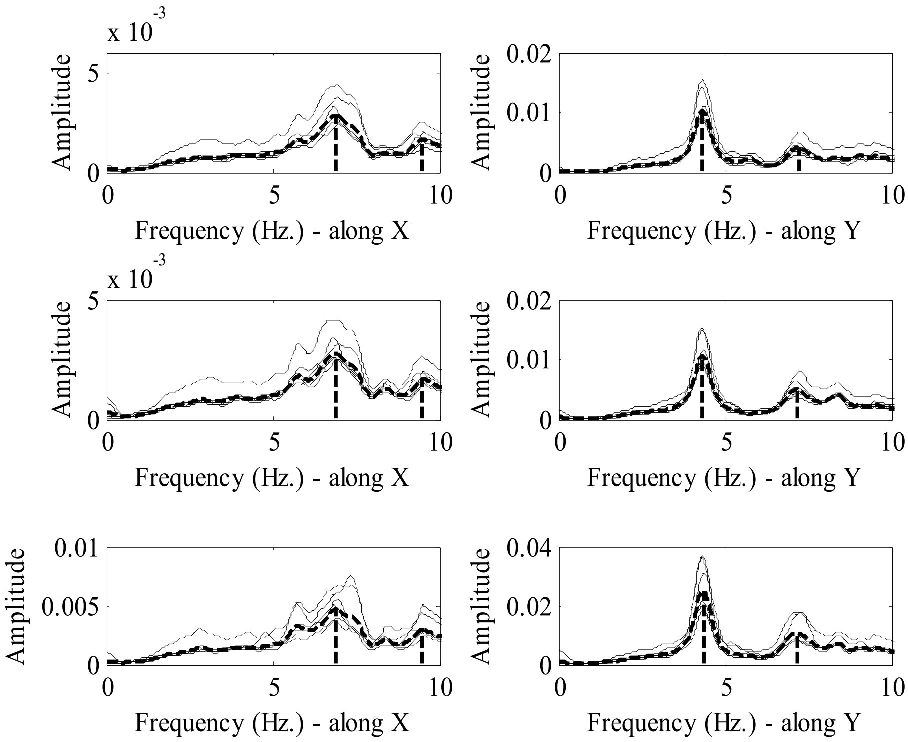


Fig. 3-2.37 Fourier spectrums

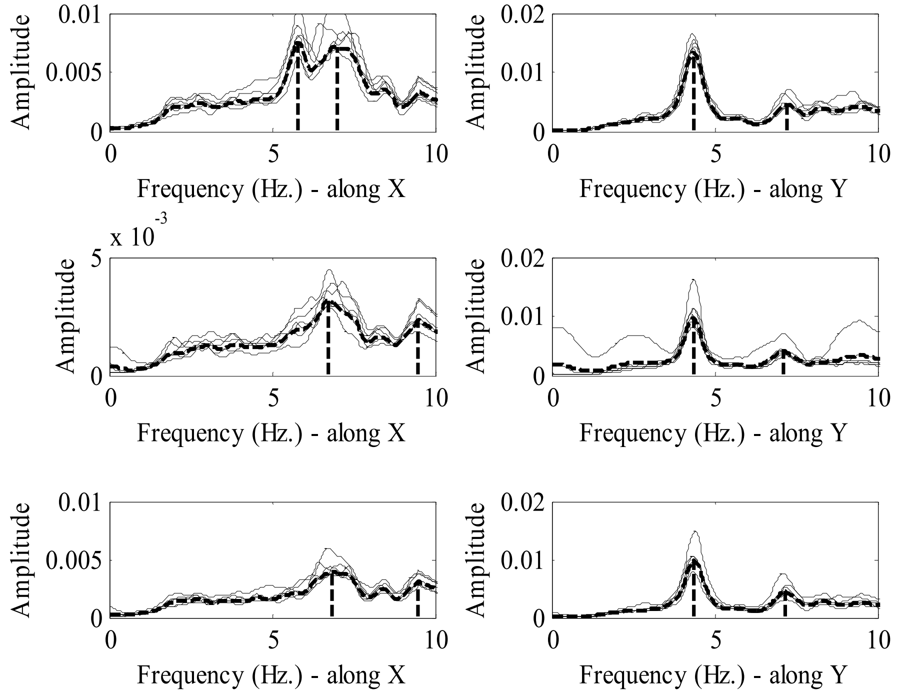


Fig. 3-2.38 Fourier spectrums

Table 3-2.11 Natural frequencies identified from two horizontal components

S.N	Plot figure	X		Y	
		mode 1	mode 2	mode 1	mode 2
1	3-2.31	6.86	9.50	4.30	5.84
2	3-2.31	6.84	9.50	4.30	5.79
3	3-2.31	6.84	9.50	4.30	6.91
4	3-2.32	6.74	9.50	4.27	5.79
5	3-2.32	6.81	9.40	4.22	5.88
6	3-2.32	6.91	9.50	4.27	6.93
7	3-2.33	6.74	9.49	4.20	8.45
8	3-2.33	6.64	9.50	4.18	8.35
9	3-2.33	6.71	9.50	4.22	5.81
10	3-2.34	6.81	9.50	4.35	5.79
11	3-2.34	7.18	9.50	4.37	5.84
12	3-2.34	6.76	9.50	4.19	5.81
13	3-2.35	6.84	9.50	4.25	5.79
14	3-2.35	6.91	9.50	4.37	5.84
15	3-2.35	6.86	9.50	4.39	5.81
16	3-2.36	6.91	9.50	4.32	5.79
17	3-2.36	6.89	9.50	4.32	5.81
18	3-2.36	6.84	9.50	4.35	7.15
19	3-2.37	6.91	9.50	4.32	7.20
20	3-2.37	6.89	9.50	4.32	7.15
21	3-2.37	6.91	9.50	4.35	7.18
22	3-2.38	5.81	7.01	4.37	7.25
23	3-2.38	6.74	9.50	4.35	7.10
24	3-2.38	6.86	9.50	4.37	7.18
Average		**6.80**	**9.39**	**4.30**	**6.52**

f) Transfer function and damping

In order see amplification of seismic motions, ratio of Fourier spectrum between first floor to ground floor, and second floor to ground floor was calculated and plotted in Fig. 3-2.39. It shows that the response along transverse direction is more amplified than in longitudinal direction. It also justifies the point that the building is vulnerable along transverse direction. From this ratio, the natural frequencies in rocking modes are identified at 4.69 and 5.85 Hz along transverse direction and 7.72Hz along longitudinal direction which is shown in Table 3-2.12. Other modes are not detectable.

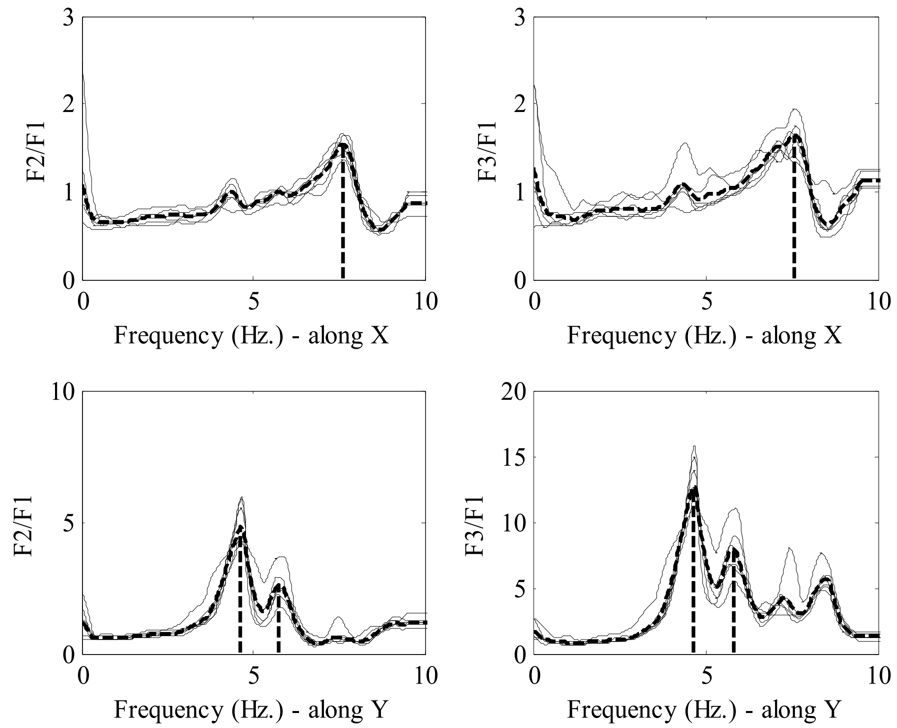


Fig. 3-2.39 Ratio of Fourier spectrum between first and ground floor

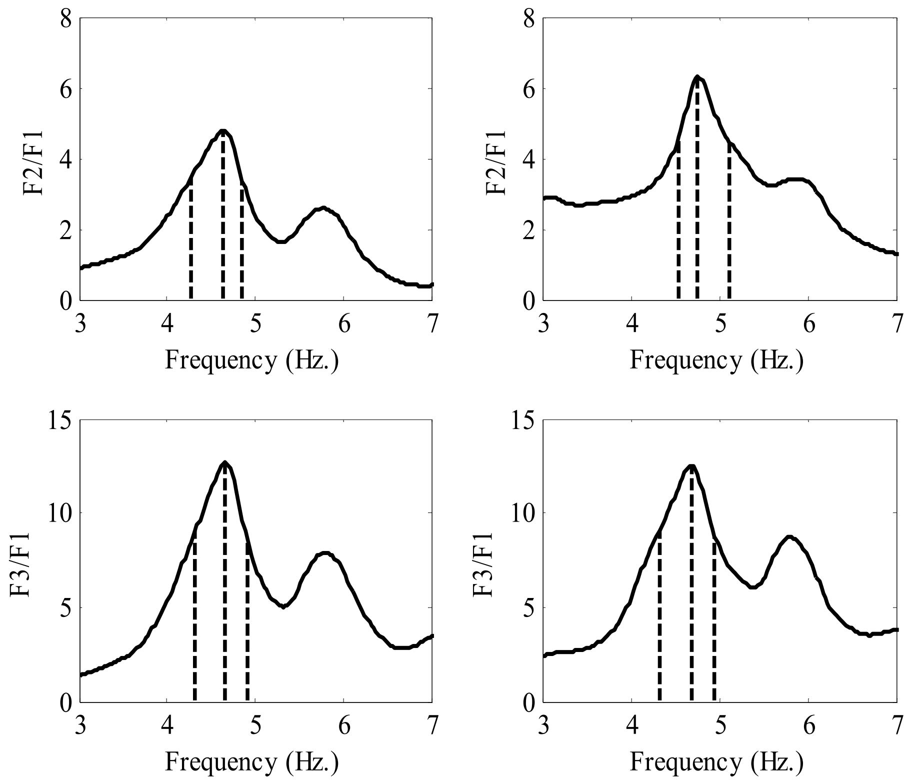


Fig. 3-2.40 Transfer function between the floors and ground responses

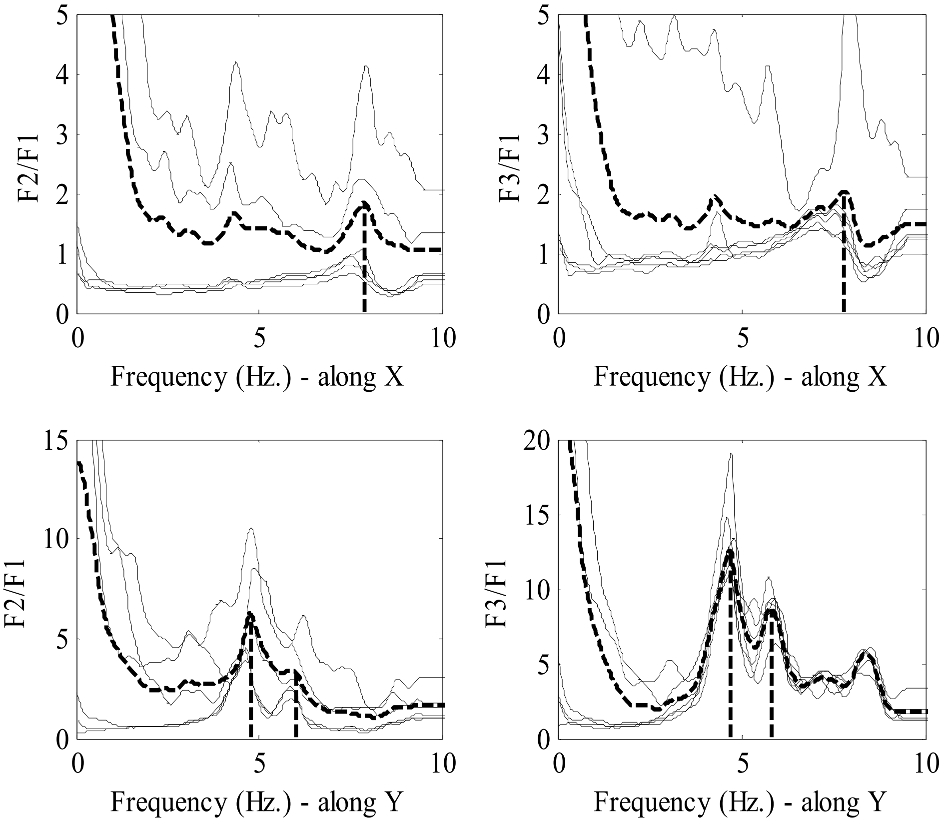


Fig. 3.41 Ratio of Fourier spectrum between first and ground floor after pushed

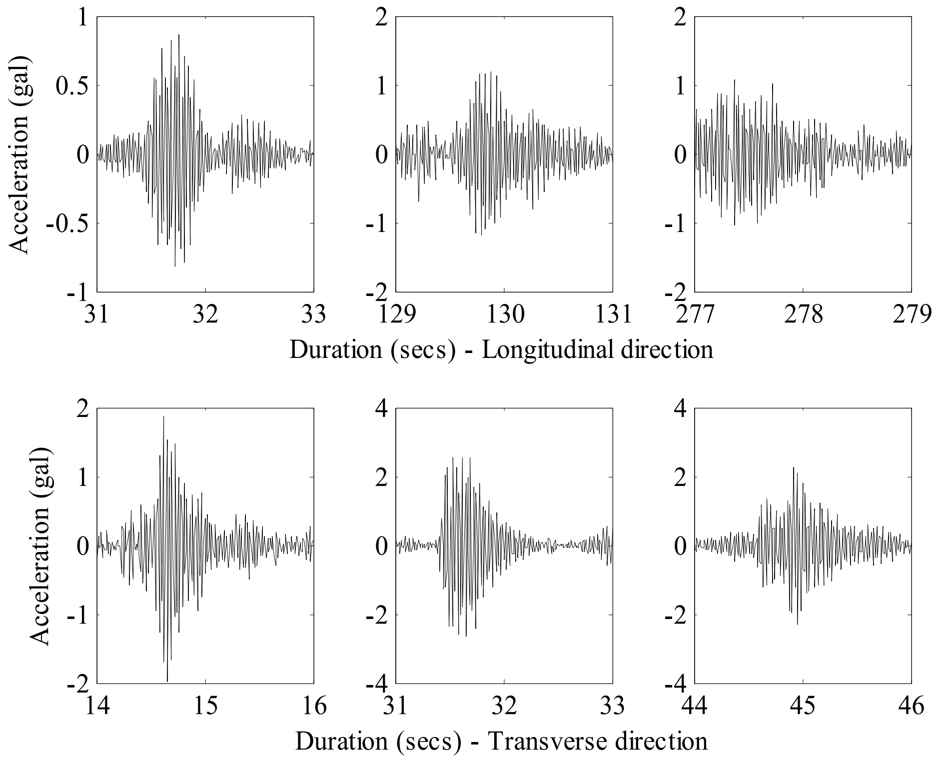


Fig. 3-2.42 Acceleration records after the buildings pushed

Table 3-2.12 Natural frequencies in rocking modes

	X		Y	
S.N	model 1	mode 2	model 1	mode 2
1	7.62	N/D	4.64	5.76
2	7.59	N/D	4.66	5.81
3	7.89	N/D	4.76	6.01
4	7.79	N/D	4.69	5.81
	7.72		**4.69**	**5.85**

Table 3-2.13 Damping calculated from transfer function

S.N	f_1	f_m	f_2	h
1	4.27	4.63	4.85	6.26
2	4.54	4.76	5.13	6.20
3	4.32	4.66	4.91	6.33
4	4.32	4.69	4.96	6.82
Average				**6.40**

In order to estimate damping, average value of transfer function obtained from the ratio between floors and ground responses are plotted in separate Figs. 3-2.39-3-2.41. Then damping is estimated by half power band width method 4) given by the equation taking the transfer function along transverse direction only and obtained results are shown in Table 3-2.13. Average damping value of damping is estimated 6.4%.

$$h = \frac{f_2 - f_1}{2 f_m} \tag{3-2.7}$$

Then, as an alternate method of estimation of damping, the building was pushed by 4-5 persons in both longitudinal and transverse direction. The micro tremor was taken when the building was pushed at each story. Then, ratio of Fourier spectrum between the floors and ground were plotted to see whether there is changes from the transfer functions plotted (Figs. 3-2.39-3-2.41) without pushing the building. No significant changes were noticed. Then, the time history records were cut just (Fig. 3-2.42) after the building was pushed and damping is calculated by random decrement method 4). From logarithmic decrement of wave, the damping ratios are obtained 3.6% and 5.2% along longitudinal and transverse directions respectively.

$$h = \frac{1}{2\pi} \frac{x_2}{x_1} \tag{3-2.8}$$

Micro-tremor ambient vibration measurements on a building and four different locations of Lalitpur sub-metropolitan city, Kathmandu, Nepal were carried out. From the measurements at grounds, H/V spectrum was plotted to estimate the predominant frequency of the ground and from the measurements at various locations of the building, natural frequencies of various excitation modes were identified. Damping of the building was investigated from random decrements, transfer function between ground and second story, and frequency domain decomposition. Mode shapes of the buildings were obtained from FDD methods. The results show that the building has dominant frequencies along shorter direction than in longer direction and damping of building is around 5%. This study is helpful to understand the dynamic behavior of building under normal loads as well as extreme loads such as those caused by seismic events or high winds.

Table 3-2.14 Calculation of damping from logarithmic decrement

SN	Longitudinal direction					Transverse direction				
	t_1	x_1	t_2	u_2	h	t_1	x_1	t_2	x_2	h
1	31.81	-0.793	31.86	-0.661	2.90	14.62	1.869	14.65	1.515	3.34
2	31.86	-0.661	31.91	-0.417	7.34	14.65	1.515	14.67	0.99	6.77
3	31.91	-0.417	31.94	-0.346	2.97	14.73	1.481	14.76	0.999	6.27
4	31.94	-0.346	31.99	-0.278	3.48	14.76	0.999	14.78	0.74	4.78
5	129.88	1.197	129.93	1.122	1.03	14.66	-1.973	14.69	-1.679	2.57
6	129.93	1.122	129.98	0.894	3.62	14.69	-1.679	14.72	-1.029	7.80
7	129.98	0.894	130.03	0.649	5.10	14.77	-1.274	14.74	-0.906	5.43
8	130.03	0.649	130.08	0.567	2.15	14.81	0.903	14.84	0.682	4.47
9	130.08	0.567	130.12	0.425	4.59	14.84	0.682	14.86	0.442	6.91
10	129.92	-1.02	129.97	-0.939	1.32	31.70	2.565	31.78	1.812	5.53
11	129.97	-0.939	130.02	-0.672	5.33	31.78	1.812	31.86	1.306	5.21
12	130.02	-0.672	130.07	-0.521	4.05	31.86	1.306	31.94	0.935	5.32
13	130.27	-0.823	130.32	-0.669	3.30	31.94	0.935	32.02	0.649	5.81
14	130.32	-0.669	130.37	-0.553	3.03	44.68	1.362	44.76	1.026	4.51
15	130.37	-0.553	130.42	-0.377	6.10	44.92	2.264	45	1.796	3.69
16	277.37	1.08	277.42	0.851	3.79	45.00	1.796	45.08	1.25	5.77
17	277.42	0.851	277.47	0.736	2.31	44.95	2.100	45.03	1.521	5.14
18	277.47	0.736	277.52	0.649	2.00	45.03	1.521	45.11	1.071	5.59
19	277.82	0.47	277.85	0.382	3.30	45.16	1.221	45.19	1.057	2.30
20	277.85	0.382	277.87	0.312	3.22	45.19	1.057	45.21	0.674	7.16
	Average				**3.55**	**Average**				**5.22**

Table 3-2.15 Summaries of results

S.N	Descriptions	Results
1	Predominant frequency of ground	2.1Hz
2	Natural Frequency along transverse direction sway	4.3Hz
3	Natural Frequency along Longitudinal direction sway	6.80Hz
4	Natural Frequency along transverse direction sway	4.69Hz
5	Natural Frequency along Longitudinal direction rocking	7.72Hz
6	Maximum damping	6.40%

The results show that the building has dominant frequencies along shorter direction than in longer direction and damping of building is around 3.55% along longer direction and 5.2-6.4% along shorter direction of the building. The key dynamic parameters are summarized in the Table 3-2.15. Since, the building has dominant frequency along shorter direction; damping estimate from longer direction may under estimate the actual value. Looking at all the analyses and Figs., the frequencies along transverse direction are more clear and detectable than in longitudinal direction. It is quite obvious that the building has big opening along transverse direction which makes stiffness weaker than in perpendicular direction. Higher modes' frequencies are not observing clearly in that direction. If its various vibration frequencies are not detecting from micro-tremor measurements, it is difficult to say exact vibration characteristics during strong motions. The damping estimate from unclear vibration modes may make wide difference than its actual value. Thus, the damping should be 5.2-6.4% for that kind of typical masonry buildings and other key parameters are given in the Table 3-2.15. Since, the dominant frequencies are seen along transverse direction, the building seems stronger along longitudinal direction. Large opening in the ground storey and big window at the first storey weaken the wall stiffness significantly which leads the building vulnerable along transverse direction.

References

1) Kramer, S.L.: Geotechnical earthquake engineering, *Prentice-Hall International series in Civil Engineering and Engineering Mechanics*, 1996
2) Parajuli, H. R., Kiyono, J., Maskey, P. N., Shiotani, T., Momoki, S., Toki, K., Masuda, K., Taniguchi, H., and Furukawa, A.: Parametric Study on Old Brick Masonry Walls by Elastic Wave Measurement, *28th conference, Japan Society for Natural Disaster Science*, Kyoto, Japan, Sept. 2009
3) Nakamura Y.: Clear identification of fundamental idea of Nakamura's technique and its applications, *Proceeding of 12th World Conference on Earthquake Engineering*, 2000.
4) Chopra A. K.: Dynamics of Structures; Theory and Applications to Earthquake Engineering, *Prientice Hall of India P. Ltd*, New Delhi, 1996.

(2) Laboratory Tests on Material properties of Traditional Masonry Buildings

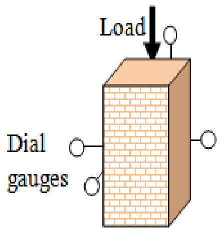
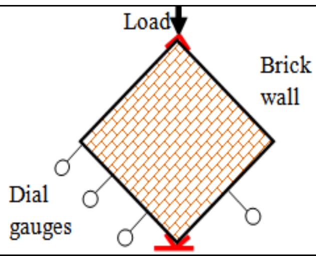
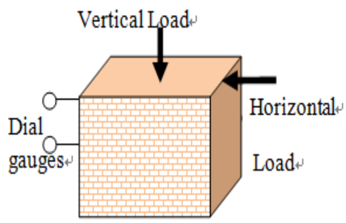
Brick masonry is heterogeneous construction in which bricks are laid one over another joined by mortar. Mortar can be mud (clay), lime, cement-sand etc. mixed with water in various proportions. In the Kathmandu Valley, modern masonry constructions are made by bricks with cement sand-mortar, but, in the past, most of the constructions had been made in mud mortar bonded bricks made of local technology since the beginning of the Malla period (300 years ago). There are hundreds of temples and residential buildings constructed with such traditional technology, which still exist and have been in usage. Most of them have gained aged value and are included as historical structures in the World Heritage Sites of the Kathmandu Valley. Deterioration of these structures due to aging and adverse environmental degradation is a prime concern.

Fig. 3-2.43 Compression loading

Fig. 3-2.44 Diagonal loading

Fig. 3-2.45 Combined loading

Table 3-2.16 Experiment plan

S.N	Description	Testing method
1	**<u>Compression</u>** Type: Bricks and mortar cubes, mud bonded wall Samples: 90 bricks, 11 mortar cubes and 3 walls Sizes: Length=35cm, Width=35cm & Height=35cm for wall, 48 mm cube mortar and approximately 45mm cut brick cubes Measure: Vertical load, vertical and lateral deformations. Plot: Compressive stress vs. strain Calculate: Ultimate compression stress, strain, elastic modulus and Poisson's ratio	
2	**<u>Shear</u>** Test: Diagonal shear Type: Mud bonded wall Samples: 4 Size: Length=60cm, Width=60cm & ,Thickness=35cm Measure: Force and deformation Plot: Shear stress vs. strain Calculate: Ultimate shear stress, shear strain and shear modulus	
3	**<u>Combined horizontal and vertical loading</u>** Type: Mud bonded wall Samples: 5 Sizes: Length=35cm, width=70cm and height=70cm Measure: Vertical load and horizontal load Plot: Horizontal and vertical stress relationship Calculate: Elastic modulus, equivalent cohesion and tangent angle	

For step towards combating the existing low strength masonry structures is to find basic material properties such as modulus of elasticity, strength in compression, tension and shear. Strengthening of these structures requires rigorous analysis using more precise properties and strengths. Advanced researches on reinforced and steel structures have already been done and some research may have been done on reinforced masonry or unreinforced masonry structures such as those lime or cement mortar bonded brick masonry. Some investigations 1, 2) regarding the material of similar kind of bricks and brick walls in the laboratory Institute of Engineering (IOE). But, they have only investigated on cement-sand mortar bonded masonry specimen focusing the modern constructions and there are no investigations on mud bonded masonry buildings have been reported so far. Thus, an experiment was planned and done which has been summarized in Table 3-2.16. Brick units, mortar cubes and wallets were applied three kinds of loadings. They are diagonal shear, vertical compression and combined vertical and lateral loads. The numbers of samples, sizes, method of load application, reading of load and deformations etc. have been given in the Table 3-2.16.

Main purpose of this test was to take the core samples from the existing buildings and test in the lab. But, it was not possible because of various reasons such as difficulty of transportation, and stability of the core after taking out of wall, and owners do not want to drill in their walls. So, bricks fabricated in Malla period were collected from the old buildings which were dismantled recently. Sample brick wallets were constructed at IOE, Pulchowk Campus. Using IOE laboratory testing facilities, three kinds of tests – compression, shear, combined shear and compression loading tests were conducted (Figs. 3-2.43 –3-2.45).

a) Compression testing

The compressive strength of masonry is measured by the materials being used to build with. An individual brick's compressive strength is a physical property that measures how compressed the material is, which determines how much strength a brick offers a structure. A brick's compressive strength is based on its solidness or hollowness, its base material and the method is used to fire the raw materials into their final consistency. When bricks with a high compressive strength are used to build a structure, that structure will be very strong because of the quality of the materials. Compression loads (Fig. 3-2.43) were applied on 3 walls and the compressive load versus deformations has been deformations were recorded at various intervals of loadings. The results initial stress and strain, modulus of elasticity (E), Poissons`s ratio (□), shear modulus (G) and shear wave velocities (V_S) of the walls are given in the Table 3-2.17. Same symbols have been used in the following sections also. All the properties have been calculated from initial tangent stress strain ratios. Shear modulus and shear wave velocity is calculated and summarized in the Table 3-2.17.

Table 3-2.17 Experiment results from compression test

S.N.	Initial stress N/mm^2	Initial strain	E (N/mm^2)	n	G (N/mm^2)	V_S (m/sec)
1	0.10	0.00043	234	0.32	88	347
2	0.10	0.00037	270	0.24	109	373
3	0.10	0.00031	319	0.17	136	406
Average			274	0.24	111	375

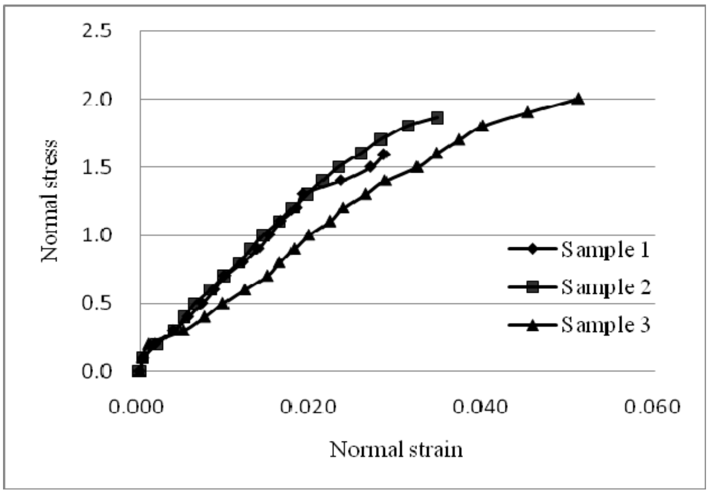


Fig. 3-2.46 Compressiive stress and strain relation

Compressive stress and strain obtained from the test is plotted in Fig. 3-2.46. At the beginning, the ratios which is in fact modulus of elasticity, is higher and after small increment of loads it starts cracking and the ratio drops. It is because of cracks at mortar joints. Again after few steps of loadings bricks start taking loads and the relationship goes linear.

b) Shear test

Four sample walls were placed diagonally on the testing plat form as shown in table 3-2.16 and Fig. 3-2.44. The load was applied at the top and increased gradually. Deformations of wall along the bed of the joint were measured. The dimensions, shear height and obtained shear modulus have been given in the Table 3-2.17. Shear modulus is calculated from the ratio of shear stress and shear deformation. The shear stress and stain relationships are plotted in the Fig. 3-2.47. As the load increases, the stress strain behavior gets nonlinear. Thus shear modulus is calculated from initial three values which are very close before starting to decline. Taking Poisson`s ratio equal to 0.24 obtained from the compression test modulus of elasticity is calculated. The obtained results are given in Table 3-2.18. Average value of shear modulus, elastic modulus and shear wave velocity are found to be 250 N/mm^2, 621 N/mm^2 and 366m/sec. respectively. Similarly, average shear stress and stain at ultimate stage are found to be 0.126N/mm^2 and 0.00608 respectively. Shear strength of the wall is governed by the mortar/interface which comes from friction due the asperities between the surface of mortar layer and the surface of the brick unit, and the bond between mortar and brick units. Normal compression perpendicular to the interface further increases its shear strength because the asperities cannot easily slide over one another. In the Fig. 3-2.47, shear stress and strain relationship looks linear, however, it behaves non linearly. Only in the very small stress levels the wall as a whole behaves and the shear modulus looks linear. However, quickly after increasing the loads, the wall tries to shear at the joint. Then crack initiates at the weakest bed and starts to slip forming likes two rigid bodies. Thus, the deformation is controlled by the mortar joint and the shear modulus for joint and wall remains same as the deformation phenomenon totally governed by joint.

Table 3-2.18 Results from diagonal shear tests

Test	Length mm	Width mm	Area mm^2	Height mm	G N/mm^2	ν	E N/mm^2	ρ kg/m^3	V_S m/sec	Stress N/mm^2	Strain
1	585	365	213525	585	225	0.24	557	17.68	353	0.154	0.00582
2	585	365	213525	585	280	0.24	695	17.68	394	0.128	0.00599
3	585	365	213525	255	352	0.24	874	17.68	442	0.128	0.00606
4	585	365	213525	485	137	0.24	339	17.68	275	0.097	0.00646
Average					**250**	**0.24**	**621**	**17.68**	**366**	**0.126**	**0.00646**

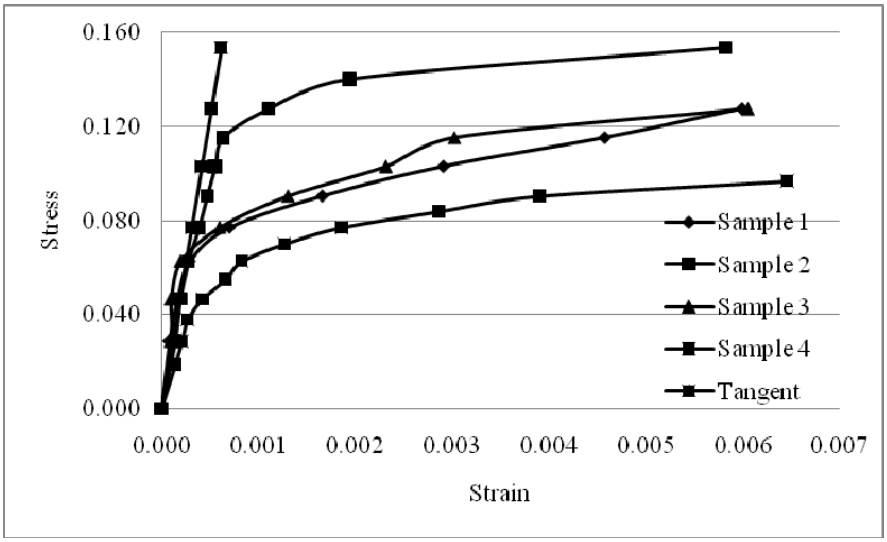


Fig. 3-2.47 Shear stress strain relation

c) Combined shear and compression

Five wallets were given vertical 1 ton load initially and then lateral load were applied at upper edge to the specimen (Fig.3-2.45). The deformations at each load increment were noted and load displacement curve is plotted. From the plot modulus of elasticity was calculated from the initial tangent stiffness (k_0) obtained from the initial load divided by initial deformation. The results are given in Table 3-2.19. In the Table, □$_0$ is horizontal displacement at initial force P_0. For historical masonry structures, it is widely accepted to examine E and G for a wall as a whole, rather than for the constituent materials, since brick masonry is not an elastic, homogeneous, or isotropic material. Because the value of initial stiffness k_0 represents the elastic stage of the wall, k_0 can be calculated (eqn. 3-2.9) for walls with the fix-ends against rotation as given by Drysdale and Hamid 3).

$$k_0 = \frac{Et}{\left(\frac{h}{l_w}\right)^3 + \left(\frac{1.2}{G/E}\right)\left(\frac{h}{l_w}\right)} \tag{3-2.9}$$

Table 3-2.19 Results from combined lateral and vertical loadings

S.N.	P_0 KN	Δ_0 mm	k_0 KN/mm	E N/mm^2	ν	ρ kg/m^3	G N/mm^2	V_S m/sec
1	6.03	0.07	86	955	0.25	17.68	382	444
2	6.03	0.08	75	836	0.25	17.68	334	415
3	6.03	0.14	43	477	0.25	17.68	191	314
4	6.03	0.12	50	557	0.25	17.68	223	339
5	6.03	0.20	30	334	0.25	17.68	134	263
Average			**57**	**632**	**0.25**	**17.68**	**253**	**355**

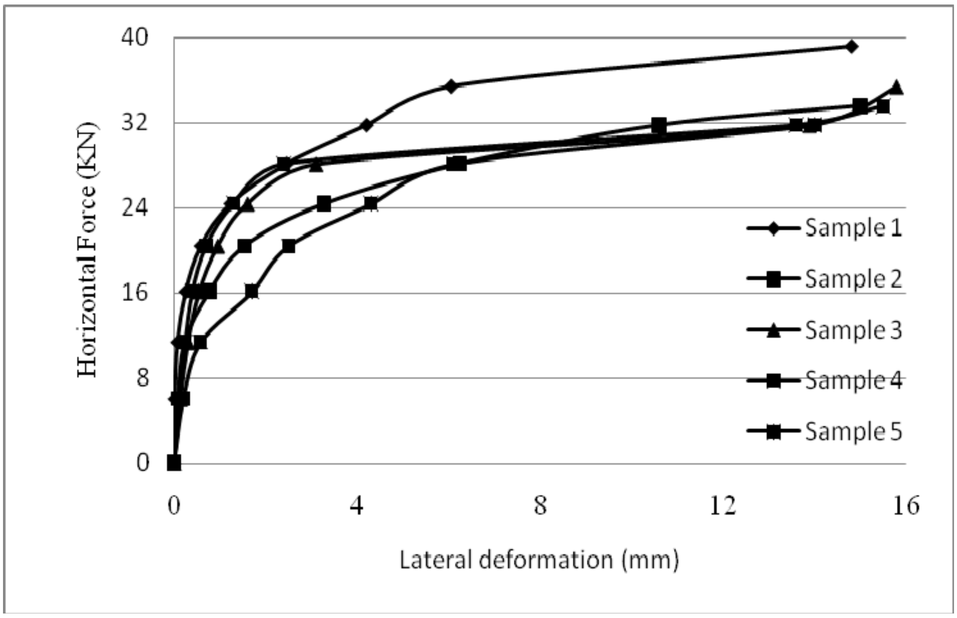


Fig. 3-2.48 Horizontal deformation of wall on 1 ton vertical force

Horizontal loads were gradually increased on four walls with vertical loads 1.0, 1.2, 1.4 and 1.6 tons. The loads were kept constant and horizontal load was increased until the wall fails. The obtained results are given in Table 3-2.20 and plotted in Fig. 3-2.49.

Table 3-2.20 Relationship between shear and normal stresses

S.N	Horizontal force KN	Vertical force KN	σ_n N/mm^2	τ N/mm^2	Remark
1	30.10	9.81	0.123	0.040	Equivalent Coulomb parameters C=0.0857 tanΦ=0.9174
2	31.50	11.77	0.129	0.048	
3	33.90	13.73	0.138	0.056	
4	35.30	15.70	0.144	0.064	

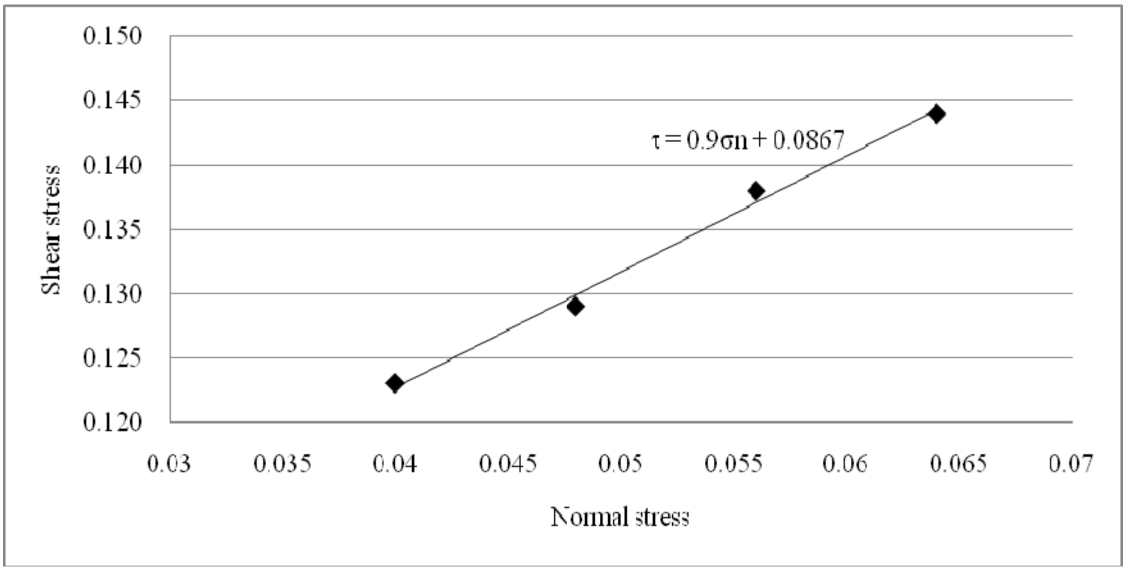


Fig.3-2.49 Relationship between normal and shear stress

Table 3-2.21 Summary

<table>
<tr><th>S.N</th><th>Type</th><th>Density kg/m^3</th><th>Compressive Strength N/mm^2</th><th>Shear Strength N/mm^2</th><th>G N/mm^2</th><th>υ</th><th>E N/mm^2</th><th>V_S m/sec</th></tr>
<tr><td>1</td><td>Brick</td><td>1768</td><td>11.03</td><td></td><td>1740</td><td>0.11</td><td>3874</td><td>1037</td></tr>
<tr><td>2</td><td>Mortar</td><td>1705</td><td>1.58</td><td rowspan="2">0.15</td><td rowspan="2">318</td><td rowspan="2">0.25</td><td rowspan="2">794</td><td rowspan="2">402</td></tr>
<tr><td>3</td><td>Wall</td><td>1768</td><td>1.82</td></tr>
</table>

In this section, we have investigated the mechanical properties of masonry from experiments by various methods. Masonry is a composite material of brick units and mortar joints and interface between mortar and unit. Together, they determine the properties of masonry. The interface is known as the weak link in the system with minimal tensile bond strength, thus masonry has limited tensile strength and usually negligible. Under uniaxial compression, state of stresses in the brick in a masonry prism is compression-tension-tension; whereas, softer mortar joint is under tri-axial compression. Under tension, masonry is linear elastic material; tensile failure is characterized by splitting along the interface. Masonry exhibits nonlinearity under biaxial loading. Cyclic loading reduces compressive strength of masonry prism by 30%. Shear behavior of masonry depends upon normal stress; under high normal stress, dilatancy is insignificant. Mohr-Coulomb model is appropriate for modeling shear behavior in joint.

References

1) Shah, U. S.: Performance of cement mortar in brick masonry, *Master's thesis,* TU, IOE, Pulchowk Campus, Nepal, 2002.
2) Pun, R : Response of load bearing brick masonry: Experiment and numerical simulations, *Master's thesis,* TU, IOE, Pulchowk Campus, Nepal, 2005.
3) Drysdale, R., and Hamid, A. : Masonry structures: behavior and design, Canada Masonry Design Centre, 2005

(3) Structural analysis of historical masonry building

With a purpose to develop the strengthening technique of the heritage buildings, as a primary step towards seismic disaster risk mitigation, the structural analysis of the historical masonry building, Lalitpura pati (Fig.3-2.50) is carried out. The selected historical masonry building is located in the Jhatapol area of the Patan city. Following the investigation of the material properties and determination of other structural parameters, as described in earlier sections, a numerical model of the building is prepared for analysis. The structural analysis is carried out by using Finite Element Method (FEM) for various possible earthquake ground motions as seismic input.

Masonry behaves distinct directional properties due to the interfaces between the constituent elements. The large number of influencing factors such as interior voids, anisotropy of bricks, dimension of bricks and joints, arrangement of bed and head joints and quality of workmanship make the numerical model of masonry wall very complex. Limited number of variables that are used in the numerical model cannot reflect the actual behavior of brick walls due to its variety of properties. Basically, two methods are used for analyzing masonry houses – one, distinct element method (DEM) which considers the brick units as non-deformable solids and their movements are evaluated through equation of motions, and the other, finite element method (FEM), which considers the masonry wall as a deformable element. The FEM analysis has a wide scope, starting with a simple approach with consideration of masonry as a single phase material to a very sophisticated approach with assumption of masonry as a two or multi-phase material. The versatility of the FEM method has made it popular and has become a well accepted tool.

Fig. 3-2.50 Lalitpura pati, the historical brick masonry house at Jhatapol area

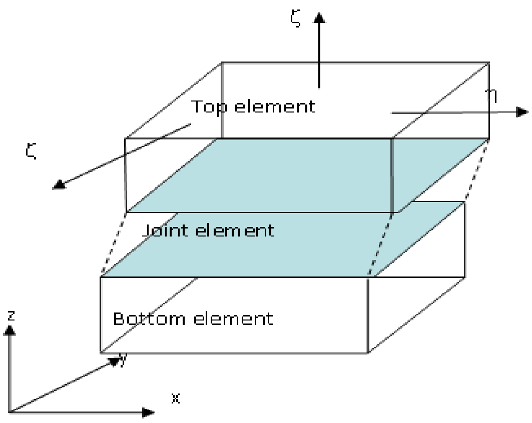


Fig. 3-2.51 Formulation of solid and joint elements

Since the brick masonry walls are composed of brick units and joints (Fig. 3-2.51), they behave non linearly governing the entire deformation phenomenon by weak joints. The ordinary FEM which is based on continuum mechanics may not be applicable for such kind of problems. Thus, the modified FEM considering discontinuities has been used in the field of rock mechanics 1) and recently, it has been used in brick and stone masonry 2,3,4) to simulate time dependent sliding and separation along the mortar joints. This concept has also been applied to investigate the effectiveness of wooden beams in dry stone masonry houses 5). Thus, similar idea of modeling the brick units as solid elements and interfaces between them with zero thickness of joint elements is employed here. However, there is large number of bricks in a house and modeling of each brick separately is very complicated and practically impossible. Thus simplified numerical model is developed making equivalent eight noded elastic solid elements for brick wall blocks and eight noded joint elements 2) for interfaces between brick elements.

For FEM modeling, the parameters such as density, modulus of elasticity, Poisson ratio, spring constants for joints along normal and shear directions are very much essential. The material properties were taken from the experiments described in the previous sections. The stiffness constants along shear and normal directions are calculated by considering the differences of module of elasticity between a single brick and wall. The wall is represented by series of two springs, one by brick element and other by joint 4) as shown in Fig. 3-2.51. The concept of joint element is to represent the non linear behavior of the adjacent elements. The model has elements with varying thickness, therefore separate values of coefficients are required depending upon their depth. However, in this study an average value of depth 25cm has been taken and calculated corresponding values of normal and shear stiffness coefficients are 3.4GN/m^3 and 1.4GN/m^3 respectively. Unit weight and modulus of elasticity for wooden elements are 4.47KN/m^3 and 8.1xGN/m^2 respectively. Regarding the damping, very limited information is available in linear solid mechanics problem, and even very less information is available in non linear dynamic analysis. For problem under consideration, the Raleigh coefficients α=0.0174 and β=0.172 are taken maintaining the damping approximately 3% following Wakai and Ugai 6).

The model building (3-2.50) has been divided into two rooms longitudinally is modeled as shown in the Figs. 3-3.52-3-2.53. It is a double storeyed building; 16.5 m long and 5.6 m wide. The wall is made of traditional brick and 60cm thick at the bottom and 50 cm at the top, a slight tapering from bottom to top. An average thickness 55cm is taken for an analysis. It was constructed three hundred years ago. It sustained damages in earthquakes and repaired many times. Recently, its original roof has been replaced by corrugated galvanized iron sheet which rests over wooden planks and battens and the interior wall has been plastered by cement sand mortar. The floor has been recently replaced by concrete which rests over wooden boards supported by planks and beams. Now, it is repaired hiding its original construction and has been using as public purpose. The building has very large opening in the front side. Wooden posts are supporting the wall of upper storey. In the upper storey, there is a big wooden window placed at mid span of the wall and is slightly projected to outside showing a nice aesthetic view.

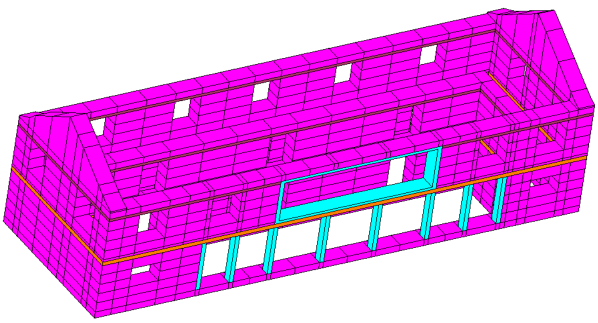


Fig. 3-2.52Simplified FEM model

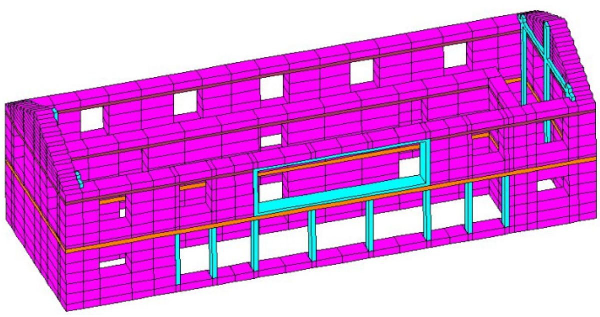


Figure 3-2.53 Modified FEM model

In the numerical model, vertical wooden posts have been put in the ground floor and an equivalent wooden frame has been put to replace the big window of the upper storey. Separate modelings of walls, floor, roof, and their component-bricks, windows, doors, posts are extremely complicated jobs. Small partition walls have not been considered in the model. Simplified model considering load bearing walls have been constructed (Fig. 3-2.52). The wall is discretized into small number of solid brick elements, vertical posts are modeled as wooden solid and the big window of the upper storey is modeled as equivalent solid elements. Total elements are 2995, solids are 1186 and joint elements are 1809. As a possible strengthening solution, two wooden frames have been added inside the building (Fig. 3-2.51). The different colors shown in the models show the different materials. In the first model (Fig. 3-2.52), there are joint elements between the different materials whereas in the second (Fig. 3-2.53), model has been modified putting the rigid joint between the posts and connecting elements, floor elements and window elements. The added frame should be fixed into the wall and thus the connectivity between them is considered rigid. These changes make significant changes in the numerical model though they look similar in the Figs. Wooden elements and bricks are inter-connected at floor and roof levels, they behave like semi-rigid floor diaphragm. If theses element are considered separately the model becomes very complicated and thus, floor mass is lumped at wall where the floor and roof rest. To differentiate the material properties separate color can be observed in the model (Figs. 3-2.52- 3-2.53). During lateral loadings, floor acts rigidly and the corresponding solid elements at floor and roofs are assigned rigid with same material properties. Joint elements are provided to connect the floors with walls. Total loads of floors were calculated 1.5KN/m^2.

At first, static analysis was run for vertical loads and self weights. And obtained stresses were used in dynamic analysis as initial stresses. In the second step, dynamic analyses were run with an input of Kobe1995 earthquake, El Centro 1940 earthquake, simulated for 98 years and 475 years return period (Fig. 3-2.54). Equations of motions were evaluated at 0.01 interval of time by Newmark's beta method satisfying the Mohr-Coulomb criterion for slide and separation 3). The residual forces obtained from deducting actual force developed and permissible force calculated from constitutive relationship produces non linear deformation at the joints which are evaluated by Newton Raphson method.

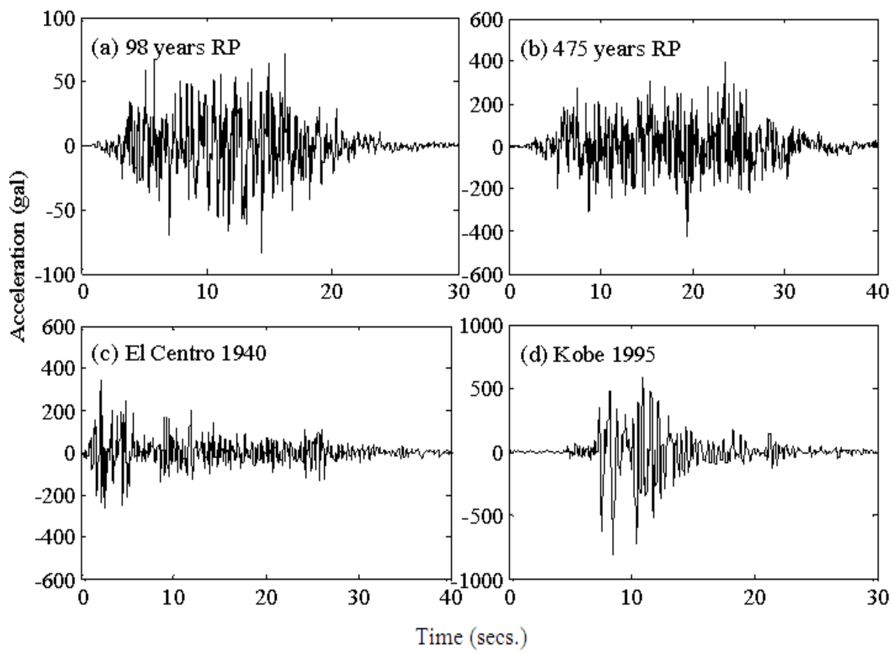


Fig. 3-2.54 Input ground motions

In full three dimensional analyses, there is possibility of obtaining tensile and compressive forces right from the beginning. So, the building experiences tension at some areas, most likely near openings and at other weak zones which goes on iteration and finds non linear deformations. If the residual forces are big, iteration takes very long time and ultimately computation becomes very lengthy. Since, there are no predefined criteria to define failure of masonry buildings; a ceiling value-30cm displacement has been set in the program. Normal length of brick is 23cm, and, if deformation exceeds 30 cm, it completely dislocates by its original position. However, it is arbitrarily assumed value and one can take its own definition and value. The purpose of termination of analysis is just to save the time only. The building experienced more than 30cm displacements in El Centro 1940, Kobe 1995 and 475 years return period earthquakes in few seconds. The deformations obtained by various time histories are shown in Figs. 3-2.55 – 3-2.60. The deformations of elements are shown in different colors (values are in meters). The deformation depends upon amplitude, frequency content and duration of earthquakes. The building sustains very large deformation quickly in Kobe 1995 earthquake since it has highest amplitude. It also gets large deformation in El Centro 1940 earthquake simulated for 475 years return period after few seconds (Figs. 3-2.55-3-2.60). The difference between them is time only. In all cases the large deformations can be seen in the gable wall and near the large openings which is quite expected. The deformations are bigger along in Y (shorter) direction than in X (longer). It is usual because the wall stiffness is greater along X than in Y.

Dislocation of vertical posts and initiation of deformations also can be seen in the simulated 98 years earthquake, however, the values are very small (Fig. 3-2.59). This shows that the building is able to resist the around 100 gal or less amplitude earthquakes, but, it cannot resist even greater than 150 gals acceleration since it has sustained large deformation in 50% reduced El Centro earthquakes (Fig. 3-2.58). However, duration and frequency content should also be considered while defining complete failure. As reported in contemporary literatures, El Centro 1940 earthquake is the first recorded earthquake and 40% houses had been damaged. It is quite reasonable to say that this brick masonry house sustain very large cracks and deformations and cannot survive. The building sustains very large deformations under all given earthquakes and it is proved to be very weak and cannot take any kind of severe earthquake loadings.

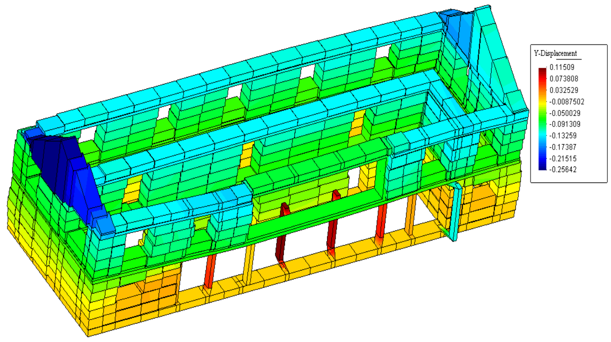


Fig. 3-2.55 Deformations in 475 years RP earthquake

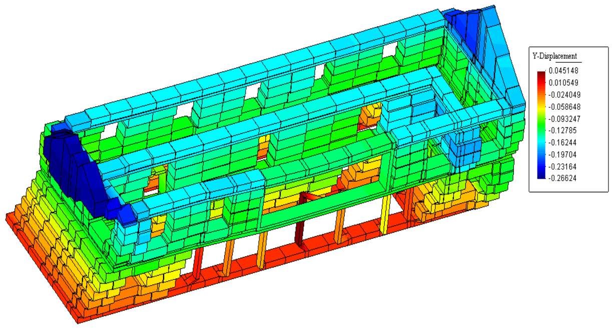


Fig. 3-2.56 Deformations in Kobe 1995 earthquake

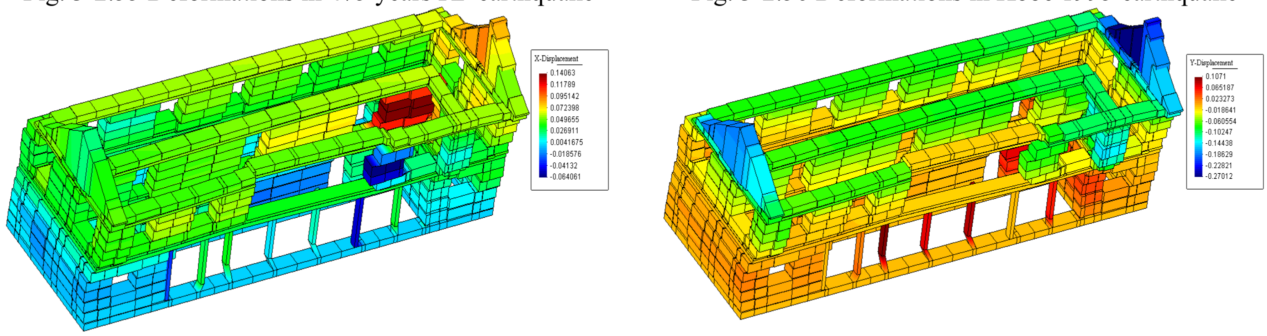


Fig. 3-2.57 Deformations in El Centro 1940

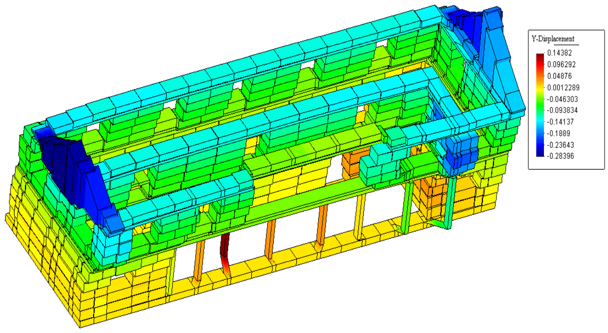


Fig. 3-2.58 Deformations in 50% de-amplified El Centro 1940

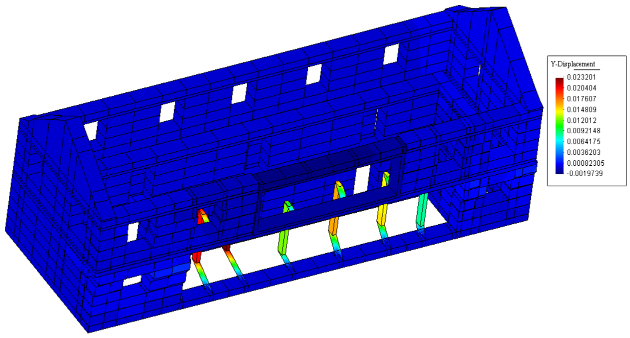


Fig. 3-2.59 Deformations in 98 years RP

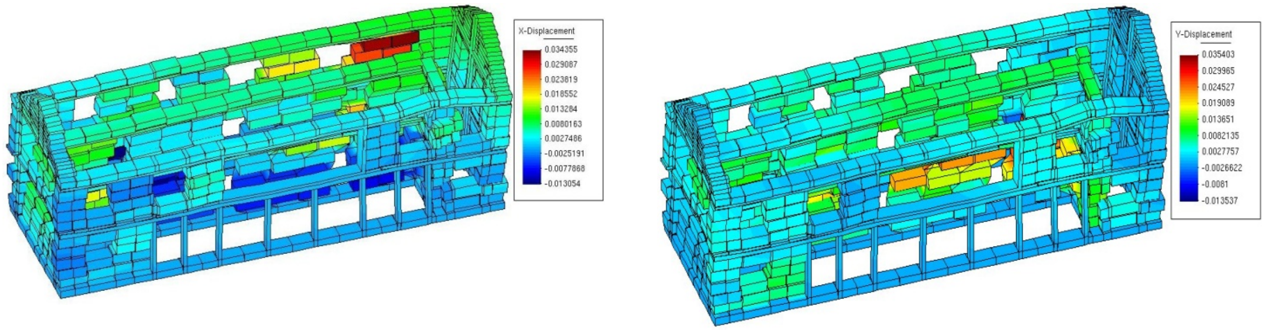


Fig. 3-2.60 Deformations on modified in El Centro 1940 earthquake

Being historical building, it has heritage value and should be protected against future earthquake. There could be many possible methods of strengthening of buildings, however, archeologists and conservationists do not allow intrusion by all kind of materials such as concrete, steel, FRPs etc. Thus, there are very few options remained; for example, addition of wooden beams and column internally could be one of the possible options. Thus, looking at the weak zones, near the openings and top of the shorter walls, strengthening measures are applied; joints between the posts and the connecting elements are made fixed, the floor elements are connected with the wooden elements placed around the openings, wooden beam and posts (Fig. 5-2.53). Then, the house is analyzed again in the El Centro 1940 earthquake ground motion. During full cycles of analysis it gets maximum displacement 3.4 cm (Fig. 3-2.60) along Y direction. It shows that simple method of strengthening can contribute significant strength and reduce the large deformation. Being old brick masonry, the wall is already stressed and propagation of crack is obvious even in small deformations. Though it might not be serviceable after earthquake, it may protect the lives. And also, wood is easily available in local areas and easily acceptable by the heritage conservation community, thus, becomes good strengthening alternative for those kinds of buildings.

References

1) Zienkiewicz, O.C., Best, B., Dullage, C. and Stagg, K.G.: Analysis of non linear problems in rock mechanics with particular reference to jointed rock systems, *Proceedings of 2nd International conference,* Society of Rock Mechanics, pp501-509, Belgrade, (3), 1970.
2) Tzamtzis, A.D. and Nath, B.: Application of three-dimensional interface element to non -linear static and dynamic finite element analysis of discontinuous systems, PD-Vol. 47-1, *Engineering System Design and Analysis*, (1), ASME, 1992.
3) Tzamtzis, A.D. and Asteris, P.G.: FE analysis of complex discontinues and jointed structural systems, *Electronic Journal of Structural Engineering*, (1), 2004.
4) Senthivel, R., Lourenco, P.B. and Vasconcelos, G.: Analytical modeling of dry stone masonry wall under monotonic and reversed cyclic loading, *Structural Analysis of Historical Constructions*, New Delhi, 2006.
5) Parajuli, H., Kiyono, J. and Ono, Y.: Effectiveness of wooden bond beams in dry stone masonry houses, *Journal of Applied Mechanics, JSCE*, Vol. 11, pp.615-623, 2008.
6) Wakai, A. and Ugai, K.: A simple constitutive model for the seismic analysis of slopes and its applications, *Soil and Foundation, Japanese Geotechnical Society*, Vol. 44, No. 4, 83-97, 2004.

3-2.4 Simulation of Building Collapse

Aiko Furukawa
Junji Kiyono
Hitoshi Taniguchi
Kenzo Toki
Masatoshi Tatsumi

(1) Introduction

In this section, seismic damage to an existing historic masonry building in Jhatapo area is simulated using the refined version of the DEM. Results for three input ground motions with different occurrence probabilities, two cases with different input ground motion directions are compared.

(2)Analytical Method

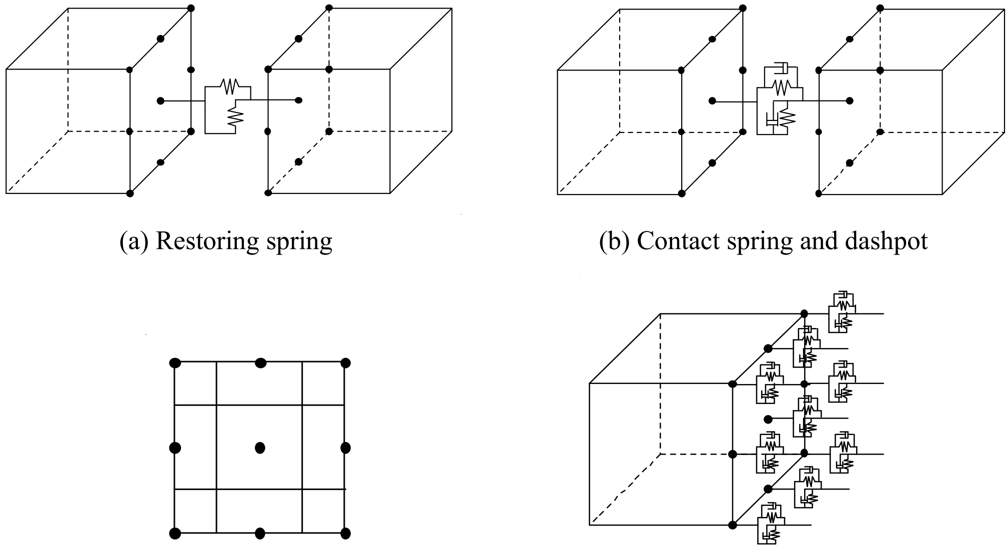
a) Basic concept

This study employs a refined version of the DEM[1)] to simulate a series of structural dynamic behaviors from elastic to failure to collapse phenomena. A structure is modeled as an assembly of rigid elements, and interaction between the elements is modeled with multiple springs and multiple dashpots that are attached to the surfaces of the elements. The elements are rigid, but the method allows the simulation of structural deformation by permitting penetration between elements.

Fig. 1 (a) shows a spring for computing the restoring force (restoring spring), which models the elasticity of elements. The restoring spring is set between continuous elements. Structural failure is modeled as breakage of the restoring spring, at which time the restoring spring is replaced with a contact spring and a contact dashpot (Fig. 1 (b)). Fig. 1 (b) shows the spring and dashpot for computing the contact force (contact spring and dashpot) and modeling the contact, separation, and recontact between elements. The dashpots are introduced to express energy dissipation due to the contact. Structural collapse behavior is obtained using these springs and dashpots. The elements shown in Figs. 1 (a) and (b) are rectangular parallelepipeds, but the method does not limit the geometry of the elements.

The surface of an element is divided into small segments as shown in Fig. 1 (c). The segment in the figure is rectangular, but the method does not limit the geometry of the segment. The black points indicate the representative point of each segment, and the relative displacement or contact displacement between elements is computed for these points. Such points are referred to as contact points or master points in this study. One restoring spring and one combination of contact spring and dashpot are attached to one segment (Fig. 1 (d)) at each of the representative points in Fig. 1 (c). The spring constant for each segment is derived on the basis of the stress-strain relationship of the material and the segment area.

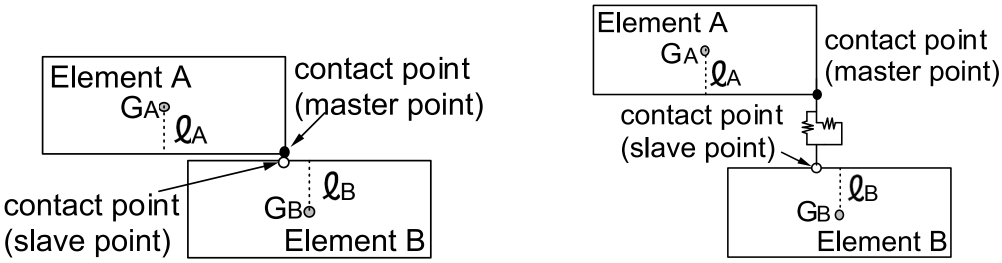
Forces acting on each element are obtained by summing the restoring force, contact force, and other external forces such as the gravitational force and inertial force of an earthquake. The behavior of an element consists of the translational behavior of the center of gravity and the rotational behavior around the center of gravity. The translational and rotational behaviors of each element are computed explicitly by solving Newton's law of motion and Euler's equation of motion.



(a) Restoring spring (b) Contact spring and dashpot

(c) Segments and contact points (d) Multiple springs and multiple dashpots

Fig. 1 Basic concept of the analysis method



(a) Continuous elements (b) Restoring spring between elements

Fig. 2 Modeling of the restoring spring between elements

b) Spring constant of each element

There are two types of springs, namely restoring and contact springs. It is assumed that the spring constants of the restoring spring and those of the contact springs are the same. It is considered that each segment has its own spring.

Springs are set for both the normal and shear (tangential) directions of the surface. Let us denote the area of the segment as dA and the relative (contact) displacement at the surface segment as u_n and u_s. The subscripts n an s indicate the values in the normal and shear directions, respectively. The spring constants per area in the normal and shear directions, k_n and k_s, are obtained as follows.

$$k_n = \frac{E}{(1-\nu^2)\ell}, \qquad k_s = \frac{E}{2(1+\nu)\ell}, \tag{1}$$

where E is Young's modulus, ν is Poisson's ratio, and ℓ is the distance from the surface at which the spring is connected to the center of gravity.

c) Modeling of elastic behavior

It is assumed that two elements, A and B, are continuous, and that a contact point of element A is continuous with element B as shown in Fig. 2(a). The contact point of element A is the "master point" and the contact point of element B is the "slave point." The slave point does not have to be the center of the segment. The combinations of continuous elements for each segment are determined in the initial stage of the computation.

Let G_A and G_B be the centers of gravity of elements A and B, respectively. Let ℓ_A be the distance from G_A to the surface of element A in contact. Let ℓ_B be the distance from G_B to the surface of element B in contact. Let E_A and E_B be Young's moduli and ν_A and ν_B be Poisson's ratios of elements A and B.

It is assumed that both elements have their own spring constants in the normal and shear directions based on their material properties as shown in Fig. 1(b). The spring constants per area for each element are obtained from Eq. (1). Assuming that these springs are connected in series, the spring constants between elements per area, $\bar{k}_n$ and $\bar{k}_s$, are shown in Fig. 2(b).

$$\bar{k}_n = \frac{1}{\dfrac{\ell_A}{E_A/(1-\nu_A^2)} + \dfrac{\ell_B}{E_B/(1-\nu_B^2)}}, \quad \bar{k}_s = \frac{1}{\dfrac{\ell_A}{E_A/2(1+\nu_A)} + \dfrac{\ell_B}{E_B/2(1+\nu_B)}}. \tag{2}$$

The spring constant between elements connected by mortar is also obtained in a similar manner. For example, in masonry structures, bricks are often connected with mortar. In this case, the spring constant per area between elements (bricks) is obtained as

$$\bar{k}_n = \frac{1}{\dfrac{\ell_A - t_M/2}{E_A/(1-\nu_A^2)} + \dfrac{t_M}{E_M/(1-\nu_M^2)} + \dfrac{\ell_B - t_M/2}{E_B/(1-\nu_B^2)}},$$

$$\bar{k}_s = \frac{1}{\dfrac{\ell_A - t_M/2}{E_A/2(1+\nu_A)} + \dfrac{t_M}{E_M/2(1+\nu_M)} + \dfrac{\ell_B - t_M/2}{E_B/2(1+\nu_B)}}, \tag{3}$$

where t_M is the mortar thickness, E_M is Young's modulus, and ν_M is Poisson's ratio of the mortar. The normal direction of forces is the direction perpendicular to the surface of the master point of element A.

Let σ and τ be the normal and shear stresses acting at the contact point, and let u_n and u_s be the relative displacements between the adjacent master and slave points in the normal and shear directions. The relation between traction (σ, τ) and relative displacement (u_n, u_s) is then written as

$$\sigma = \bar{k}_n u_n, \quad \tau = \bar{k}_s u_s. \tag{4}$$

It is noted that the method cannot handle Poisson's effect since it considers the contact between two elements.

d) Modeling of failure phenomena

The elastic behavior of structures is demonstrated by the linear multiple restoring springs between continuous elements until the restoring force of a spring reaches its elastic limit. The elastic limits are modeled using the criteria of tension, shear, and compression failure. When a spring reaches one of these limits, it is judged that failure has occurred at that segment of the spring.

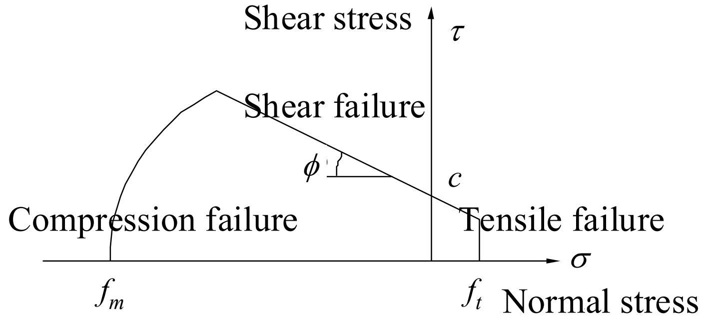


Fig. 3 Modeling of failure phenomena

After the failure, the restoring spring is replaced with a contact spring and dashpot at this segment. The method can trace the expansion of failure between elements. The three failure modes—namely, tension, shear, and compression failure modes—are defined based on the Mohr-Coulomb cap model as follows.

Tension failure mode

For the tension failure mode, the parameter considered is tensile strength f_t. When the normal stress of spring σ exceeds the tensile strength, the restoring spring is assumed to be broken by the tension failure. The yield function has the following form (Fig. 3).

$$f_1(\sigma) = \sigma - f_t \,. \tag{5}$$

The normal restoring stress cannot exceed this limit.

Shear failure mode

For the shear failure mode, the Coulomb friction envelope is used. The parameters considered are bond strength c and friction angle ϕ. The yield function has the following form (Fig. 3).

$$f_2(\sigma) = |\tau| + \sigma \tan\phi - c \,. \tag{6}$$

The shear restoring stress cannot exceed this limit.

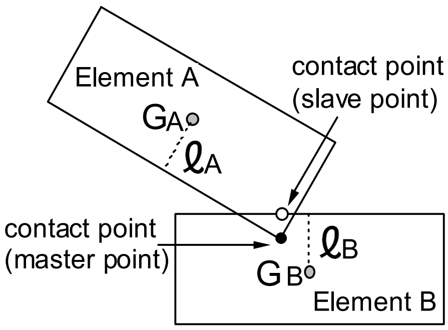
Compression failure mode

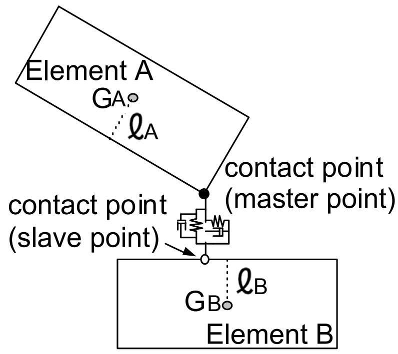
For the compression mode, an ellipsoid cap model is used. The yield function has the following form (Fig. 3).

$$f_3(\sigma) = \sigma^2 + C_s\tau^2 - f_m{}^2 \,, \tag{7}$$

where f_m is the compressive strength and Cs is the material model parameter. $Cs = 9$ is adopted on the basis of past research[2)]. When the restoring stress exceeds this limit, both the normal and shear restoring stresses are reduced in the same proportion to meet this limit.

In real structures, failure occurs not only at the mortar, but also inside bricks and at the interface between brick and mortar. However, in this study, it is assumed that failure does not occur inside bricks but at the interface between bricks, and we do not discriminate failure at the mortar from failure





(a) Discontinuous elements in contact (b) Contact spring and dashpot between elements

Fig. 4 Modeling of the contact spring and dashpot between elements

at the interface between brick and mortar. The restoring spring between elements is linear until any failure occurs. In tension, linear tensile reaction force acts until it reaches the tensile strength. After tensile failure, no tensile reaction force acts between these elements until they are in contact again. In compression, the linear compressive reaction force acts until it reaches the compression strength, and the compressive reaction force then obeys the Mohr-Coulomb cap model. During the unloading process after the failure, the compressive strength decreases linearly with the initial stiffness, and it becomes 0 when elements are separated. The shear reaction force is also linear until it reaches the shear strength, and it then obeys the Mohr-Coulomb cap model. During the unloading process after the failure, the shear strength decreases linearly with the initial stiffness and obeys the limit of Eq. (6). Since the face of the elements is divided into many segments and each segment has its own spring, these springs gradually break during loading.

e) Modeling of contact and recontact between elements

If a segment of an element is in contact with another element with which the segment is not continuous via the restoring spring, the contact spring and dashpot generate contact force between the elements. Contact between a segment and the surface of another element is detected at each time step for all segments that are not continuous with other elements via a restoring spring.

Figure 4(a) illustrates the situation where a contact point of element A is in contact with discontinuous element B, and Fig. 4(b) shows the contact spring and dashpots between these elements.

The spring constant and the contact forces in the normal and shear directions are calculated in the same manner as for the restoring force. The differences from the case for the restoring force are that the contact force is generated only while the compression force acts and that the shear force is bounded by the friction limit.

$$\tau = \sigma \tan \phi , \tag{8}$$

where ϕ is the friction angle.

The dashpot is introduced to express the energy dissipation of the contact. The damping coefficient per area is calculated as follows.

$$c_n = 2h_n \sqrt{m_{ave} k_n} , \qquad c_s = 2h_s \sqrt{m_{ave} k_s} , \tag{9}$$

where h_n and h_s are the damping constants for the normal and shear directions. m_{ave} is the equivalent mass per area relevant to this contact. In this study, m_{ave} is calculated as

$$m_{ave} = \rho_A \ell_A + \rho_B \ell_B, \tag{10}$$

where ρ_A and ρ_B are the mass densities of elements A and B.

The damping constants should be evaluated according to the properties of the elements, but this study adopts critical damping ($h_n = h_s = 1.0$) by considering the fact that most structural components tend not to bounce greatly and that their oscillation tends to quickly disappear when they collide with each other.

f) Restoring and contact forces acting at each element

The restoring and contact forces are calculated at all segments for both elements A and B. The restoring and contact forces acting at each point are obtained by considering the area of the master point. Assuming dA is the area of the master point, the effective area can be half the segment area dA to avoid double counting of the contact force, since the contact points of both elements A and B can be the master points. Therefore, assuming that $dA/2$ is the effective area of the master point, the spring constants and damping coefficients are

$$K_n = \bar{k}_n dA/2, \quad K_s = \bar{k}_s dA/2, \quad C_n = c_n dA/2, \quad C_s = c_s dA/2. \tag{11}$$

In consideration of this, the size of the segments should be small and the same for elements A and B.

Finally, the spring forces ($\mathbf{e_n}$, $\mathbf{e_s}$) and damping forces ($\mathbf{d_n}$, $\mathbf{d_s}$) in the normal and shear directions are written as

$$\mathbf{e}_n = K_n \mathbf{u}_n, \quad \mathbf{e}_s = K_s \mathbf{u}_s, \quad \mathbf{d}_n = C_n \Delta \mathbf{u}_n / \Delta t, \quad \mathbf{d}_s = C_s \Delta \mathbf{u}_s / \Delta t, \tag{12}$$

where $\Delta\mathbf{u_n}$ and $\Delta\mathbf{u_s}$ are the increments of relative displacement and Δt is the time interval. The variables written in bold face are vectors.

g) Structural damping

In this study, structural damping that expresses energy dissipation in structural vibration is considered. This differs from energy dissipation of the contact expressed by damping constants c_n and c_s.

Damping constant c is introduced as mass-proportional damping as

$$c = \alpha m, \tag{13}$$

where α is a variable defining the damping constant. α can be calculated as

$$\alpha = 2 h_i \omega_i, \tag{14}$$

where ω_i is the angular frequency and h_i is the corresponding damping constant.

Mass-proportional damping is adopted since the method solves the equation of motion for each element and mass-proportional damping is easily applicable. The dashpot attached between contact elements is a stiffness-proportional damping. We plan to investigate an appropriate model of damping in a future study.

h) Equations of motion

Equations of motion can be constructed using the restoring and contact forces and other external forces. The motion of each element is obtained by solving the two equations of motion. One is the equation for the translational motion of the center of gravity, and the other is the equation for the rotational motion around the center of gravity.

Translational motion of the center of gravity

The forces acting on an element are the sum of external forces, such as the gravitational force and inertial force due to an earthquake, and the restoring and contact forces between elements. The equation of motion for the translational motion is

$$m\ddot{\mathbf{x}}_g(t) + c\dot{\mathbf{x}}_g(t) = m\mathbf{g} - m\ddot{\mathbf{z}}(t) + \sum \mathbf{F}(t), \tag{15}$$

where $\mathbf{x_g}(t)$ is the displacement vector of the center of gravity of an element at time t, m is the mass of the element, c is the damping constant of the element, $\mathbf{g}$ is the gravitational acceleration vector, $\ddot{\mathbf{z}}_\mathbf{t}$ is the ground acceleration vector at time t, and $\sum\mathbf{F}(t)$ is the sum of the restoring and contact force vectors at time t.

Rotational motion around the center of gravity

First, the angular velocity vector $\boldsymbol{\omega}(t)$ is obtained by solving the following Euler equation of motion.

$$\mathbf{I}\dot{\boldsymbol{\omega}}(t) + \boldsymbol{\omega}(t) \times \mathbf{I}\boldsymbol{\omega}(t) = \sum \mathbf{R}(t)\mathbf{r}(t) \times \mathbf{R}(t)\mathbf{F}(t). \tag{16}$$

Here, $\mathbf{I}$ is the tensor of the moment of inertia, $\mathbf{r}(t)$ is the vector between the center of gravity and the point where force $\mathbf{F}(t)$ is applied. $\mathbf{R}(t)$ is the matrix representing the transformation from the absolute coordinate system to the inertial frame of reference.

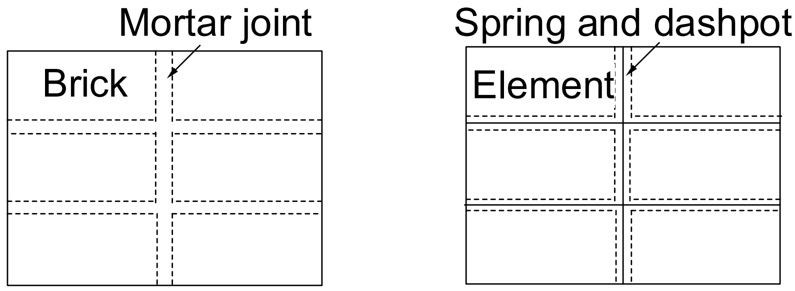
The vector from center of gravity $\mathbf{x_g}(t)$ to arbitrary point $\mathbf{x_p}(t)$ in an element, $\mathbf{x_{gp}}(t)$, is obtained by solving the following differential equation using the angular velocity vector.

$$\dot{\mathbf{x}}_{gp}(t) = (\mathbf{R}^T(t)\boldsymbol{\omega}(t)) \times \mathbf{x}_{gp}(t), \tag{17}$$

where $\boldsymbol{\omega}(t)$ is the angular velocity vector in the inertial frame of reference obtained by solving Eq. (16). The coordinates of point $\mathbf{x_p}(t)$ are then obtained as follows.

$$\mathbf{x}_p(t) = \mathbf{x}_g(t) + \mathbf{x}_{gp}(t). \tag{18}$$

In this study, Eqs. (15)–(18) are solved explicitly using the central difference scheme.



(a) Masonry structure (b) Analytical model

Fig. 5 Analytical modeling of masonry structures

i) Critical time interval

Since the equations of motion are solved explicitly, the solution is conditionally stable[3)]. The following inequality is used to determine the time interval for the computation.

$$\Delta t \leq \min\left(\ell / \sqrt{\frac{E/(1-\nu^2)}{\rho}} \right). \tag{19}$$

The term on the right hand side of Eq. (19) is obtained for all segments of all elements, and the time interval is determined. If the solution is unstable using this time step, a smaller value is used until stability is attained.

j) Modeling of bricks and mortar

In this study, individual components of the masonry structure shown in Fig. 5 (a) (i.e., brick and mortar joints) are modeled in a simple manner as shown in Fig. 5 (b). The bricks are modeled with rigid elements and the mortar joint between elements is modeled with multiple springs and multiple dashpots. The size of one element is the sum of the brick size and the thickness of mortar together. The multiple springs and multiple dashpots interact with the surfaces of adjacent elements.

The modeling in this study is three-dimensional, and the elements are modeled with rigid rectangular parallelepipeds and hexahedrons. Faces surrounding the elements are divided into segments. The interval between contact points of neighboring segments is set to a quarter of the shortest edge length according to a past study[1)]. The method was verified through a comparison with the experimental results of monotonic loading and the analytical results of free vibration by the FEM[1)]. Verification by comparison with cyclic loading and shaking table tests is very important and comprises one of our future plans. The dynamic failure patterns and scattering of elements, which will be shown later, are numerical solutions based on the assumptions shown in the previous section. Moreover, adobe brick is a very brittle material compared to other masonry elements such as burnt brick, stone, and concrete block, and the adobe bricks themselves fracture during earthquakes. However, due to the limitation of the analytical method, we do not model failure inside such bricks. Therefore, the solution focuses on structures where mortar failure is dominant and brick failure is negligible. In this respect, the method may not be appropriate for structures where failure inside the bricks occurs with high probability. Modeling of failure of the brick itself is also one of our future plans.

Fig. 6 Target area (Jhatapo area) and the location of the target building

(a) Western wall (b) Eastern wall (c) Eastern room of the 2nd floor

Fig.7 Target historic masonry building

(3)Analysis outline

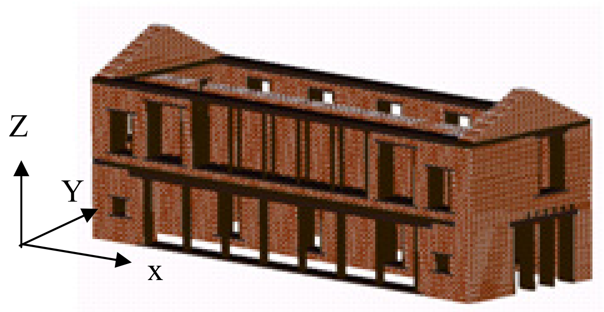
a) Target historic masonry building

A building to be analyzed is a historic composite building whose location and pictures are shown in Figs. 6 and 7. It was built in the seventeenth century, but it has been damaged by many earthquakes, and repaired many times.

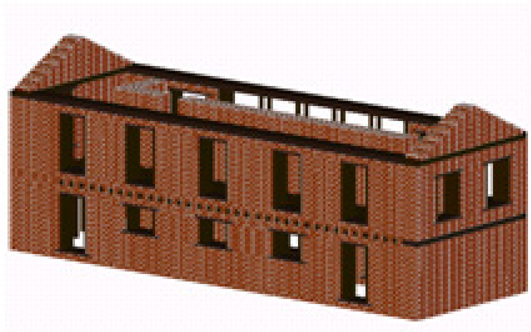
The building is two-storied, and has the dimensions of 16.5m × 5.6m. The height of the 1st and 2nd floors is 2.4m and 2.1m, respectively. The maximum height is 6.5m. Each wall has openings, and the western wall has the largest openings as shown in Fig.7(a). The walls are composed of bricks, and the bricks are connected with each other by mortar. The size of bricks is not unified and depends on when and where they were made. There are two separated rooms with the width of 2-3m as shown in Fig.7(c).

b) Analytical model

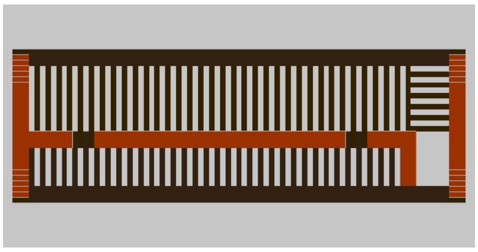
An analytical model is shown in Fig. 8(a) (b) (c). Members except bricks, such as columns, beams and ring beams, are shown in Fig. 8(d). The axis of the coordinate is shown in Fig. 8(a). In the western wall, there are eight wooden vertical columns in the 1st floor, and 6 wooden vertical columns and 2 wooden horizontal beams on the 2nd floor. Moreover, there are two ring beams on the top of the 1st and 2nd floors. The depth of the walls is 55cm.



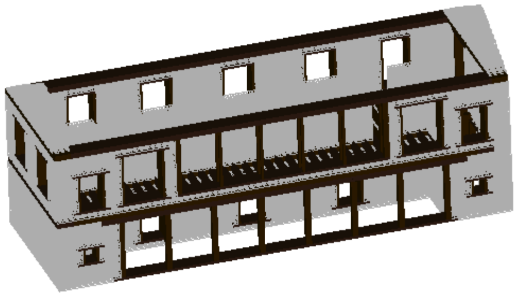
(a) View from the northwest



(b)View from the northeast



(c) View from the top



(d)Columns and beams

Fig. 8 Analytical model

Table 1 Analytical parameter

Variable	Adobe Brick	Mortar	Wood
Mass density (kg/m^3)	1.8×10^3	-	7.0×10^2
Young's modulus (N/m^2)	2.7×10^8	2.7×10^8	6.3×10^8
Poisson's ratio	0.11	0.25	0.3
Tensile strength f_t (N/m^2)	-	0.0	1.1×10^8
Shear strength c (N/m^2)	-	9.0×10^4	9.0×10^6
Friction angle ϕ	-	42.5°	0°
Compressive strength (N/m^2)	-	1.58×10^6	4.5×10^7

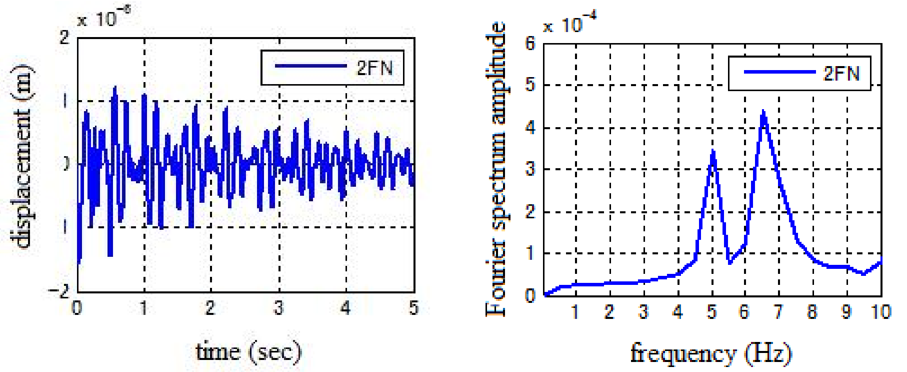
Material properties, such as mass density, Young's modulus and poisson's ratio are listed in Table 1. The values used for bricks are not the actual values of a single brick, but averaged values of the wall including bricks and mortar. The properties are estimated through an experiment using an bricks taken from an existing building. Since the material properties of wooden columns and beams on the western wall were not measured, a general values for woods are used instead. The roof is a tin roof and not modeled considering that its weight is very light and it does not affect the vibration behavior.

The building is modeled as an assembly of rigid elements of rectangular parallelepipeds. The size of each brick is 10cm x 10cm x 20cm. The total number of elements is 63978. The rigid elements are connected with each other by springs. The rigid element itself does not deform, but the building as a whole shows deformation by overlapping of the rigid elements. When one of the tensile, shear and compression stresses acting at the spring exceeds the strength, the spring is cut and the failure occurrence is expressed.

Table 1 also lists the tensile strength, bonding strength, friction angle and compressive strength.

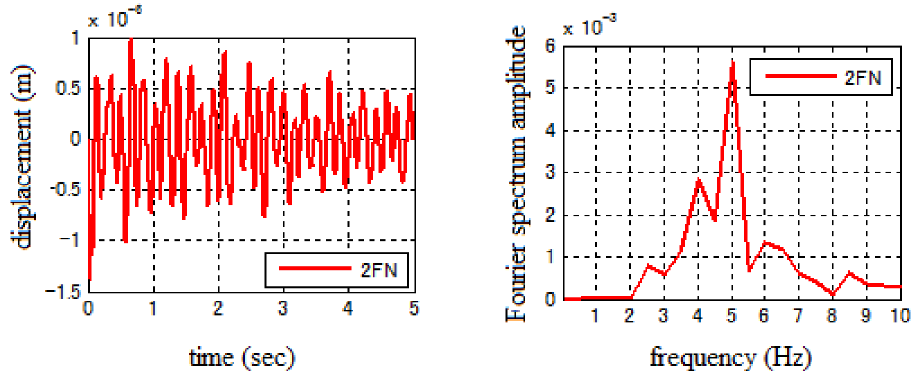
Table 2 Natural frequencies by microtremor observation

Mode number	1st mode	2nd mode	3rd mode
Frequency	4.3Hz	5.8Hz	6.8Hz
Mode	Translational mode in Y direction	Torsional mode	Translational mode in X direction



(a) Time history (b)Fourier spectrum

Fig.9 Dispalacement response in X direction when the impact acceleration is input in X direction



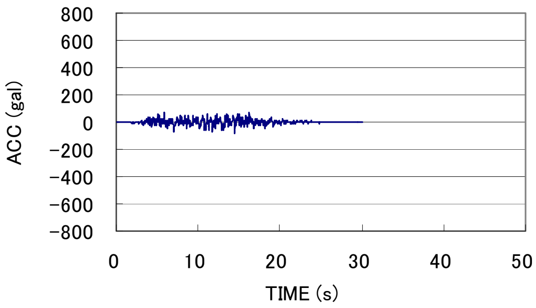
(a) Time history (b)Fourier spectrum

Fig.10 Dispalacement response in Y direction when the impact acceleration is input in Y direction

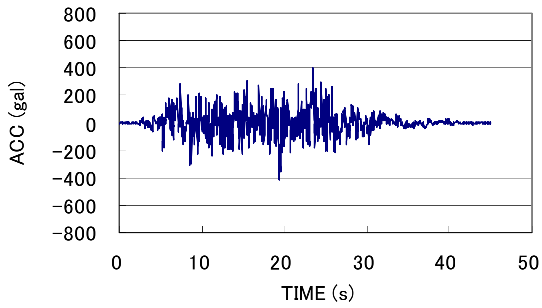
c) Verification of analytical model

The microtremor observation was done for the target building and the natural frequencies was observed as shown in Table 2. The first natural frequency is 4.3Hz and it is a translational mode in Y direction. The second natural frequency is 5.8Hz and it is a torsional mode. The third natural frequency is 6.8Hz and it is a translational mode in x direction.

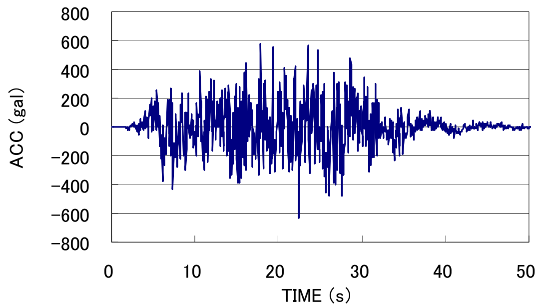
To obtain the natural frequencies of the analytical model, the impact acceleration of 100 gal is input to the building in X and Y directions, separately, and the displacement response is computed at one point at the 2nd floor. The displacement history in X direction when the impact acceleration is input in X direction is shown in Fig.9(a). The Fourier spectrum is also shown in Fig.9(b). The displacement history in Y direction when the impact acceleration is input in Y direction is shown in Fig.10(a). The Fourier spectrum is also shown in Fig.10(b). From these figures, the Fourier spectra for Y direction has its peak around 4.0Hz, so it corresponds to the translational mode in Y direction of the first mode. The Fourier spectra in the both X and Y directions has their peak around 5.0 Hz, and it can be considered to be the torsional mode of the second mode. The Fourier spectrum for X direction has its peak around 6.6Hz and it can be considered to be the translational mode in X direction of the third mode. From this comparison between the analytical results and the microtremor observation, the analytical model expresses the vibration characteristics almost correctly, therefore the validity of the analytical model is confirmed.



(a) 40% (Return period of 98 year)



(b) 10% (Return period of 475 year)



(c) 5% (Return period of 975 year)

Fig. 11 Input ground acceleration with respective occurrence probability in 50 years

Table 3 Cases with different directions and occurrence probabilities of input ground motion

Cases	Direction of input ground motion	Occurrence probability of input ground motion
Case 1X	X	40%
Case 1Y	Y	40%
Case 2X	X	10%
Case 2Y	Y	10%
Case 3X	X	5%
Case 3Y	Y	5%

d) Input ground acceleration

Input ground accelerations are shown in Fig. 11. These are estimated from the Nepalese historic seismic data and active fault data by seismic hazard analysis. Fig. 11(a) is an acceleration with the occurrence probability of 40% in 50 years (return period is 98 year) and has the peak acceleration of 84gal. Fig. 11(b) is an acceleration with the occurrence probability of 10% in 50 years (return period is 475 year) and has the peak acceleration of 420gal. Fig. 11(a) is an acceleration with the occurrence probability of 5% in 50 years (return period is 975 year) and has the peak acceleration of 630gal.

e) Analytical cases

The analytical cases are shown in Table 3. There are 6 cases with 2 different directions and 3 different occurrence probabilities of input ground motion.

(4) Simulation results

The three input ground motions with differente occurrence probabilities are input to the model. The seismic behavior for every 5 second is whosn in Figs. 12-17.

Based on damage investigation surveys, the typical failure patterns of adobe buildings have been discussed by many researchers[4),5)]. Here, the outcome of the DEM is compared with the observed failure patterns to confirm the performance of the DEM.

Mohmoud et al. [4] reported that the out-of-plane failure of walls was the main failure mode of the adobe houses. They describe that lack of proper connections between the perpendicular walls resulted in the separation of walls from each other, failure of the walls, and subsequent collapse of the roof. The DEM succeeded in demonstrating this failure mode. For example, in Fig.13, the out-of-plane deformation of the shorter walls in the south side can be seen. Zahrai and Heidarzadeh[5] pointed out that the failure of adobe buildings normally started in a corner by separation of the walls from the top. The DEM also demonstrates this behavior. For example, from Fig. 13, it is found that the separation between walls starts from the top.

a) Input ground acceleration with the occurrence probability of 40% in 50 years

The input ground motion with the occurrence probability of 40% in 50 years is input in X and Y directions, separately. Seismic behavior for these directions are shown in Figs. 12 and 13.

When we look at the triangle part on the shorter wall, the difference due to the input ground motion directions cannot be seen. And the northern wall completely fell down while the southern wall partially fell down. This is because the northern wall has larger openings at the second floor and this made the northern wall vibrate more largely.

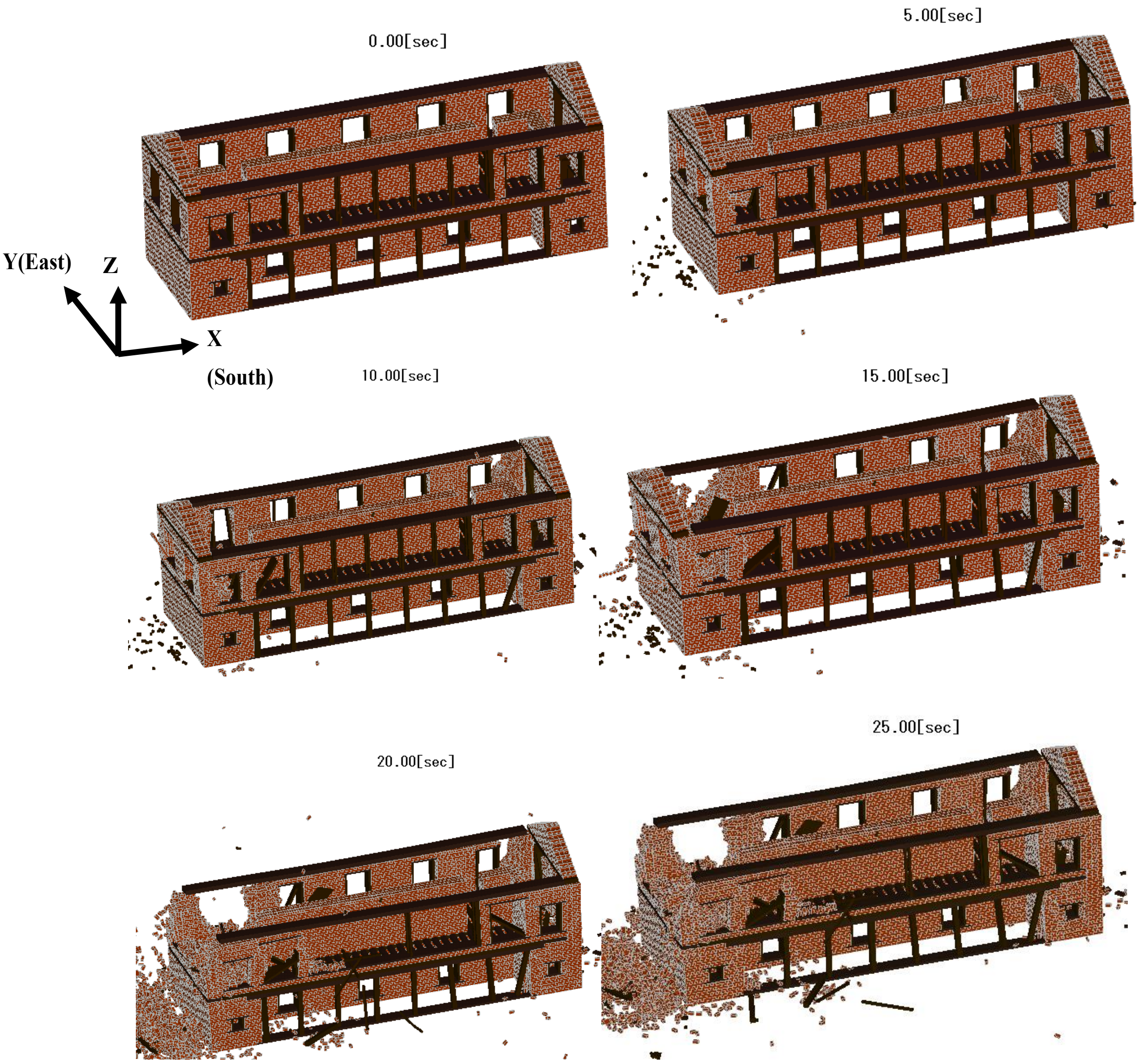


Fig.12 Seismic behavior of Case 1X (input ground motion : X direction, occurrence probability in 50 years : 40%)

When the input ground motion is input in X direction (case1X), more damage occurred to the wider wall compared to the case when the input ground motion is input in Y direction (case1Y).

As for case1X, the northern part of the wall in the east side suffered severe damage. This is because the ground motion in X direction excites the vibration of the shorter wall in the out-of-plane direction. The wall in the north side has larger openings in the 2nd floor, and it is more likely to vibrate compared to the wall in the south side. Therefore, the upper portion of the northern wall separated from the other part and fell down, which caused failure of the eastern wall. The northern wall suffered from the failure due to the vibration in the out-of-plane direction, whereas the southern wall suffered from slight damage. From those observation, it can be said that the southern part with smaller openings in the 2nd floor has more earthquake-resistance compared to the northern wall.

As for case1Y, the both eastern and western walls are wide with larger openings and easy to vibrate in the out-of-plane direction, but the western wall has less damage than the eastern wall. This is because the columns in the western wall acted as the reinforcement and avoided the western wall from failure.

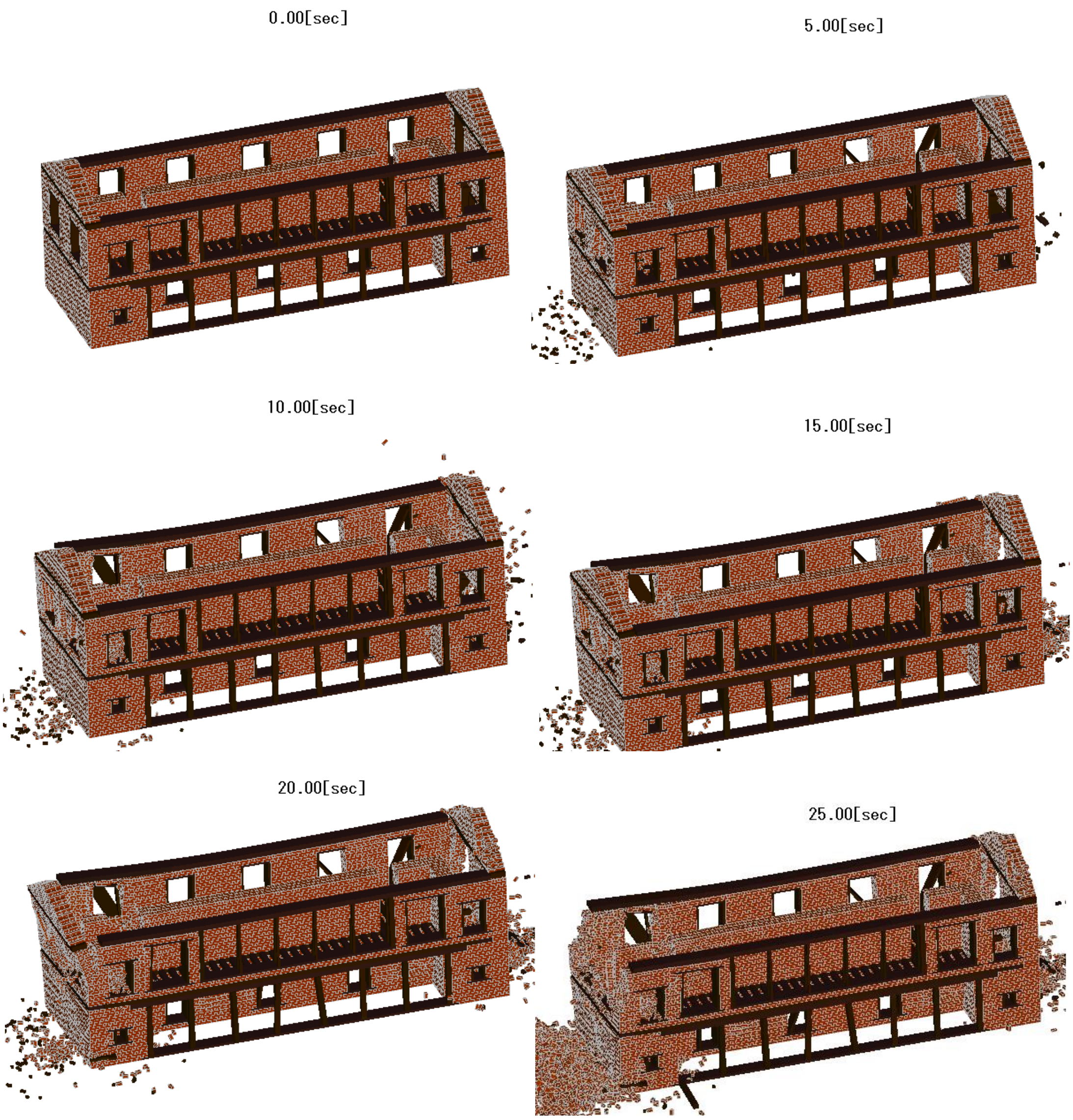


Fig.13 Seismic behavior of Case 1Y(input ground motion : Y direction, occurrence probability in 50 years : 40%)

b) Input ground acceleration with the occurrence probability of 10% in 50 years

The input ground motion with the occurrence probability of 10% in 50 years is input in X and Y directions, separately. Seismic behavior for these directions are shown in Figs.14 and 15.

When the ground motion is input in Y direction, the building completely collapsed. The eastern wall firstly separated from the buildings and collapsed, then the shorter wall collapsed, and the western wall collapsed lastly. The reason why the western wall with the largest openings collapsed lastly is the columns resist against the tensile failure. When the ground motion is input in X direction, the both shorter walls get damaged in the second floor followed by the failure of the 1st floor, and this causd the damage to the southern parts of the eastern and western walls.

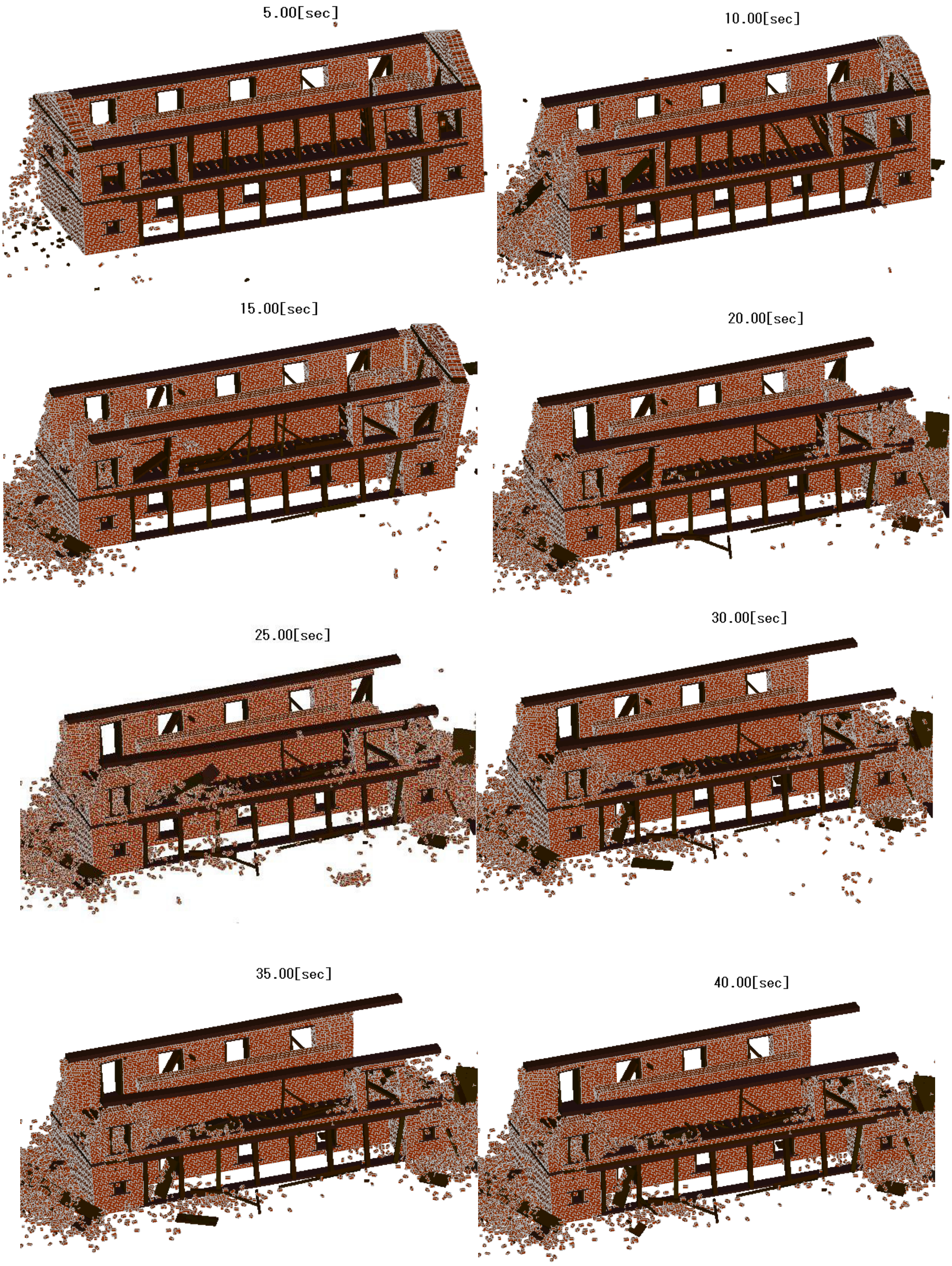


Fig.14 Seismic behavior of Case 2X(input ground motion : X direction, occurrence probability in 50 years : 10%)

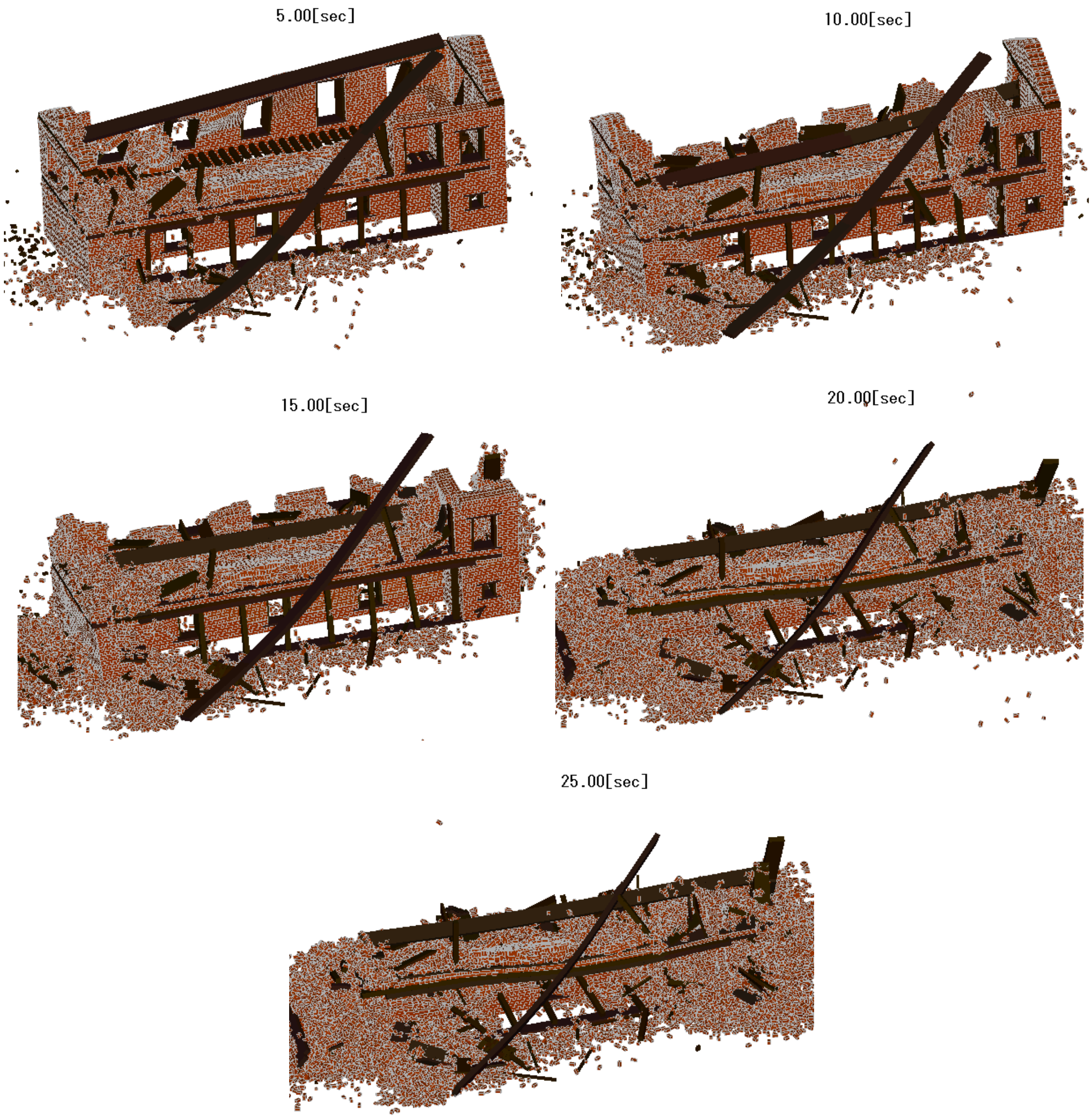


Fig.15 Seismic behavior of Case 2Y
(input ground motion : Y direction, occurrence probability in 50 years : 10%)

c) Input ground acceleration with the occurrence probability of 5% in 50 years

Finally, the input ground motion with the occurrence probability of 5% in 50 years is input in X and Y directions, separately. Seismic behavior for these directions are shown in Figs.16 and 17. Similar to the case with the occurrence probability of 10%, the buildings collapsed when the ground motion is input in Y direction, When the ground motion is input in X direction, the 2nd floor collapsed, but the 1st floor did not collapsed.

(5) Conclusions

The seismic behavior of the existing masonry building is analyzed using the refined version of the DEM. The first three natural frequencies of the analytical model coincide with those obtained from the microtremor observations. When the ground motion with the occurrence probability of 10% and 5% in 50 years is input, severer damage occurred especially when the ground motion is input in Y direction.

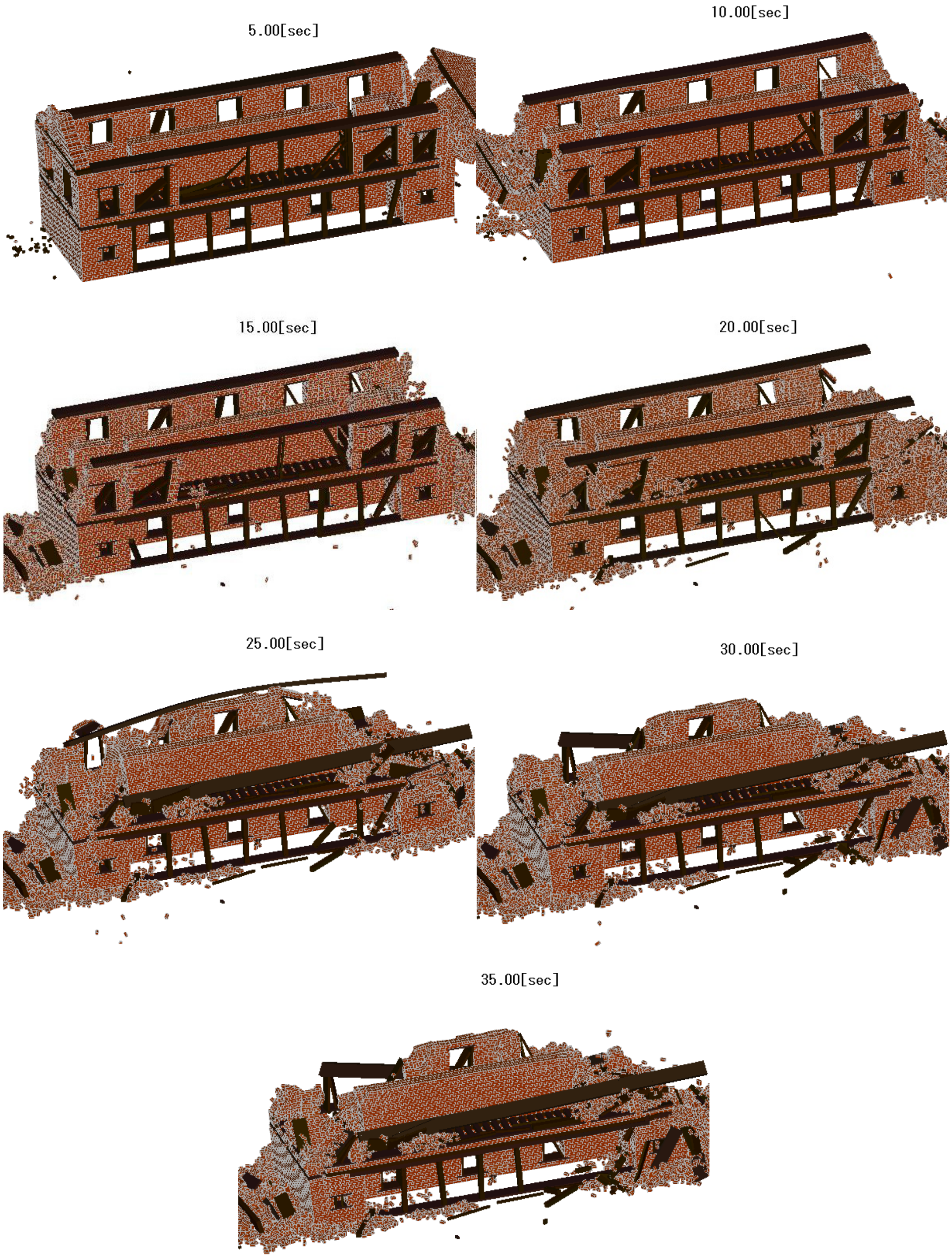


Fig.16　Seismic behavior of Case 3X
(input ground motion : X direction, occurrence probability in 50 years : 5%)

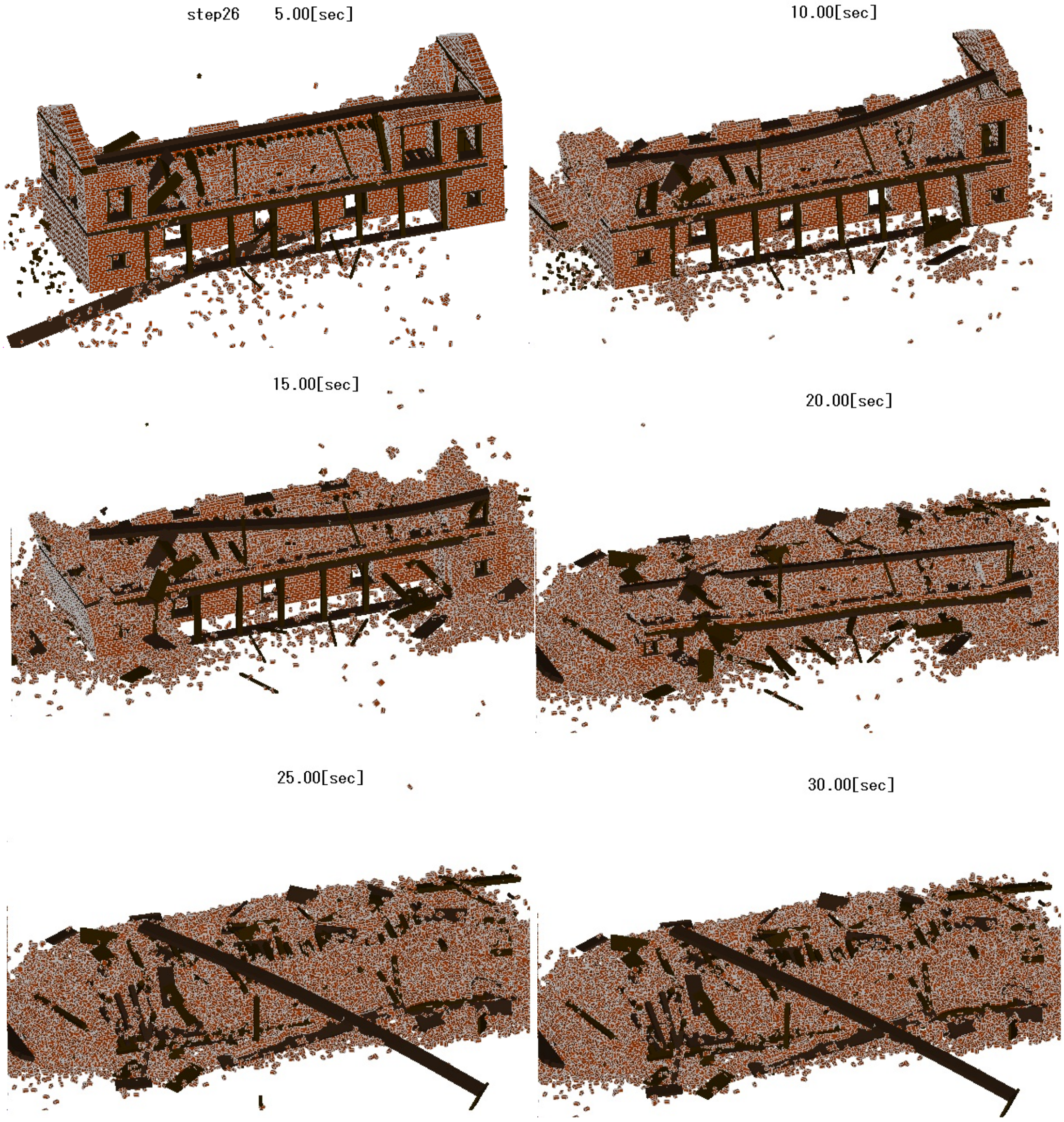


Fig.17 Seismic behavior of Case 3Y
(input ground motion : Y direction, occurrence probability in 50 years : 5%)

References

1) Furukawa, A., Kiyono, J., Toki, K.: Proposal of a numerical simulation method for elastic, failure and collapse behaviors of structures and its application to masonry walls, *Journal of Disaster Research*, Vol.6, No.1, Paper: Dr6-1-4524, 2011.
2) Lourenco, P.B.: Analysis of masonry structures with interface elements, theory and applications, Delft University of Technology, Faculty of Civil Engineering, TU-DELFT report no.03-21-22-0-01, TNO-BOUW report no.94-NW-R0762, 1994.
3) Courant, R.L.:Variational Methods for the solution of problems of equilibrium and vibration. *Bulletin of the American Mathematical Society*, 49:1-23, 1943.
4) Mahmoud, R.M., Farzad, N., Michael, M.: Performance of adobe residential buildings in the 2003 Bam, Iran, Earthquake, *Earthquake Spectra*, Vol.21 No.S1, pp.S337-S344, 2005.
5) Zahrai, S.M., Heidarzadeh, M.:Destructive effects of the 2003 Bam earthquake on structures, *Asian Journal of Civil Engineering (Building and Housing)*, Vol.8, No.3, pp.329-342, 2007.

3-3 Damage Potential of Buildings in Case Study Area

3-3.1 Characteristics of Traditional Brick Masonry Buildings

Hari Ram Parajuli

Out of the three block clusters of the Jhatapol area (Fig. 3-2.1) the focus for the disaster risk management plan is on Block B. The buildings surveyed in Block B is indexed with nomenclature B1, B2, B3 etc. as shown in Fig. 3-2. 2. Block B contains altogether 90 buildings comprising of brick masonry buildings and buildings made of reinforced concrete buildings. Block B contains a total of 76 traditional brick masonry buildings with different characteristics. Details of each of these buildings as a result of the building survey are presented in Appendix B. The traditional brick masonry buildings constitute about 85% of the total buildings in the area. The brick masonry buildings in the study area have the following characteristics:

i. Almost all the buildings have a rectangular plan. Some buildings have foot prints based on combination of more than one rectangle but of different size. However, the plinth area of such buildings being relatively small, the torsional effect due to non-coincidence of the center of mass and center of rigidity is not substantial. Few buildings having more than 3 storeys, modified with addition of one or two storeys, have irregular plan in the added upper storeys, causing horizontal torsional effect as well wise additional vertical bending effect.
ii. The length to breadth ratio in almost all the buildings is 2 or less. This indicates a positive characteristic of the buildings in regard with stability.
iii. The openings for windows and doors in majority of the buildings are small in size and are located symmetrically, and the numbers of the openings are limited. These are positive features towards sustaining the necessary horizontal stiffness of the walls. The large openings for windows in buildings, especially in the buildings with extended storeys, and located without symmetry, reduces the wall stiffness and the horizontal shear area of walls needed to withstand the earthquake force.
iv. The buildings with original configuration, especially buildings with maximum number of storeys not more than 4 storeys, have retained the traditional double framing of the openings, to strengthen the openings. This traditional detailing is missing in the buildings with added floors, which in general, have large openings.
v. The thickness of the structural walls is 230 mm or more in all buildings. The thickness is larger of about 450 mm in the ground floor of the buildings with original configuration. The upper floors have wall thickness limited to 230 mm, which is the minimum required for the longitudinal stability of the walls of length or height up to about 2.5m.
vi. The floor to floor height of all the buildings are less than 2.5 m, crucial for limiting the storey drift of buildings during the lateral action due to earthquake.
vii. The number of storeys ranges from 1 to 6 storeys. The storey-wise distribution of the brick masonry buildings is presented in Table 3-3.1.

Table 3-3.1 Storey wise distribution of brick masonry buildings in Block B

	Number of buildings	% of total	Remarks
Single - storeyed	10	13.16	Originally 1 storeyed
Two - storeyed	0	0.00	No 2 storeyed building exist
Three - storeyed	8	10.53	Originally 3 storeyed
Four - storeyed	31	40.79	8 originally 4 storeyed (25.8%); 23 originally 3 storeyed (74.2%)
Five - storeyed	25	32.89	6 originally 5 storeyed (24%); 12 originally 4 storeyed (48%); 7 originally 3 storeyed (28%);
Six - storeyed	2	2.63	Originally 4 storeyed
Total	76	100.00	

The 4-storeyed buildings are maximum in number constituting 40.79%, of the brick masonry building stock in the area. About 74.2% (23 numbers) of these 4-storeyed buildings evidently used to be originally 3-storeyed, and only 8 numbers of these had been constructed originally as 4-storeyed buildings. Only 32 buildings, constituting about 42% of the building stock have retained the original number of stories. These are 10 numbers of single storyed, 8 numbers of 3-storeyed, 8 numbers of 4 storeyed and 6 numbers of 5 storeyed buildings. Rest buildings are with additional one or two storeys as is clear from Table 3-3.1.

i. Usage wise, about two-third of the buildings is being used for only residential purpose, whereas the remaining one-third is used for residential and commercial purposes.
ii. Most of the brick masonry buildings are observed with blemishes like inclination, cracks in walls and deterioration of materials due to aging. About one-third of the buildings do not show any sign of such damages. The basic reason for the two-third of the buildings having defects and damages is the lack of maintenance and repair.

As is known, the brick masonry buildings have a limited capacity against the earthquake actions. Recognizing the brittle properties of the brick masonry, the optimum seismic resistance of the buildings is achieved best by proper planning, connection details, and workmanship. Extensive use of timber as the structural or non-structural elements in the traditional brick masonry buildings in mud mortar is crucial in development of the load path and necessary redundancy. It has been well recognized that such buildings with low rise and limited unsupported length of structural walls may exhibit adequate resistance to earthquakes. The configuration with horizontal and vertical regularity in terms of mass and stiffness is the most important feature of such buildings, and any deviation from these result into a substantial increase in seismic vulnerability. Of the brick masonry building stock in the study area of Jhatapol, in general, the buildings with number of storeys up to 3 have relatively the best structural characteristics in terms of the original configuration, connection details and load path. However, the current status of these buildings with a lot of defects without repair and restoration makes them more vulnerable to seismicity. On the other hand, the buildings with more than 3 storeys, modified in later stage, including addition of extra storeys to the originally 3-storeyed buildings, are more seismically vulnerable by virtue of larger height, disturbed structural integrity of members, mass and stiffness irregularity, larger openings and lack of clear load path. The seismic and structural characteristics of the buildings, if confined to their original configuration with 3-storeys, and repaired and restored to the original condition by eradicating the defects due to aging and environmental degradation, would have been the best.

3-3.2 Classification of Building Type for Traditional Brick Masonry

Prem Nath Maskey

The traditional brick masonry buildings of the Kathmandu Valley originated and developed in the of Malla (Medieval) Period during 1200 – 1769 A.D., This period is marked with the culmination of the art and architecture in Nepal. The masonry building built in this period has, in general, the structural system based on wall system with a spine wall at the middle of the plan, symmetrical configuration, with small opening ratio and pitched roof covered with local roof tiles (*djhingti*). The original brick masonry building is presented in Fig. 3-3.1.

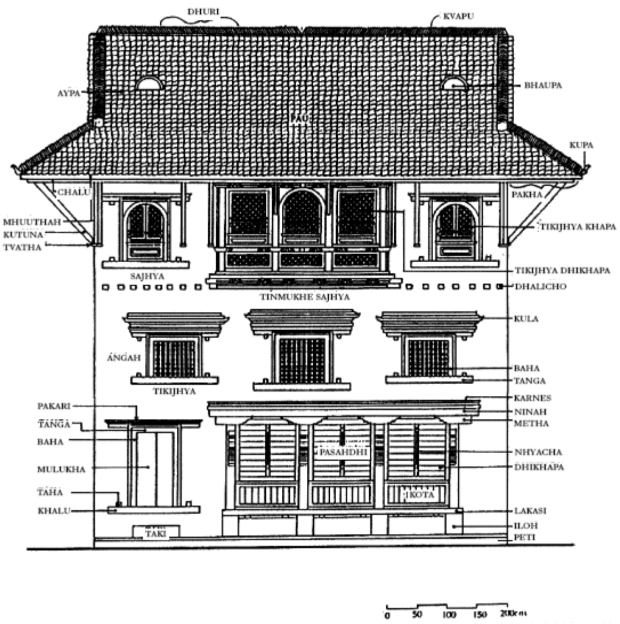


Fig. 3-3.1 Elevation of typical traditional brick masonry building. Source: Drawing: N. Gutschow. Newari towns and buildings.

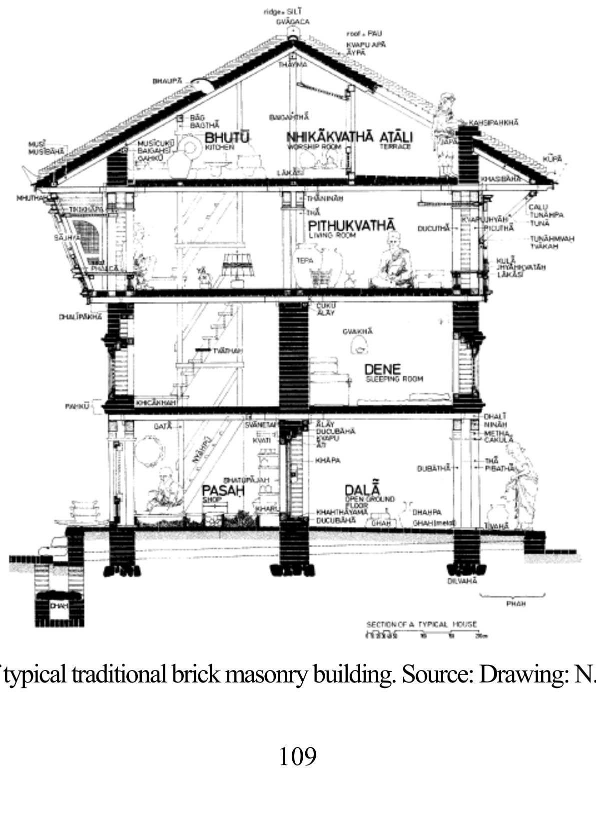


Fig. 3-3.2 Cross section of typical traditional brick masonry building. Source: Drawing: N. Gutschow. Newari towns

The cross section of the typical traditional brick masonry building (Fig. 3-3.2) clearly gives a picture of the structural system, integrity of the structural elements and the load path in such buildings. This style of the buildings was retained with a little change, basically enlarging in proportion, in the later Shah period (1769 – 1846) with a blending of Mughal architecture. The substantial influence of other countries in the brick masonry buildings is observed during the later stage of Rana period (1846 – 1951), marked with imported technology and materials. Based on these developments the brick masonry buildings are classified, in general, by the name of Malla style, Shah style and Rana style. The basic transformation of the style from the Malla period to later periods is shown in Fig. 3-3.3.

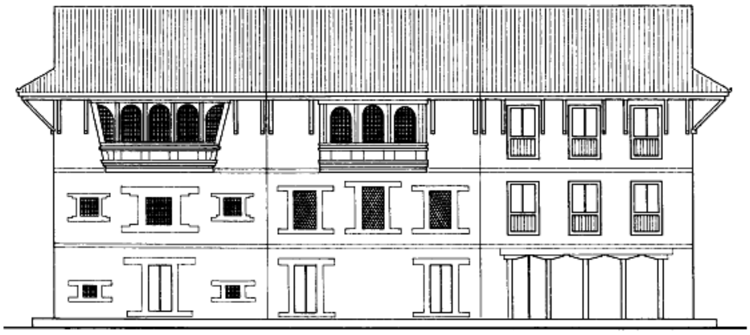


Fig. 3-3.3 Transformation of style of traditional brick masonry building from Malla to Rana period. Source: Drawing: W. Korn. The TraditionalArchitectural of the Kathmandu Valley.

At present, the building stock of the old settlement of Patan, represented by the Jhatapol area is the conglomeration of all these three types of buildings, with or without modifications. The structural system and the general features of traditional brick masonry buildings of some buildings have changed since their construction; these are the buildings which have been raised with additional floors or reconstructed in a later stage. However, the pattern of the settlement, characterized by the integrated location of the buildings, and having the common open spaces, remain unaltered.

The buildings distributed in the area of Jhatapol today, have different levels of seismic vulnerability by virtue of their difference in configuration, size, irregularity, opening size and age of the building. For the purpose of detailed structural analysis of the buildings, the brick masonry buildings have been classified into 6 types based on their configuration, size, area of openings, most importantly on number of storeys. Each category of the buildings is designated by a building index number, and represents a set of buildings, which fall into the category or closer to the category. The classification of the brick masonry buildings based on their structural characteristics to obtain the seismic vulnerability is shown in Table 3-3.2.

Table 3-3.2 Classification of traditional masonry buildings

Class	MB1	MB2	MB3	MB4	MB5	MB6
Number of storeys	3	3	4	4	5	5
Building Representing Class	A75	C25	A48	C26	B47	B4-B5
	A15	A12	A3	A1	A4	A21
	A34	A30	A6	A2	A5	A25
	A37	A40	A7	A18	A13	A54

Buildings Falling under the classes	A47	A45	A9	A19	F53	A56
	A69	A46	A16	A20	A22	A58
	A73	A57	A24	A29	A23	A62
	A77	B17	A27	A38	A26	A76
	B18	B32	A31	A39	A36	A84
	B42	B52	A32	A41	A50	B12
	B88	B65	A33	A44	A51	B24
	C9	B85	A35	A55	A64	B29
		B89	A42	A61	A66	B41
		C0	A43	A71	A74	B50
		C6	A60	A72	A79	B54
		C7	A70	A83	A80	B69
		C10	A78	A85	A81	B77
		C22	B1	B4	A82	C1
		C32	B2	B5	B3	C5
		C55	B6	B8	B15	C30
		C59	B7	B9	B19	C58
		C60	B14	B10	B20	
			B16	B11	B26	
			B33	B13	B28	
			B38	B25	B30	
			B47	B27	B31	
			B57	B49	B39	
			B66	B68	B40	
			B67	B79	B43	
			B75	B80	B45	
			B78	B86	B48	
			B81	B87	B76	
			B82	C56	C2	
			B84		C3	
			C13		C29	
			C17			
			C23			
			C24			
			C26			
			C35			
			C57			
			C61			

In the above classification the single storey buildings are not included because the vulnerability analysis for them is not necessary. The building classes of MB1 and MB2 are having the same number of storeys, indicating that the classification in not based only upon the number of storeys, but also on other parameters, like, size, configuration, openings in the walls and age etc. Similarly the classes of MB3 and MB4 both have 4 storeys, the classes of MB5 and MB6 both are 5 storeyed buildings for the same reasoning.

References

1) Gutschow, N., Kolver, B. and Shresthacharya, I.: *Newar Towns and Buildings: an Illustrated Dictionary Newari - English.* VGH Wissenschaftsverlag Sankt Augustin, Germany. 1987.
2) Korn, W.: *Traditional Architectural of the Kathmandu Valley*. Ratna Pustak Bhandar, Kathmandu. 1979.

.

3-3.4 Characteristics of RC Buildings

Prem Nath Maskey

Out of the three block clusters of the Jhatapol area (fig. 10) the focus for the disaster risk management plan is on Block B. The buildings surveyed in Block B is indexed with nomenclature B1, B2,… as shown in figure 11. Block B contains altogether 90 buildings comprising of brick masonry buildings and buildings made of reinforced concrete buildings. Block B contains a total of 14 buildings constructed in reinforced concrete frame as the structural system. These buildings have different structural characteristics. Details of each of these buildings as a result of the building survey are presented in Appendix C. The traditional brick masonry buildings constitute about 15% of the total buildings in the area. The brick masonry buildings in the study area have the following characteristics:

i. Almost all the buildings have a rectangular plan, or a plan with combination of more than one rectangle. In general, the buildings have irregular plans, presumably the reason for choosing the framed structure in concrete. These buildings are prone to torsional effects when subjected to horizontal or lateral loads, for example, due to earthquakes.
ii. The length to breadth ratio in almost all the buildings is 2 or less. This indicates a positive characteristic of the buildings in regard with stability.
iii. The buildings have large area of openings, as is envisaged in framed structures. The openings for windows and doors are located rather based on the location of columns, and hence not necessarily followed the rule of symmetry. The provision of brick masonry infill walls, especially the external infills of marginal thickness, coupled with the large window openings leads to the weak storey stiffness, indicating the vulnerability feature of the buildings. Many of the buildings have isolated lintels above the openings rather than providing a continuous lintel band, losing the opportunity to tie the masonry walls with the structural frame at the strategic levels.
iv. The buildings, which have not undertaken the vertical expansion, have retained the original configuration. But the buildings with recent addition of extra floors have possess large projections for extra covered space or balcony, resulting into the state of mass and stiffness irregularity. Worse is when walls are constructed supported on projected slab or beam to enclose the space. This is the most undesirable practice in case of low rise buildings constructed in concrete frames.
v. The thickness of the partition walls (infill wall) is limited to not more than 230 mm. The thickness is reduced to 115 mm, particularly in the upper floors, with a notion that these are non structural elements. The provision of such thin walls of brick masonry without reinforcement and anchorage with the supporting structural frame makes the building more vulnerable due to reduction of storey stiffness as well as the reduced out-of-plane capacity of the wall.
vi. The floor to floor height of all the buildings are less than 2.5 m, crucial for limiting the storey drift of buildings during the lateral action due to earthquake.
vii. One of the best features of the buildings constructed with reinforced concrete frame

structural system is the solid concrete slab at each floor level, imparting very high in-plane stiffness. However, more important is the concrete column size, its flexural and shear capacity to resist the inertia force due to earthquake. In most of the buildings the size of the column is limited to 230 mm square. The concrete frame with such size of column will not be able to perform as intended. It will be like the concrete frames are made to tie the brick masonry walls vertically and horizontally, undermining the role of the principal structural system. Providing maximum reinforcements in beams and slabs, and providing minimum reinforcements (in most cases limited to 4 numbers of 16 mm dia.) have been the usual practice, as a result the columns are weaker and the building hence may not be able to resist the horizontal loads effectively.

viii. Most of the buildings have no engineering design, and constructed following the rules of the thumb, and relying on the traditional masons, who are highly skilled in traditional brick masonry buildings, but not in reinforced concrete construction. As a consequence, the buildings, on one hand do not comply with the prevailing codes of practice on concrete construction, and on the other, the technology of construction is applied not with the needed details of construction.

ix. The number of storeys ranges from 1 to 5 storeys. The storey-wise distribution of the reinforced concrete buildings is presented in Table 10.

Table 3-3.3 Storey wise distribution of reinforced concrete buildings in Block B

	Number of buildings	% of total	Remarks
Single - storeyed	2	14.29	Originally 1 storeyed
Two - storeyed	0	0.00	No 2 storeyed building exist
Three - storeyed	1	7.14	Originally 3 storeyed
Four - storeyed	3	21.43	2 originally 4 storeyed (66.7%); 1 originally 3 storeyed (33.3%)
Five - storeyed	8	57.14	4 originally 5 storeyed (50%); 1 originally 4 storeyed (12.5%); 1 originally 3 storeyed (12.5%); 2 originally 2 storeyed (25%)
Total	14	100.00	

The 5-storeyed buildings are maximum in number constituting 57.14%, of the reinforced concrete building stock in the area. About 50% (4 numbers) of these 5-storeyed buildings evidently used to be originally 5-storeyed, that is, without any extension; about one fourth (2 numbers) of the 5-storeyed buildings was originally constructed as 2-storeyed, and 1number (12.5%) used to be originally 3-storeyed and another (12.5%) 4-storeyed structures. Only 9 buildings, constituting about 65% of the building stock have retained the original number of stories. These are 2 numbers of single storyed, 1 number of 3-storeyed, 2 numbers of 4 storeyed and 4 numbers of 5 storeyed buildings. Rest buildings are with additional one or two storeys as is clear from Table 10.

i. Usage wise, about 71% of the buildings is being used for only residential purpose, and about 15% of the building stock is used for residential and commercial purposes, whereas the remaining 14% is used for other usage like monument or store.

ii. Most of the buildings (about 93%) are observed without any kind of damage, where as one building (about 7%) is observed with a lot of defects and damages like inclination, joint damage and lack of mortar.

As is well recognized, the structural performance of framed concrete buildings is better than that of brick masonry buildings, and hence the framed buildings generally have a better resisting capacity against the earthquake actions. However, as the reinforced concrete construction is a sophisticated technology, it should be designed and constructed according to the accepted standards and codes of practice. Numerous factors influencing the concrete strength may not be known to common local masons, the development of required ductility in the structure by proper rebar details may be even far from their experience. The buildings constructed with RC cannot be efficiently built in non-engineering approach. Further, irrespective of better strength of the RC framed structure, the configuration with horizontal and vertical regularity in terms of mass and stiffness is still the most important feature of the buildings, and any deviation from these result into a substantial increase in seismic vulnerability. The concrete framed buildings without extension of floors, in general, possess relatively the best structural characteristics in terms of the original configuration, connection details and load path. On the other hand, the buildings with 5 storeys, modified in later stage, including addition of extra storeys to the originally 2 to 4 -storeyed buildings, are more seismically vulnerable by virtue of larger height, disturbed structural integrity of members, mass and stiffness irregularity, larger openings and lack of clear load path.

3-3.5 Development of Seismic Risk Evaluation method

Prem Nath Maskey

(1) Classification of Reinforced Concrete Buildings

The reinforced concrete buildings of the area are relatively new, because these have been constructed substantially later. However, these buildings distributed in the area of Jhatapol today, have different levels of seismic vulnerability by virtue of their difference in configuration, size, irregularity, opening size and age of the building. For the purpose of detailed structural analysis of the buildings, the RC buildings have been classified into 6 types based on their configuration, size, area of openings, most importantly on number of storeys. The six classes of the buildings are designated by CB1to CB6, each one of which represents a set of buildings, which fall into the category or closer to the category. The plans of the reinforced concrete buildings representing the classification are shown in Figs. 3-3.4 – 3-3.9. The classification of the reinforced concrete buildings based on their structural characteristics to obtain the seismic vulnerability is shown in Table 3-3.4.

In the above classification the single storey buildings are not included because the vulnerability analysis for them is not necessary. The building classes of MB1 and MB2 are having the same number of storeys, indicating that the classification in not based only upon the number of storeys, but also on other parameters, like, size, configuration, openings in the walls and age etc. Similarly the classes of MB3 and MB4 both have 4 storeys, the classes of MB5 and MB6 both are 5 storeyed buildings for the same reasoning

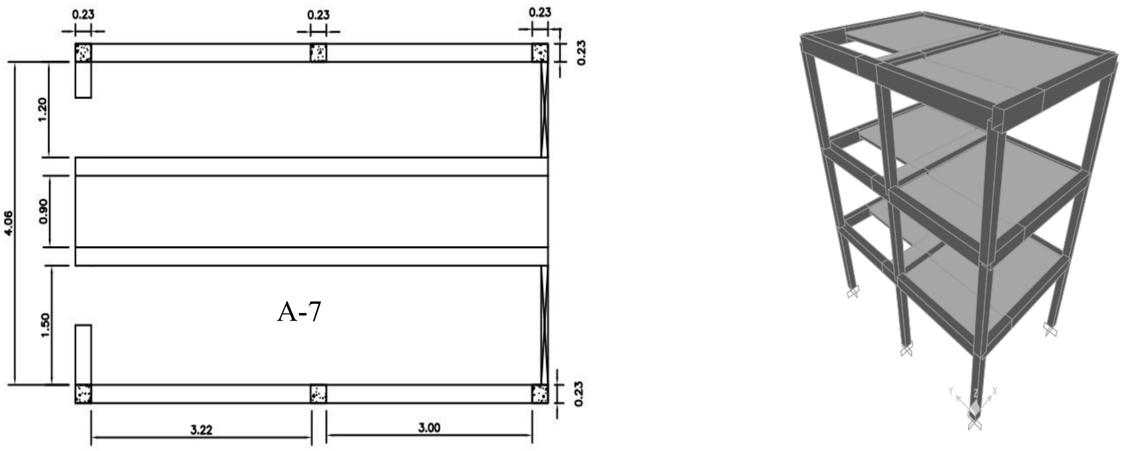


Fig. 3-3.4 Plan and isometric view of the building of class CB1

Fig. 3-3.5 Plan and isometric view of the building of class CB2

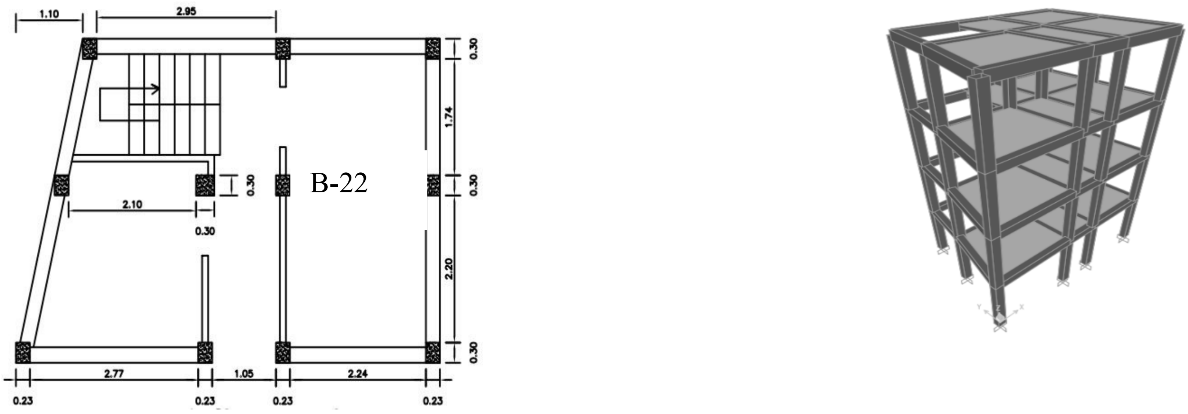


Fig. 3-3.6 Plan and isometric view of the building of class B3

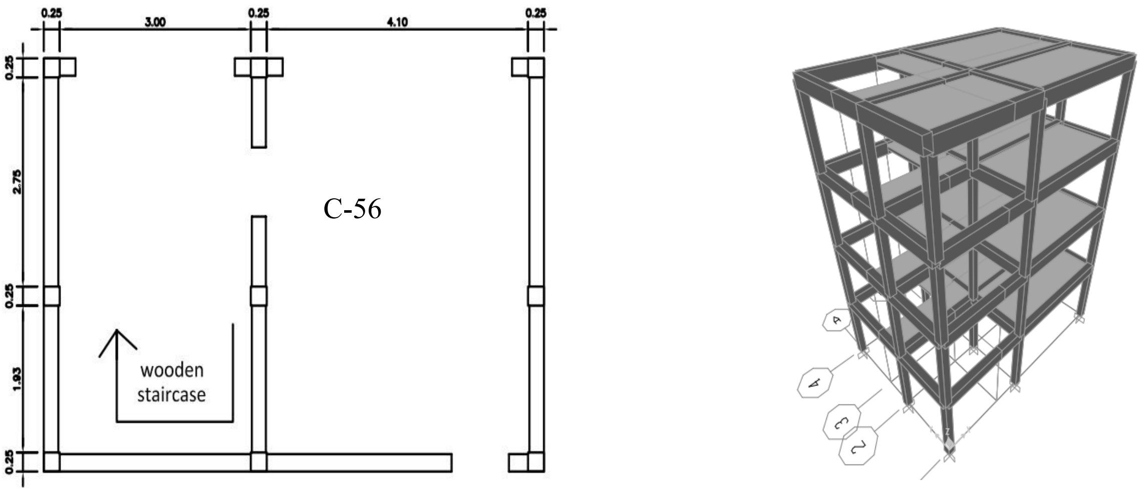


Fig. 3-3.7 Plan and isometric view of the building of class CB4

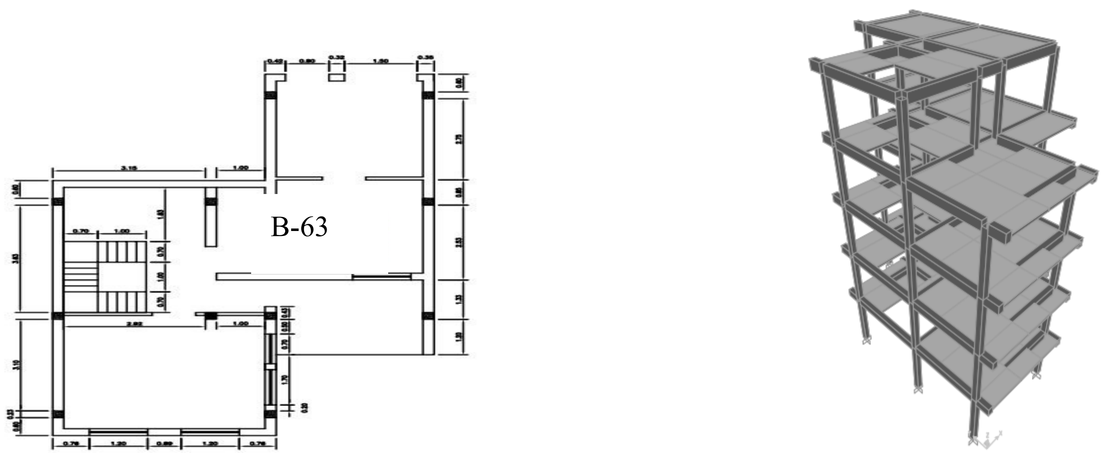


Fig. 3-3.8 Plan and isometric view of the building of class CB5

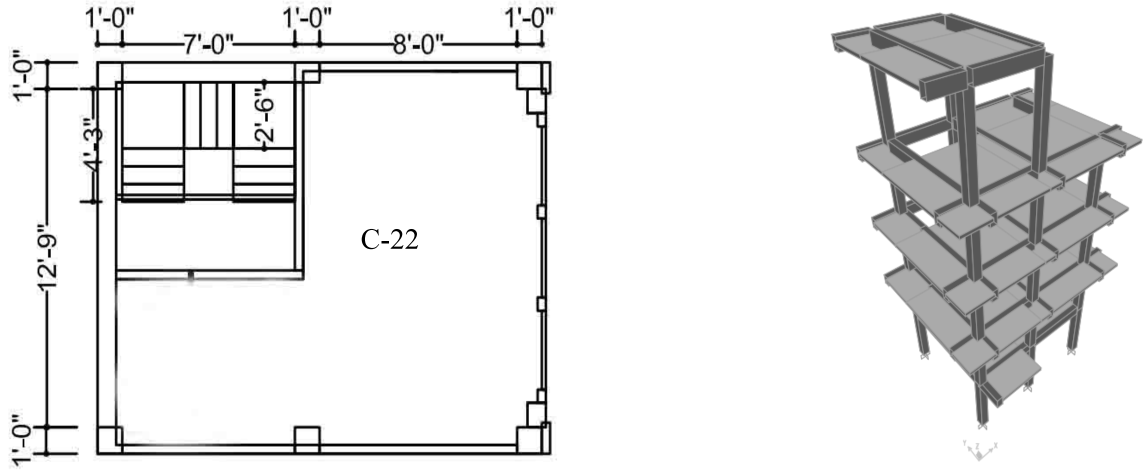


Fig. 3-3.9 Plan and isometric view of the building of class CB6

Table 3-3.4 Classification of reinforced concrete buildings

Class	CB1	CB2	CB3	CB4	CB5	CB6
Number of storeys	3	3	4	4	5	5
Building Representing Class	A7	A30	B22	C56	B63	C22
Buildings falling in the class	B23		A16	A28	B21	A52
	C31		A59	A49	B55	A53
	C33		B22	A63	B63	A65
			C8	B36	B64	A67
			C15	B44	B73	A68
			C16	C11		A80
			C18	C14		B46
			C34	C56		B90
			C12			B72
						C19
						C20
						C21
						C22
						C58

(2) Fragility Analysis of RC Buildings

With an objective to estimate the seismic vulnerability of the buildings in the area, a probabilistic risk analysis is carried out resulting into fragility curves. The probability of failure against different seismic intensity parameter, usually represented by peak ground acceleration (PGA) is obtained from the fragility curves. The simulated earthquake ground motion acceleration time history at the free field of Jhatapol area and named Lalitapura time history (Fig. 3-3.10) is used as the seismic input for the analysis f of buildings. The time history has a peak value of 0.437g. The time history data, for simplicity are rescaled to PGAs of 0.45g.

The modal analysis is carried out to obtain the vibration properties of the structures to understand the structural behaviour. The non–linear static procedure (Pushover Analysis) is applied to determine the capacity of the structure for the fragility analysis. The results of the pushover analysis is demonstrated as resistance of the structure in terms of defined collapse mechanism as formation of hinges to the top displacement, that is used as the capacity of the building structure. The time history dynamic analysis is used to determine the response of the structures, required for the fragility analysis. To obtain fragility curve, a failure mode for the frame is assumed, and the probability of exceedance of that particular mode is determined. Building fragility curves, which express the probability of a building reaching or exceeding certain damage state for a given ground motion parameter. For seismic loading, the fragility simply looks at the probability that the seismic demand placed on the structure (D) is greater than the capacity of the structure (C). Capacity of the building is defined as the top displacement at the stage formation of at least one frame mechanism. This probability statement is conditioned on a chosen intensity measure (IM) which represents the level of seismic loading. The generic representation of this conditional probability is given as by equation 3-3.1.

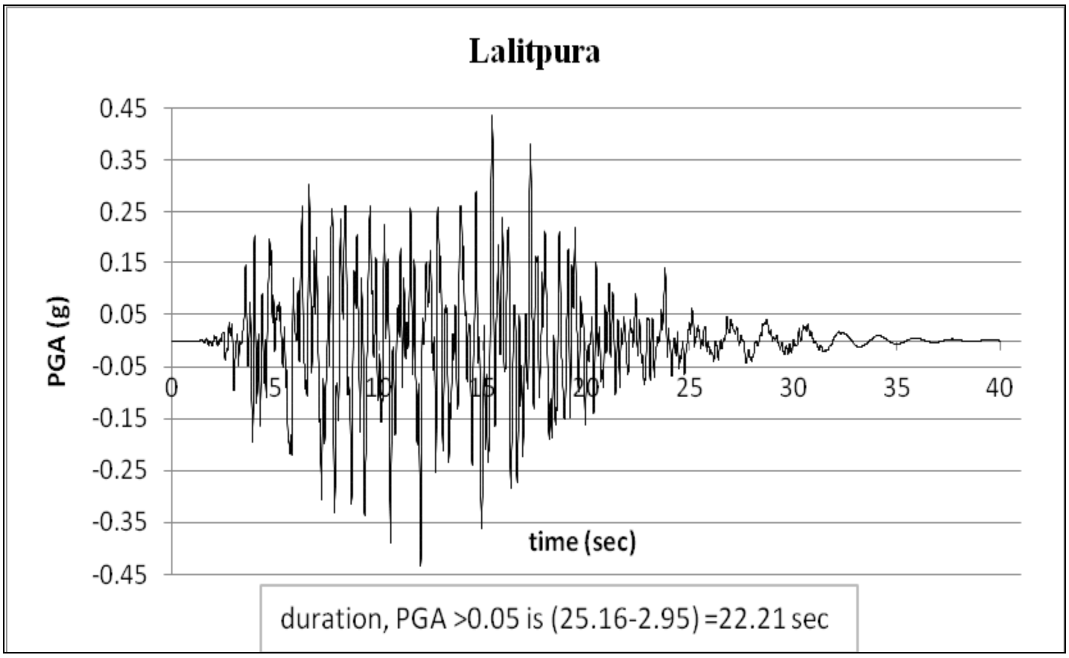


Fig. 3-3.10 Earthquake time histrorry Lalitpura

$$Fragility = P[D \geq C / IM] = P[C - D \leq 0.0 / IM] \tag{3-3.1}$$

The structural reliability is calculated using first order second moment (FOSM) method. In this method random variables are characterized by their first and second moments. In evaluation of the first and second moments of the failure function, say mean and standard deviation the first order Taylor's approximation is used. In order to calculate the reliability, the limit state equation is defined using equation 3-3.2.

$$M = S_c - S_d \tag{3-3.2}$$

In which C is capacity, D is response of building. M is margin of safety.

$$Pf = P\left[\frac{S_d}{S_c} \geq 1\right] \tag{3-3.3}$$

The probability of failure is determined by:

$$Pf = \Phi\left(\frac{\ln\left(\frac{S_d}{S_c}\right)}{\sqrt{\beta_d^2 + \beta_c^2}}\right) \tag{3-3.4}$$

Table 3-3.5 Response of the building PGA 0.45g

Building A7 (CB1)				
Storey level	Height (m)	Displacement (mm)	Drift ratio (%)	Base shear (KN)
Ground	0	0	0	864.05
First	2.84	69.7	2.454	
Second	5.68	140.13	2.480	
Third	8.52	176.09	1.266	
Building A30 (CB2)				
Ground	0	0	0	435.17
First	2.74	21.83	0.797	
Second	5.48	42.38	0.750	
Third	8.22	53.72	0.414	
Building B22 (CB3)				
Ground	0	0	0	1373.32
First	2.6	47.06	1.810	
Second	5.2	101.99	2.113	
Third	7.8	140.41	1.478	
Fourth	10.4	158.01	0.677	
Building C56 (CB4)				
Ground	0	0	0	1480.27
First	2.4	54.2	2.258	
Second	4.8	109.66	2.311	
Third	7.2	145.11	1.477	
Fourth	9.6	158.34	0.551	
Building B63 (CB5)				
Ground	0	0	0	2814.91
First	2.7	37.01	1.371	
Second	5.4	88	1.889	
Third	8.1	133.24	1.676	
Fourth	10.8	165.87	1.209	
Fifth	13.5	181.5	0.579	
Building C22				
Ground	0	0	0	1505.15
First	2.44	42.88	1.757	
Second	4.73	88.82	2.006	
Third	7.17	128.4	1.622	
Fourth	9.61	151.95	0.965	
Fifth	12.61	165.88	0.464	

The RC buildings are modeled and analyzed using finite element (FEM). The fragility curves for particular damage state are derived for the selected buildings from the capacity and response analysis of corresponding buildings. For the response analysis the seismic input in the form of ground motion time history is applied for the individual model, and the structural response in the form of displacement at each storey level is obtained. The storey displacements, storey drifts, base shear are shown in Fig. 3-3.5. These fragility curves for 6 classes of buildings are presented in Fig. 3-3.11 to 3-3.16.

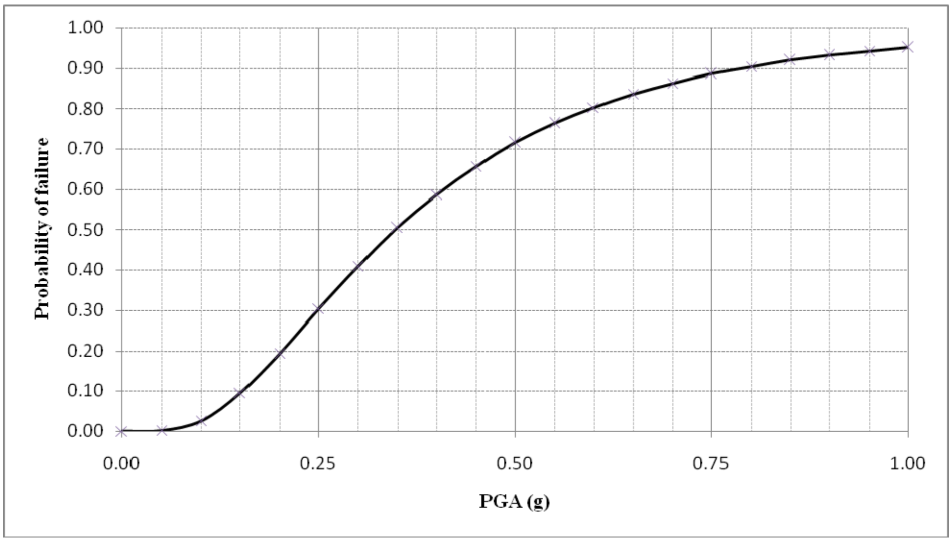


Fig. 3-3.11 Fragility curves for building A7 (CB1)

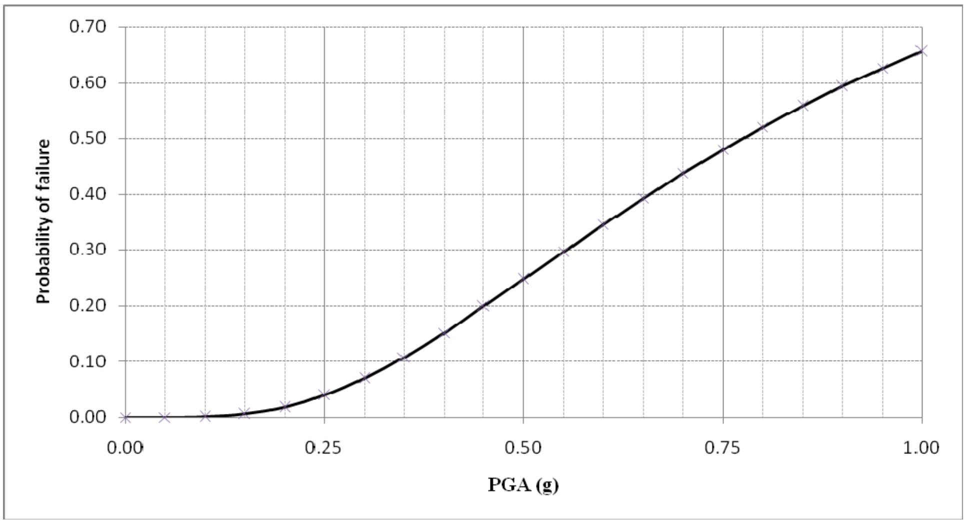


Fig. 3-3.12 Fragility curves for building A30 (CB2)

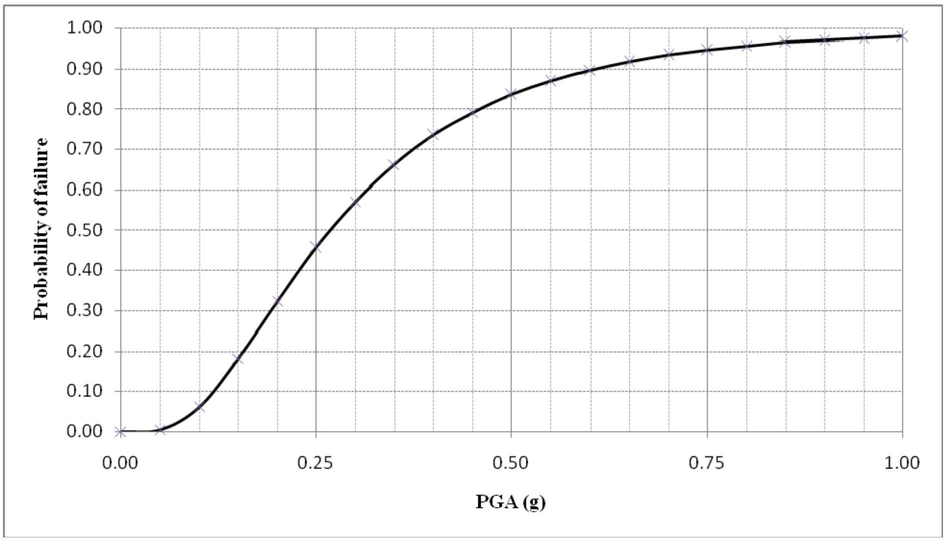


Fig. 3-3.13 Fragility curves for building B22 (CB3)

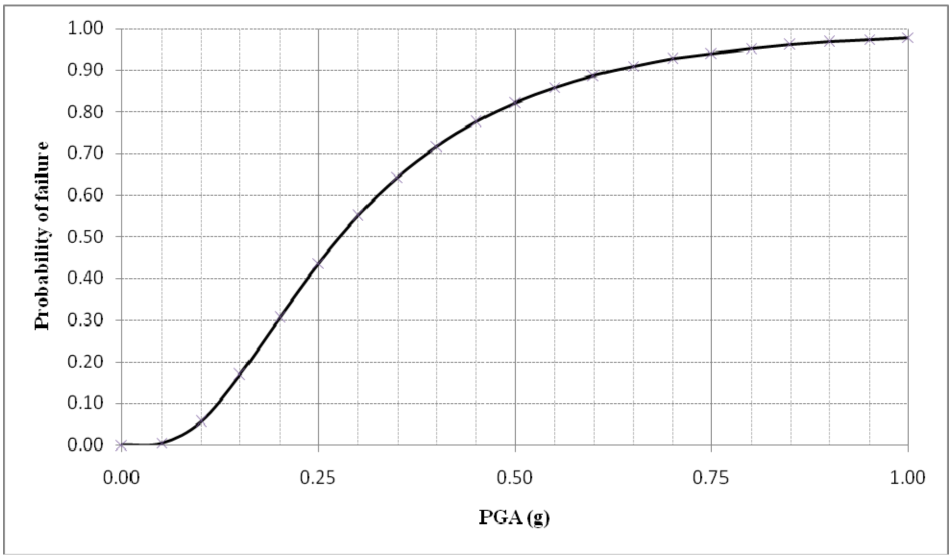


Fig. 3-3.14 Fragility curves for building C56 (CB4)

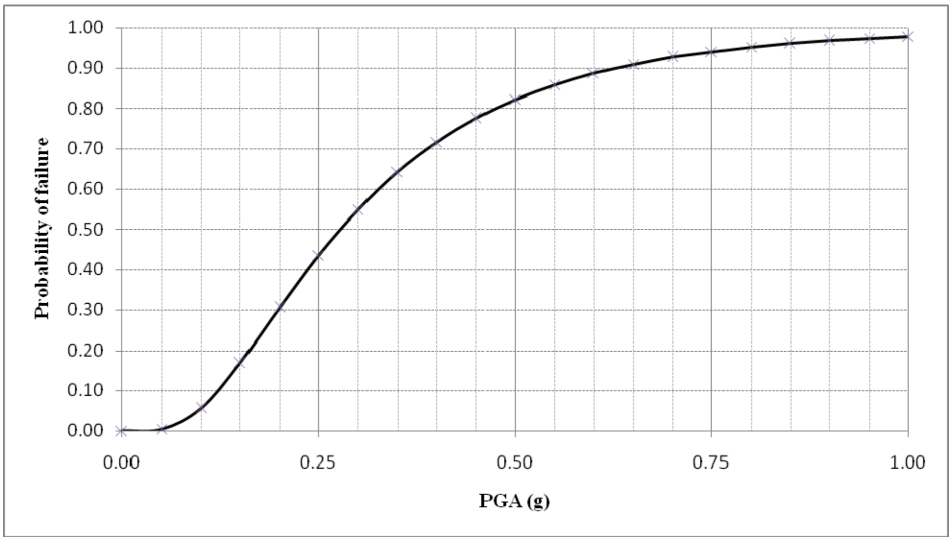


Fig. 3-3.15 Fragility curves for building B63 (CB5)

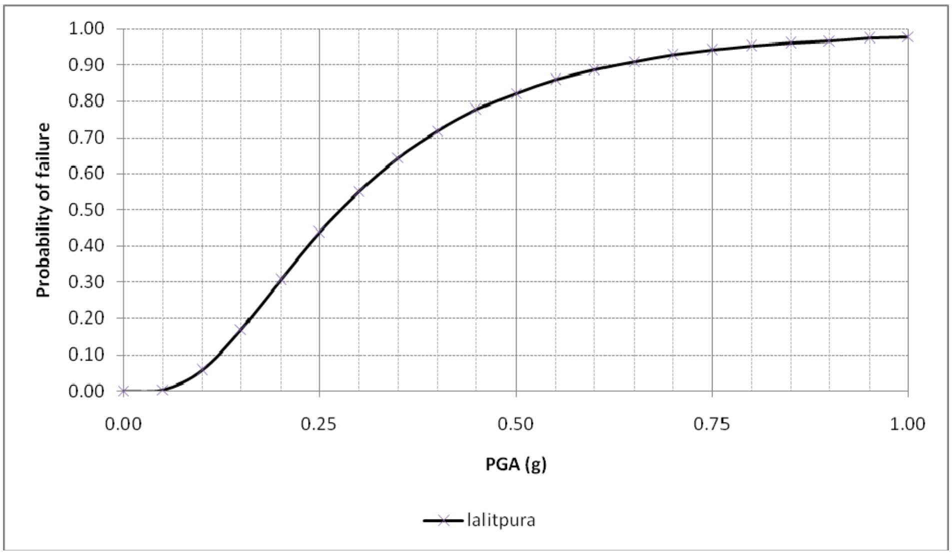


Fig. 3-3.16 Fragility curves for building C22 (CB6)

Non linear static (pushover) analysis is carried out to determine the capacity of each building. Various iterations are performed to find the minimum top displacement at which at least one frame forms base column hinges. Only all columns of the ground floor are assigned hinges. The hinge property is taken default FEMA 356 hinges for concrete column. The top displacement at that stage is defined as capacity of the building. The peak ground acceleration for a return period of 475 years expected at the surface of the Jhatapol area is 0.45g. Correspondingly the probabilities of failure of each of the 6 building classes are obtained from the fragility curves. The probabilities of failure of the buildings are presented in Table 3-3.6.

Table 3-3.6 Probability of Failure of the buildings for PGA =0.45g

Building Class	Damage state	Max. I.S.D. (%)	Bldg drift (%)	Probability of Failure (%)
CB1	heavily damaged	2.48	2.07	66
CB2	partially damaged	0.797	0.65	20
CB3	heavily damaged	2.113	1.52	79
CB4	heavily damaged	2.31	1.65	77
CB5	heavily damaged	1.89	1.23	77
CB6	heavily damaged	2.01	1.32	78

(3) Fragility Analysis of the traditional brick masonry buildings

As in the case of RC buildings, the seismic risk analysis in a probabilistic format is carried out to estimate the probability of failure for different values of peak ground acceleration. Specific to the properties of brick masonry buildings, a failure mechanism, different from that of RC buildings is assumed. At the outset, modal analysis of the brick masonry buildings is carried out to obtain the vibration properties. From linear time history analysis, parameters like the base shear and the displacement at different storey level are determined for the rescaled PGA value of 0.45g in the ground motion acceleration time history.

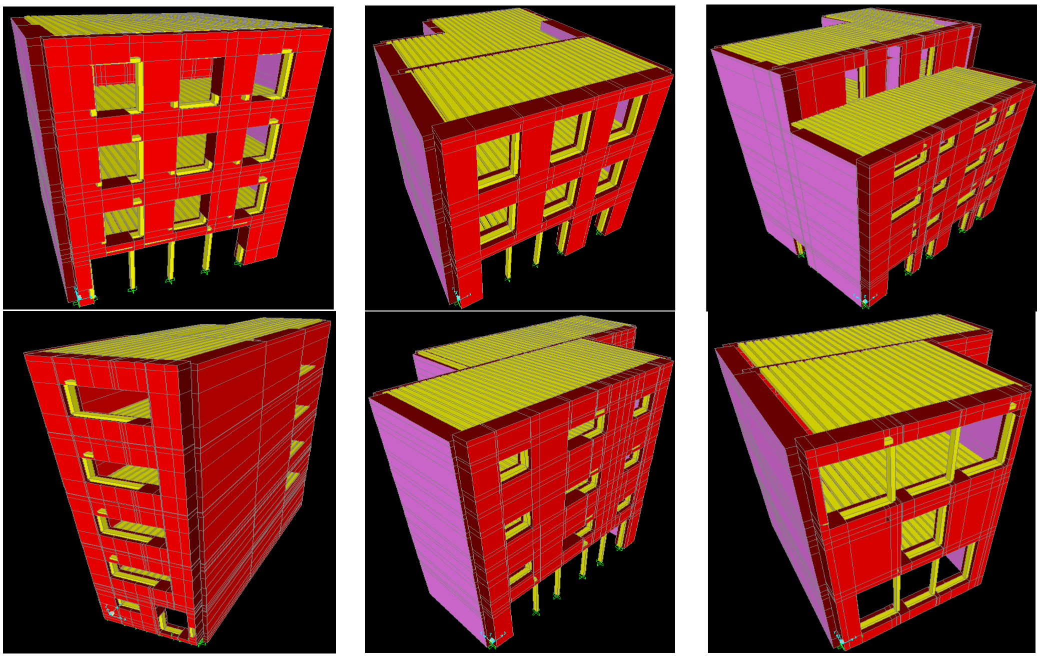


Fig. 3-3.17 FEM models of brick masonry building

The simulated ground motion acceleration time history with a PGA of 0.45g is used as the seismic input as in the case of RC buildings. The shear capacity of the brick masonry building is found out by manual calculation to develop fragility curve. The finite element modeling for masonry building is not that simple compared to that of RC buildings. The masonry buildings consist of walls of 45-60cm thickness reduced in upper floors, and with wooden usually doubly framed, door and windows. The floor is made of timber joists with planks or boards overlaid with mud or clay materials, and hence flexible floors. There is no actual data showing or proving exact percentage of fixity of flexibility of these floors which poses difficulties in modeling. In this analysis, wooden purlins are modeled as beam members and end moments are released then it act like hinge member. Six models considered for the building are shown in the Fig.3-3.17. The buildings are A48, A04, B47, C75, B15 and C26 respectively. The prefixes A, B and C stand for three blocks of the study area. Though survey has been done in three areas, only B block's result has been shown in this study. The response of the brick masonry buildings are the stresses in the walls due to earthquake ground motion, where as the failure mechanism is assumed to the shear failure in the wall, which is the usual reason for the serious damage , if not complete collapse. The calculation for the development of the fragility curves of the masonry buildings is presented in Table 3-3.7. The fragility curves for the six types of the masonry buildings classes are presented in Figs. 3-3.18 to 3-3.23.

Table 3-3.7 Development of Fragility Curve for Brick Masonry Building

PGA(g)	**Shear capacity N/mm^2, (S_c)**	**Sear demand N/mm^2, (S_d)**	**log normal distribution, ln(S_d/S_c)/0.64**	**Probability of failure**
0	0.3	-0.00006	0	0
0.05	0.3	0.0867	-1.939576	0.02621562
0.1	0.3	0.17346	-0.855993	0.19600086
0.15	0.3	0.26022	-0.222274	0.41205049
0.2	0.3	0.34698	0.2273198	0.58991246
0.25	0.3	0.43374	0.5760356	0.71770445
0.3	0.3	0.5205	0.8609491	0.80536696
0.35	0.3	0.60726	1.1018353	0.86473335

0.4	0.3	0.69402	1.3104973	0.90498618
0.45	0.3	0.78078	1.4945484	0.93248382
0.5	0.3	0.86754	1.6591862	0.95146086
0.55	0.3	0.9543	1.8081181	0.96470594
0.6	0.3	1.04106	1.9440816	0.97405719
0.65	0.3	1.12782	2.0691553	0.98073424
0.7	0.3	1.21458	2.1849549	0.98555392
0.75	0.3	1.30134	2.2927614	0.98906913
0.8	0.3	1.3881	2.3936074	0.9916582
0.85	0.3	1.47486	2.4883373	0.9935829
0.9	0.3	1.56162	2.5776509	0.99502628
0.95	0.3	1.64838	2.662134	0.99611765
1	0.3	1.73514	2.7422827	0.99694931

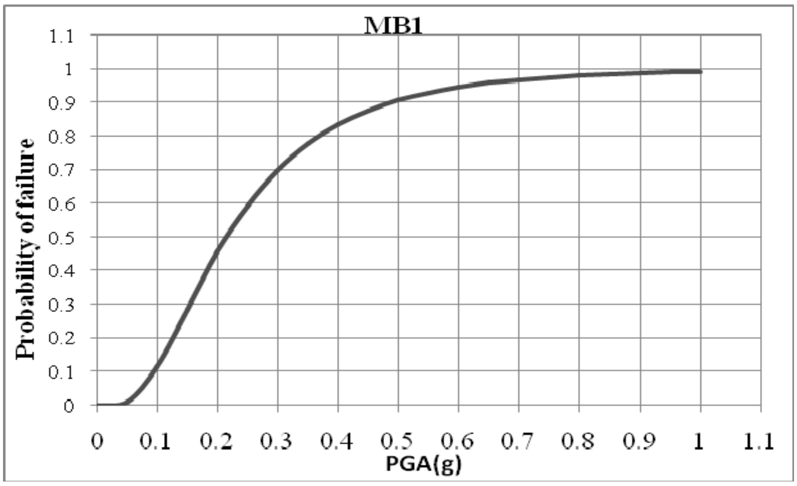


Fig. 3-3.18 Fragility curve for MB1

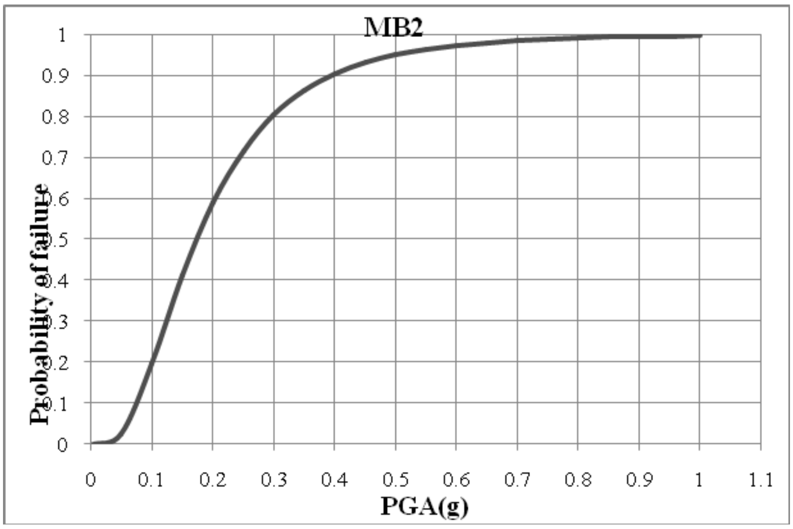


Fig. 3-3.19 Fragility curve for MB2

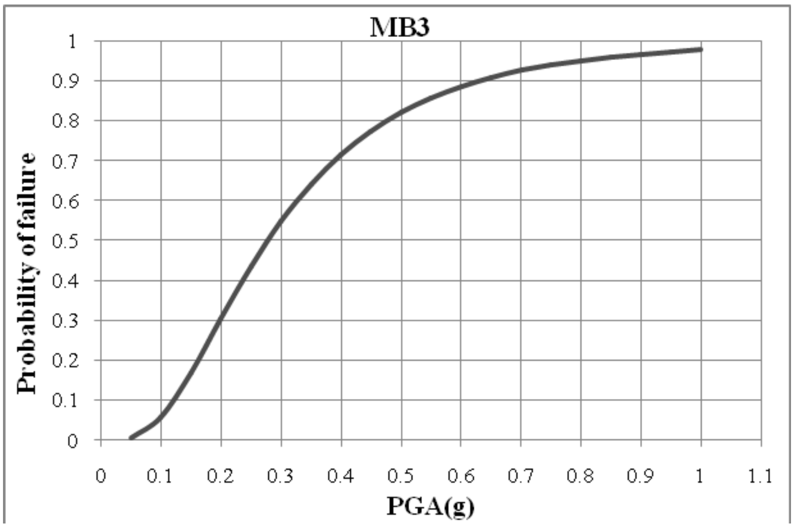


Fig. 3-3.20 Fragility curve for MB3

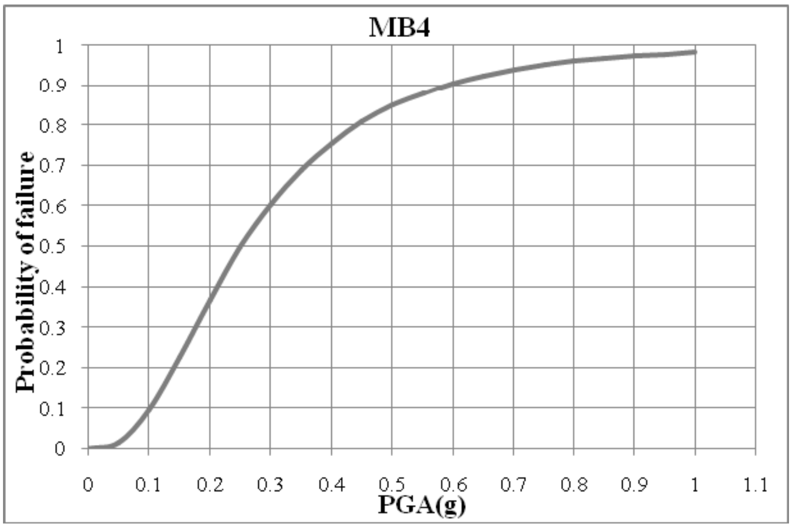


Fig. 3-3.21 Fragility curve for MB4

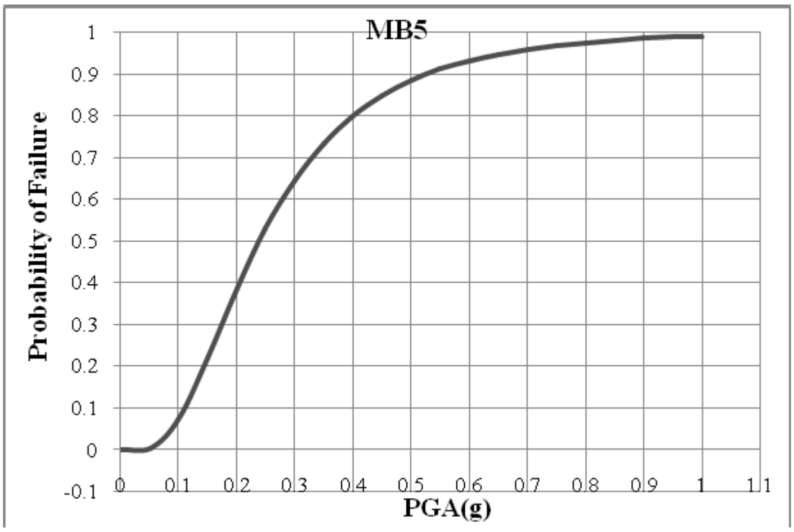


Fig. 3-3.22 Fragility curve for MB5

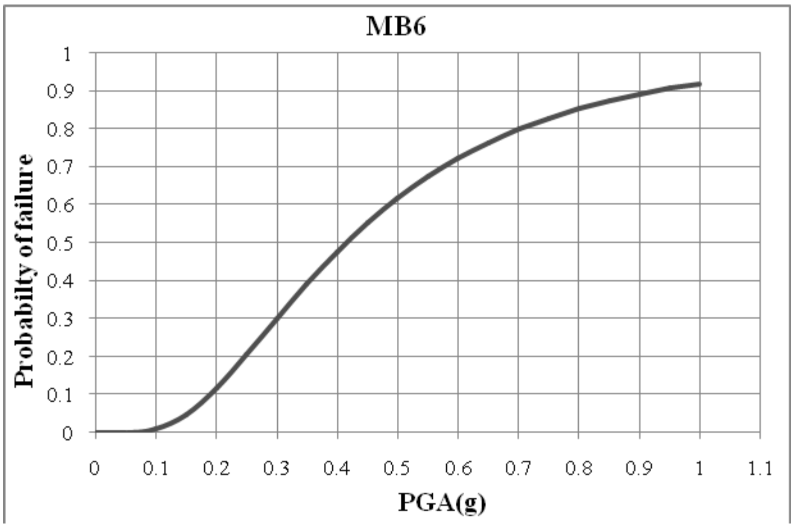


Fig. 3-3.23 Fragility curve for MB6

The probability of failure of the brick masonry buildings obtained for the value of peak ground acceleration 0.45g determined from the fragility curves is presented in Table 3-3.8.

Table 3-3.8 Probability of Failure of the buildings for PGA =0.45g

Building Class	Damage state	Location	Crushing state	Probability of failure (%)
MB1	Heavy damage	Top right corner of door at ground floor 1st floor	Crushing of masonry panel at bottom far end	88
MB2	Heavy damage	Around the door opening at ground floor	No	93
MB3	Heavy damage	Top right corner of door at ground floor and wall opening at 1st and 2nd floor	Crushing of masonry panel at bottom far end	77
MB4	Heavy damage	Around the door opening at ground floor	Crushing failure	81
MB5	Heavy damage	Around the small window opening at ground floor	Crushing of masonry panel at bottom far end	85
MB6	No damage		Crushing of masonry panel at bottom far end	55

(4) Vulnerability in the line of EMS-98

The vulnerability of the buildings, brick masonry buildings as well as the RC buildings are also studied for their seismic vulnerability in the line of the world accepted European Macroseismic Scale - EMS-98 1). In this method vulnerability index values are given based on the vulnerability classes for different building typologies as shown in Table 3-3.9. As the vulnerability (in the EMS-98) depends also on other factors such as: quality of workmanship, state of preservation, regularity, ductility, position, strengthening, and earthquake resistant design level; the methodology suggests the following definition of the vulnerability index (equation 3-3.5):

$$V_I = V_{I^*} + \Delta V_R + \Delta V_m \tag{3-3.5}$$

Where, V_I* is a Typological Vulnerability Index, ΔV_R is a Regional Vulnerability Factor, and ΔV_m is a Behavior Modifier Factor. The scoring for the vulnerability factors for masonry buildings is done as per Table 3-3.10.

Table 3-3.9 Attribution of vulnerability classes and vulnerability Index values for different building typologies 1)

Typologies		Building type	Vulnerability Classes A	B	C	D	E	F	Vulnerability Classes $V_{I\,min}$	V_I^-	V_I^*	V_I^+	$V_{I\,max}$
Masonry	M1	Rubble stone	■						0.62	0.81	0.873	0.98	1.02
	M2	Adobe (earth bricks)	■	▒					0.62	0.687	0.84	0.98	1.02
	M3	Simple stone	░	■					0.46	0.65	0.74	0.83	1.02
	M4	Massive stone		▒	■	░			0.3	0.49	0.616	0.793	0.86
	M5	Unreinforced M (old bricks)	░	■	░				0.46	0.65	0.74	0.83	1.02
	M6	Unreinforced M with r.c. floors		▒	■	░			0.3	0.49	0.616	0.79	0.86
	M7	Reinforced or confined masonry			░	■	▒		0.14	0.33	0.451	0.633	0.7
Reinforced Concrete	RC1	Frame in r.c. (without E.R.D)	░	▒	■				0.3	0.49	0.644	0.8	1.02
	RC2	Frame in r.c. (moderate E.R.D.)		░	▒	■	▒		0.14	0.33	0.484	0.64	0.86
	RC3	Frame in r.c. (high E.R.D.)			░	▒	■	▒	-0.02	0.17	0.324	0.48	0.7
	RC4	Shear walls (without E.R.D)		░	■	▒			0.3	0.367	0.544	0.67	0.86
	RC5	Shear walls (moderate E.R.D.)			░	■			0.14	0.21	0.384	0.51	0.7
	RC6	Shear walls (high E.R.D.)				░	■	▒	-0.02	0.047	0.224	0.35	0.54
Steel	S	Steel structures			░	▒	■	▒	-0.02	0.17	0.324	0.48	0.7
Timber	W	Timber structures		░	▒	■	▒		0.14	0.207	0.447	0.64	0.86

Situations: ■ Most Likely; ▒ Probable; ░ Less Probable (exceptional cases)

Table 3-3.10 Scores for the vulnerability factors for Masonry Buildings 1)

Vulnerability Factors	*Parameters*	V_m
State of preservation	Good maintenance	-0,04
	Bad maintenance	0.04
Number of floors	Low (1 or 2)	-0.02
	Medium (3, 4 or 5)	0.02
	High (6 or more)	0.06
Structural system	Wall thickness Distance between walls Connection between walls (Tie-rods, angle bracket) Connection horizontal structures-walls	-0,04 ÷ +0,04
Soft-story	Demolition/ Transparency	0.04
Plan Irregularity	…	0.04
Vertical Irregularity	…	0.02
Superimposed floors		0.04
Roof	Roof weight + Roof Thrust Roof Connections	0.04
Retrofitting interventions Aseismic Devices	Barbican, Foil arches, Buttresses	-0,08 ÷ +0,08
Aggregate building: position	Middle	-0.04
	Corner	0.04
	Header	0.06
Aggregate building: elevation	Staggered floors	0.02
	Buildings of different height	-0,04 ÷ +0,04
Foundation	Different level foundation	0.04
Soil Morphology	Slope	0.02
	Cliff	0.04

The vulnerability indices, thus obtained, for all buildings of the site are shown in Table 3-3.11.

3-3.11Table Vulnerability Index of the buildings

Code No.	Building Type RCC	Brick Masonry	State of prevention	No. of floors	Structural system	Plan irregularity	Vertical irregularity	Superimposed floors	Roof	Bldg position	Foundation	TOTAL V_m
B1		√	0.04	0.02	0.04	0	0	0.04	0.04	0.04	0	0.22
B2			0.04	0.02	0.04	0.04	0	0.04	0.04	-0.04	0	0.18

B3			0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.14
B4			0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.14
B5			0.04	0.02	0.04	0	0	0.04	0.04	0.06	0	0.24
B6			0.04	0.02	0.04	0	0	0	0.04	0.06	0	0.2
B7			0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.14
B8			0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.14
B9		√	-0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.06
B10		√	-0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.06
B11		√	0.04	0.02	0.04	0.04	0	0.04	0.04	-0.04	0	0.18
B12		√	-0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.06
B13			0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.14
B14			0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.14
B15			0.04	0.02	0.04	0.04	0	0.04	0.04	-0.04	0	0.18
B16			0.04	0.02	0.04	0.04	0.02	0.04	0.04	-0.04	0	0.2
B17			0.04	0.02	0.04	0	0	0	0.04	0.04	0	0.18
B18			0.04	0.02	0.04	0	0	0	0.04	0.06	0	0.2
B19			0.04	0.02	0.04	0	0	0	0.04	-0.04	0	0.1
B20		√	-0.04	0.02	0.04	0	0	0	0.04	-0.04	0	0.02
B21	√		0.04	0	0	0.04	0	0	0	0	0.04	0.12
B22			0.04	0	0	0	0	0	0	0	0.04	0.08
B23			0.04	0	0	0	0	0	0	0	0.04	0.08
B24		√	0.04	0.02	0.04	0	0	0	0.04	-0.04	0	0.1
B25		√	-0.04	0.02	0.04	0	0	0	0.04	-0.04	0	0.02
B26		√	0.04	0.02	0.04	0	0	0	0.04	-0.04	0	0.1
B27		√	0.04	0.02	0.04	0.04	0	0	0.04	0.04	0	0.22
B28		√	-0.04	0.02	0.04	0.04	0	0.04	0.04	-0.04	0	0.1
B29		√	-0.04	0.06	0.04	0	0	0.04	0.04	-0.04	0	0.1
B30A		√	-0.04	-0.02	0.04	0	0	0	0.04	0	0	0.02
B30		√	0.04	0.02	0.04	0.04	0.03	0.04	0.04	-0.04	0	0.21
B31		√	0.04	0.02	0.04	0	0	0.04	0.04	0.04	0	0.22
B32		√	0.04	0.02	0.04	0.04	0	0	0.04	-0.04	0	0.14
B33		√	-0.04	0.02	0.04	0	0	0	0.04	-0.04	0	0.02
B34			-0.04	-0.02	0.04	0	0	0	0.04	-0.04	0	-0.02
B35	√		0.04	-0.04	0	0	0	0	0	0	0.04	0.04
B36	√		0.04	0	0	0.04	0	0	0	0	0.04	0.12
B37		√	-0.04	0.02	0.04	0	0	0.04	0.04	0.06	0	0.16
B38		√	-0.04	0.02	0.04	0	0.02	0	0.04	-0.04	0	0.04
B39		√	0.04	0.02	0.04	0	0	0	0.04	0.04	0	0.18
B40		√	0.04	0.02	0.04	0.04	0	0	0.04	-0.04	0	0.14
B41		√	0.04	0.02	0.04	0.04	0	0.04	0.04	-0.04	0	0.18
B42		√	-0.04	0.02	0.04	0	0	0	0.04	0.04	0	0.1
B43			0.04	0.02	0.04	0.04	0	0.04	0.04	-0.04	0	0.18
B44	√		0.04	0	0	0.04	0	0	0	0	0.04	0.12
B45		√	0.04	0.06	0.04	0.04	0	0.04	0.04	-0.04	0	0.22
B46	√		0.04	0	0	0	0	0	0	0	0.04	0.08
B47			0.04	0.02	0.04	0.04	0	0.04	0.04	-0.04	0	0.18

B48			0.04	0.02	0.04	0.04	0.02	0.04	0.04	-0.04	0	0.2
B49		√	0.04	0.02	0.04	0.04	0	0	0.04	0.04	0	0.22
B50		√	0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.14
B51			0.04	-0.02	0.04	0	0	0	0.04	0.06	0	0.16
B52			-0.04	-0.02	0.04	0.04	0	0	0.04	-0.04	0	0.02
B53		√	-0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.06
B54			0.04	0.02	0.04	0.04	0	0.04	0.04	0.04	0	0.26
B55			0.04	0	0	0.04	0	0	0	0	0.04	0.12
B56			-0.04	-0.02	0.04	0	0	0	0.04	-0.04	0	-0.02
B57			0.04	0.02	0.04	0.04	0	0	0.04	-0.04	0	0.14
B58			0.04	-0.02	0.04	0	0	0	0.04	0.06	0	0.16
B59			0.04	-0.02	0.04	0	0	0	0.04	-0.04	0	0.06
B60			-0.04	-0.02	0.04	0	0	0	0.04	-0.04	0	-0.02
B61			0.04	0.02	0.04	0	0	0	0.04	-0.04	0	0.1
B62			0.04	-0.02	0.04	0	0	0	0.04	-0.04	0	0.06
B63			0.04	0	0	0.04	0	0	0	0	0.04	0.12
B64			0.04	0	0	0.04	0	0	0	0	0.04	0.12
B65			0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.14
B66			0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.14
B67			0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.14
B68			0.04	0.02	0.04	0	0	0	0.04	-0.04	0	0.1
B69			0.04	0.02	0.04	0	0.02	0.04	0.04	-0.04	0	0.16
B70			0	0	0.04	0	0.02	0	0.04	-0.04	0	0.06
B71			0.04	-0.02	0.04	0	0	0	0.04	0.06	0	0.16
B72			0.04	0	0	0	0.04	0	0	0	0.04	0.12
B73	√		0.04	0	0	0.04	0	0	0	0	0.04	0.12
B75			0.04	0.02	0.04	0.04	0	0.04	0.04	-0.04	0	0.18
B76		√	-0.04	0.02	0.04	0.04	0	0.04	0.04	-0.04	0	0.1
B77			-0.04	0.02	0.04	0.04	0	0.04	0.04	-0.04	0	0.1
B78			0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.14
B79			0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.14
B80		√	-0.04	0.02	0.04	0.04	0	0.04	0.04	-0.04	0	0.1
B81		√	0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.14
B82			0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.14
B83		√	-0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.06
B84			0.04	0.02	0.04	0.04	0	0.04	0.04	-0.04	0	0.18
B85		√	-0.04	0.02	0.04	0.04	0	0	0.04	-0.04	0	0.06
B86		√	-0.04	0.02	0.04	0	0	0.04	0.04	-0.04	0	0.06
B87			0.04	0.02	0.04	0.04	0	0	0.04	-0.04	0	0.14
B88			0.04	0.02	0.04	0.04	0	0	0.04	-0.04	0	0.14
B89			0.04	0.02	0.04	0.04	0	0	0.04	-0.04	0	0.14
B90			0.04	0	0	0	0.04	0	0	0	0.04	0.12
B91	√		0.04	-0.04	0	0	0	0	0	0	0.04	0.04

The damage indices for each of the buildings are obtained by combining all the scores obtained from

Vulnerability Index, Probability of failure from Fragility analysis, Vulnerability Class and Visual Inspection. Using the expert opinion, the weightage for each of the method in the calculation of Damage Index is as follows;

Vulnerability Index 10%
Probability of failure 60%
Vulnerability Class 10%
Visual Inspection 20%

The damage indices of the buildings are presented in Table 3-3.12 and in the Fig. 3-3.24.

Table 3-3.12 Damage Index of the buildings

Code No.	Building Type		TOTAL V_m	Normalized V_m	Original for RCC	Probability of failure (%)	Vulnerability class	Visual inspection	Total	Max/ Min
	RCC	Brick Masonry								
B1		√	0.22	0.37		77	0.644	0.20	0.60	0.75
B2			0.18	0.30		77	0.644	0.50	0.66	0.34
B3			0.14	0.23		85	0.644	0.20	0.64	
B4			0.14	0.23		81	0.644	0.05	0.58	
B5			0.24	0.40		81	0.644	0.20	0.63	
B6			0.2	0.33		77	0.644	0.50	0.66	
B7			0.14	0.23		77	0.644	0.50	0.65	
B8			0.14	0.23		81	0.644	0.50	0.67	
B9		√	0.06	0.10		81	0.644	0.00	0.56	
B10		√	0.06	0.10		81	0.644	0.00	0.56	
B11		√	0.18	0.30		81	0.644	0.20	0.62	
B12		√	0.06	0.10		55	0.644	0.00	0.40	
B13			0.14	0.23		81	0.644	0.50	0.67	
B14			0.14	0.23		77	0.644	0.20	0.59	
B15			0.18	0.30		85	0.644	0.50	0.70	
B16			0.2	0.33		77	0.644	0.20	0.60	
B17			0.18	0.30		93	0.644	0.50	0.75	
B18			0.2	0.33		88	0.644	0.50	0.73	
B19			0.1	0.17		85	0.644	0.05	0.60	
B20		√	0.02	0.03		85	0.644	0.00	0.58	
B21	√		0.12	0.20	77	50	0.74	0.00	0.39	
B22			0.08	0.13	79	50	0.74	0.00	0.39	
B23			0.08	0.13	65	50	0.74	0.00	0.39	
B24		√	0.1	0.17		55	0.644	0.50	0.51	
B25		√	0.02	0.03		81	0.644	0.00	0.55	
B26		√	0.1	0.17		85	0.644	0.50	0.69	
B27		√	0.22	0.37		81	0.644	0.50	0.69	
B28		√	0.1	0.17		85	0.644	0.00	0.59	
B29		√	0.1	0.17		55	0.644	0.00	0.41	
B30A		√	0.02	0.03		55	0.644	0.00	0.40	
B30		√	0.21	0.35		85	0.644	0.50	0.71	

B31		√	0.22	0.37		85	0.644	0.05	0.62	
B32		√	0.14	0.23		93	0.644	0.20	0.69	
B33		√	0.02	0.03		77	0.644	0.00	0.53	
B34			-0.02	-0.03		55	0.74	0.00	0.40	
B35	√		0.04	0.07	55	50	0.74	0.00	0.38	
B36	√		0.12	0.20	77	50	0.74	0.00	0.39	
B37		√	0.16	0.27		55	0.644	0.00	0.42	
B38		√	0.04	0.07		77	0.644	0.00	0.53	
B39		√	0.18	0.30		85	0.644	0.50	0.70	
B40		√	0.14	0.23		85	0.644	0.20	0.64	
B41		√	0.18	0.30		55	0.644	0.20	0.46	
B42		√	0.1	0.17		88	0.644	0.00	0.61	
B43			0.18	0.30		85	0.644	0.20	0.64	
B44	√		0.12	0.20	77	50	0.74	0.00	0.39	
B45		√	0.22	0.37	50	85	0.644	0.00	0.61	
B46	√		0.08	0.13	77	50	0.74	0.00	0.39	
B47			0.18	0.30		77	0.644	0.50	0.66	
B48			0.2	0.33		85	0.644	0.50	0.71	
B49		√	0.22	0.37		81	0.644	0.20	0.63	
B50		√	0.14	0.23		55	0.644	0.50	0.52	
B51			0.16	0.27		55	0.644	0.50	0.52	
B52			0.02	0.03		93	0.644	0.00	0.63	
B53		√	0.06	0.10		55	0.644	0.00	0.40	
B54			0.26	0.43		55	0.644	0.50	0.54	
B55			0.12	0.20	77	50	0.74	0.00	0.39	
B56			-0.02	-0.03		55	0.644	0.00	0.39	
B57			0.14	0.23		77	0.644	0.20	0.59	
B58			0.16	0.27		55	0.644	0.20	0.46	
B59			0.06	0.10		55	0.644	0.20	0.44	
B60			-0.02	-0.03		55	0.644	0.00	0.39	
B61			0.1	0.17		55	0.644	0.20	0.45	
B62			0.06	0.10		55	0.644	0.20	0.44	
B63			0.12	0.20	77	50	0.74	0.00	0.39	
B64			0.12	0.20	77	50	0.74	0.20	0.43	
B65			0.14	0.23		93	0.644	0.50	0.75	
B66			0.14	0.23		77	0.644	0.50	0.65	
B67			0.14	0.23		77	0.644	0.50	0.65	
B68			0.1	0.17		81	0.644	0.20	0.61	
B69			0.16	0.27		55	0.644	0.50	0.52	
B70			0.06	0.10		55			0.34	
B71			0.16	0.27		55	0.644	0.20	0.46	
B72			0.12	0.20	77	50	0.74	0.00	0.39	
B73	√		0.12	0.20	77	50	0.74	0.00	0.39	
B75			0.18	0.30		77	0.644	0.50	0.66	
B76		√	0.1	0.17		85	0.644	0.00	0.59	
B77			0.1	0.17		55	0.644	0.00	0.41	

B78			0.14	0.23		77	0.644	0.20	0.59	
B79			0.14	0.23		81	0.644	0.50	0.67	
B80		√	0.1	0.17		81	0.644	0.00	0.57	
B81		√	0.14	0.23		77	0.644	0.50	0.65	
B82			0.14	0.23		77	0.644	0.50	0.65	
B83		√	0.06	0.10		85	0.644	0.00	0.58	
B84			0.18	0.30		77	0.644	0.05	0.57	
B85		√	0.06	0.10		93	0.644	0.00	0.63	
B86		√	0.06	0.10		81	0.644	0.00	0.56	
B87			0.14	0.23		81	0.644	0.50	0.67	
B88			0.14	0.23		88	0.644	0.50	0.72	
B89			0.14	0.23		93	0.644	0.20	0.69	
B90			0.12	0.20	77	50	0.74	0.00	0.39	
B91	√		0.04	0.07	77	50	0.74	0.00	0.38	

Fig. 3-3.24 Damage indices

The damage indices obtained for the B block are shown in the Fig. 3-3.24 with different level of values. The dark black color shows higher damage potential and the light color shows lower damage potential. It is seen from the figure that the buildings: B15, B17, B18, B30, B39, B48, B65 and B88 are having the maximum damage index in the range of 0.71 – 0.75, calling for the need of strengthening of the buildings. The high value of the damage index is attributed to the high score for visual screening condition and the probability of failure as output of the detailed structural analysis for the possible seismic hazard at the site. The minimum damage index is in the range of 0.34 – 0.40, indicating basically the need for restoration, that is, reinstatement of the strength of the structural members at par with their original state, and necessary repairs.

Reference

1) Grunthal, G.: *European Macroseismic Scale 1998; EMS-9*8, Cahiers du Centre de European de Geodynamique et de Seismologie 15. Luxembourg: European Seimological Commission, Sub commission on Engineering Seismology, Working Group Macroseismic Scales, 1998

3-3.6 Application of Risk Evaluation Methods to Case Study Area

Aiko Furukawa
Junji Kiyono
Hitoshi Taniguchi
Kenzo Toki
Masatoshi Tatsumi

(1) Jhatapo area

Our target area is Jhatapo area shown in Fig. 1. In Fig. 1, the buildings are classified by number of stories and building types. There are many historic masonry building in the target area as shown in Fig.2, and most of them are unreinforced masonry buildings which have the high possibility to get severe damage during earthquakes.

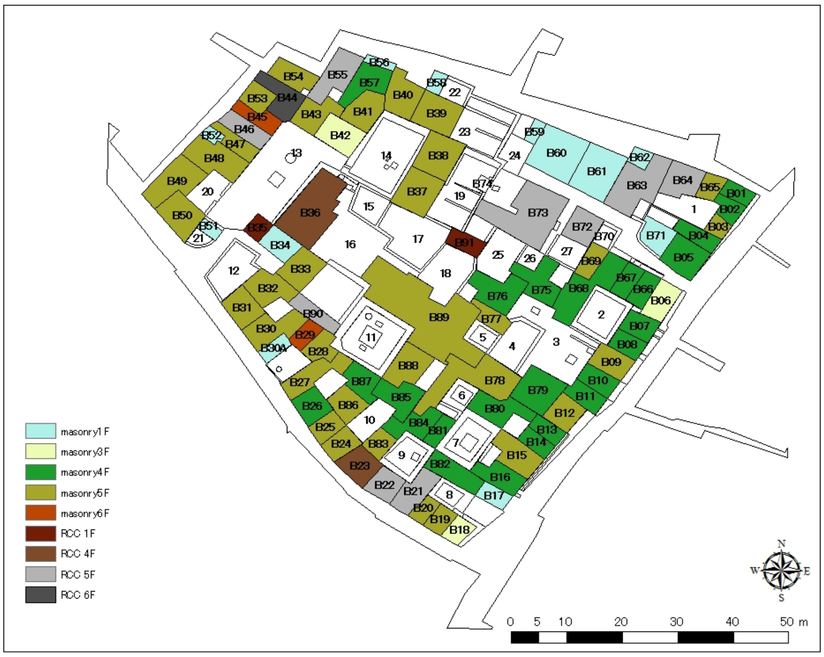


Fig.1 Target area (Jhatapo area)

(a) residence and stores (b) residence

Fig.2 Typical masonry buildings in the target area

(2) Complete enumeration

A complete enumeration was done for the buildings in the target area. There are 13 buildings with one story, but they are neglected since they are monuments of vacant buildings for nonresidential use.

When the buildings are classified with the number of stories, the ratio of 3, 4, 5 and 6 story buildings are 3, 37, 56 and 4%, respectively as shown in Fig. 3. The buildings with 4 and 5 stories occupies 93% of the buildings. The number of buildings with 3 and 6 stories are scant, and there are no buildings with two stories.

When the buildings are classified with the building type, 86% of the buildings are unreinforced masonry buildings, and the other buildings are confined masonry buildings with reinforced concrete (RCC) as shown in Fig.4.

Most of the buildings experienced repair and an addition of stories to the original buildings. All of the buildings with 5 and 6 stories experienced the addition of stories. The number of added stories for 4 and 5-story buildings are shown in Fig. 5. From the figure, the most of the buildings were originally 3 story buildings before the addition.

The buildings are classified into 7 types by the number of stories and building types as shown in Fig. 6 and Table 1. They are masonry buildings of 3, 4, 5 and 6 stories, and confined masonry buildings with reinforced concrete (RCC) of 4, 5 and 6 stories. We made 7 analytical models according to this classification, and estimates the seismic risk in this section.

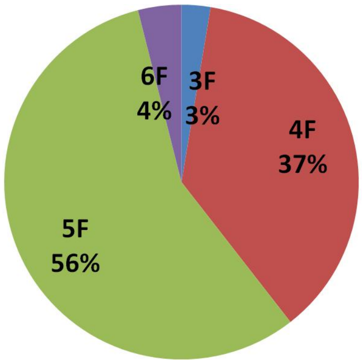


Fig. 3 Classification by the number of stories

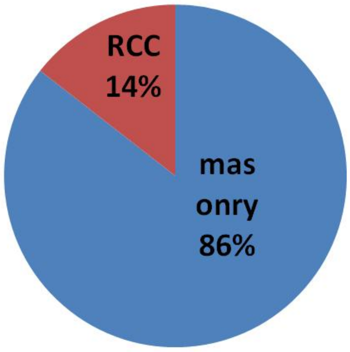
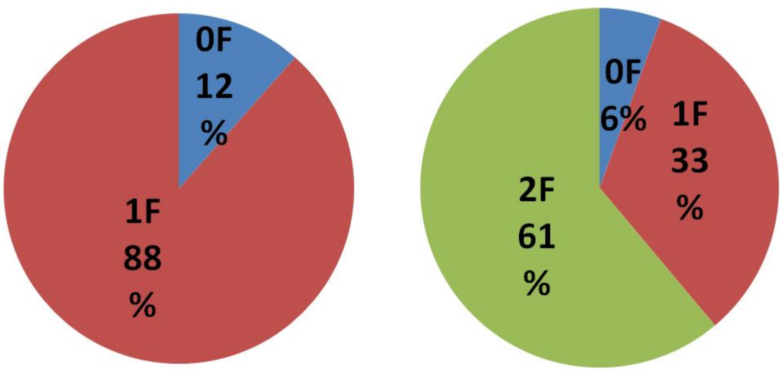


Fig. 4 Classification by building type



(a) 4-story masonry buildings (b) 5-story masonry buildings

Fig. 5 Number of additional stories

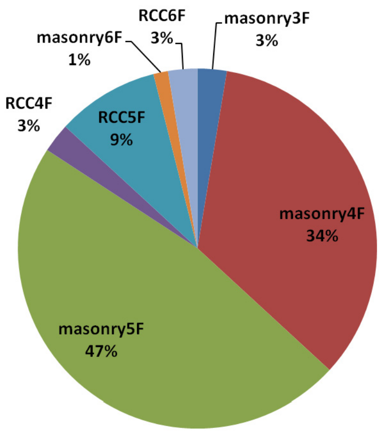


Fig.6 Classification of buildings

Table 1 Classification of buildings

building type	masonry3F	masonry4F	masonry5F	masonry6F	RCC4F	RCC5F	RCC6F
number of buildings	2	26	36	1	2	7	2
ratio (%)	2.63%	34.21%	47.37%	1.32%	2.63%	9.21%	2.63%

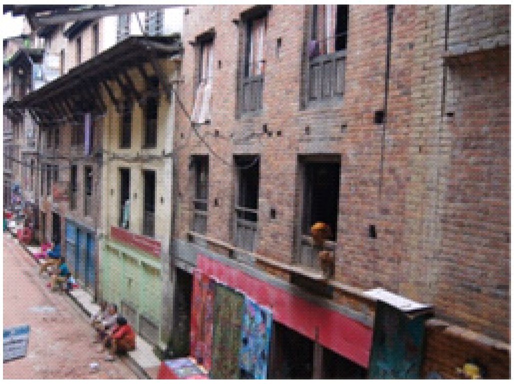


Fig.7 Continuous buildings

(3) Seismic risk evaluation

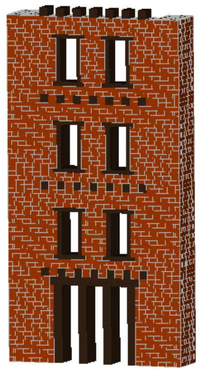
a) Analytical modeling

All buildings are continuous and shear the party walls with the adjacent buildings as shown in Fig.7. Moreover, according to the previous study by D'Ayala[1)], they assume the damage to façade for the continuous buildings. Therefore, we considered that the failure to the façade is the dominant failure mode and numerically estimated the damage to façade.

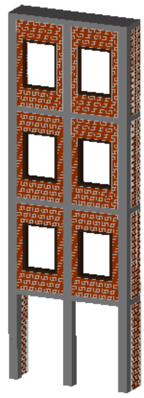
We made 7 analytical models according to the classification. The analytical models for a 4-story masonry building, and a 4-story confined masonry building with reinforced concrete is shown in Fig.8. The size of each brick is 10cm x 10cm x 20cm. The façade and side wall of 1m are modeled. The back of the side wall is supported by the fixed elements which cannot be seen in Fig. 8.

For masonry buildings, each story is 2.0m high, and the width is 4.2m. Each floor is composed of timber beams, The stories higher than 3rd floor is considered to be added, and the depth of the wall is 60cm for 1st-3rd floors, and 40cm for floors higher than 3rd floor based on the interviews to the local people.

For confined masonry buildings, each story is 3.0m high, the width is 4.1m, and the section of RC frame is 30cm x 30cm. The depth of the wall is 20cm for 1st-3rd floors, and 10cm for floors higher than 3rd floor.



(a) 4-story masonry building



(b) 4-story confined masonry building

Fig. 8 Analytical models

Table 2 Material properties and failure criteria

Variable	Adobe Brick	Mortar	Wood	RC
Mass density (kg/m^3)	1.8×10^3	-	7.0×10^2	2.3×10^3
Young's modulus (N/m^2)	2.7×10^8	2.7×10^8	6.3×10^8	2.5×10^{10}
Poisson's ratio	0.11	0.25	0.3	0.2
Tensile strength f_t (N/m^2)	-	0.0	1.1×10^8	1.91×10^6
Shear strength c (N/m^2)	-	9.0×10^4	9.0×10^6	2.2×10^6
Friction angle ϕ	-	42.5°	0°	32°
Compressive strength (N/m^2)	-	1.58×10^6	4.5×10^7	2.4×10^7

The parameters used are shown in Table 2. The time interval is 4.0×10^{-5}sec for the masonry buildings, and 1.0×10^{-5}sec for the confined masonry buildings.

The ground motion is input in the out-of-plane direction of the buildings.

b) Damage index

To evaluate structural damage, damage index proposed by Okada and Takai is used. Their definition of each damage index is shown in Table 3 and Fig. 9[2)]. Coburn summarized the progression of damage to the masonry structural system and damage grade as shown in Fig. 9[3)], and then Okada and Takai scored damage index which takes between 0 (No damage) and 1 (Total Collapse) for each damage grade according to the definition by the European Macroseismic Scale 1998 (EMS98)[4)].

c) Damage index function and fragility curve

This study developed a damage index function for each classified building and a fragility curves for all buildings except one-story buildings.

Damage index function is the relationship between the seismic intensity and damage index. The fragility curve is the relationship between the seismic intensity and the probability which exceeds a certain damage index.

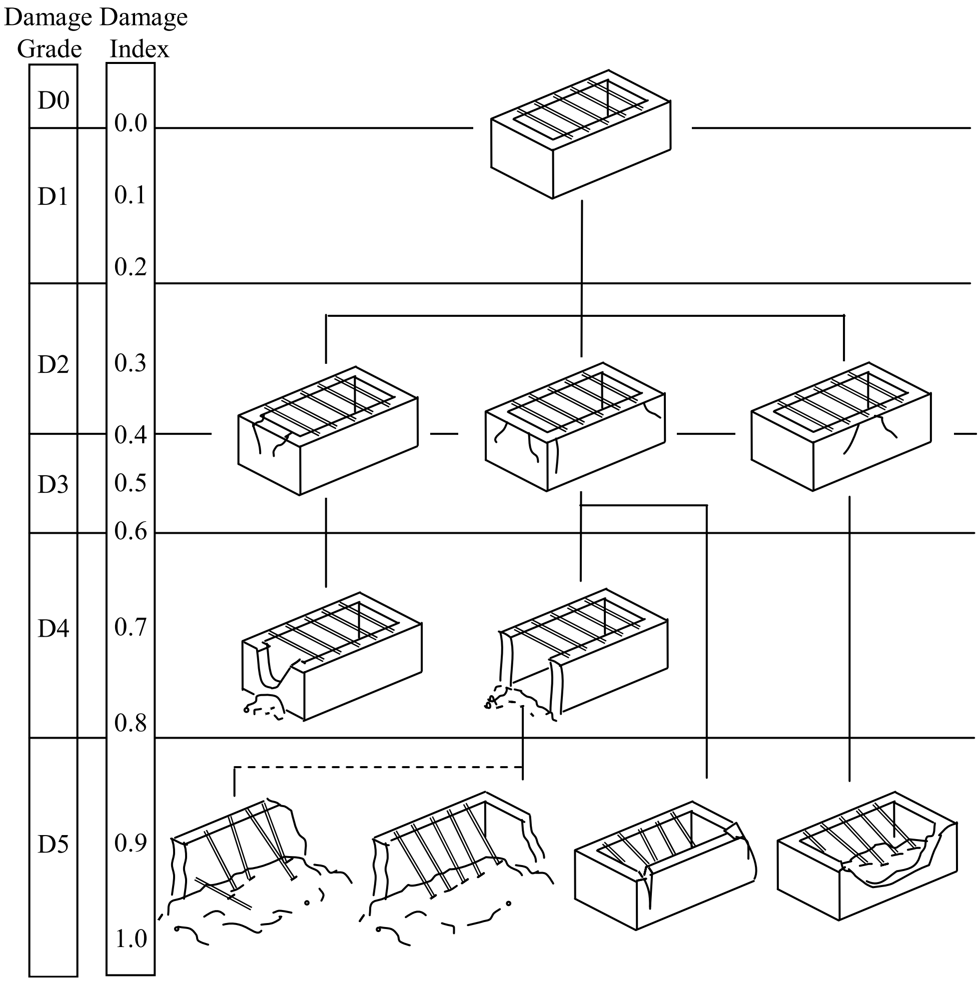


Fig.9 Damage pattern and its relevant damage index [2),3)]

Table 3 Damage index and classification of damage to masonry buildings (EMS98)[4)]

Damage Grade	Damage Index	Damage Description	Damage State
D0	0.0	No damage	No damage
D1	0.0～0.2	Negligible to sight damage	Hair-line cracks in very few walls. Fall of small pieces of plaster only. Fall of loose stones from upper parts of buildings in very few cases.
D2	0.2 ～ 0.4	Moderate damage	Cracks in many walls. Fall of fairly large pieces of plaster. Partial collapse of chimneys.
D3	0.4 ～ 0.6	Substantial to heavy damage	Large and extensive cracks in most walls. Roof tiles detach. Chimneys fracture at the roof line; failure of individual non-structural elements (partitions, gable walls).
D4	0.6 ～ 0.8	Very heavy damage	Serious failure of walls, partial structural failure of roofs and floors
D5	0.8 ～ 1.0	Destruction	Total of near total collapse

(4) Results

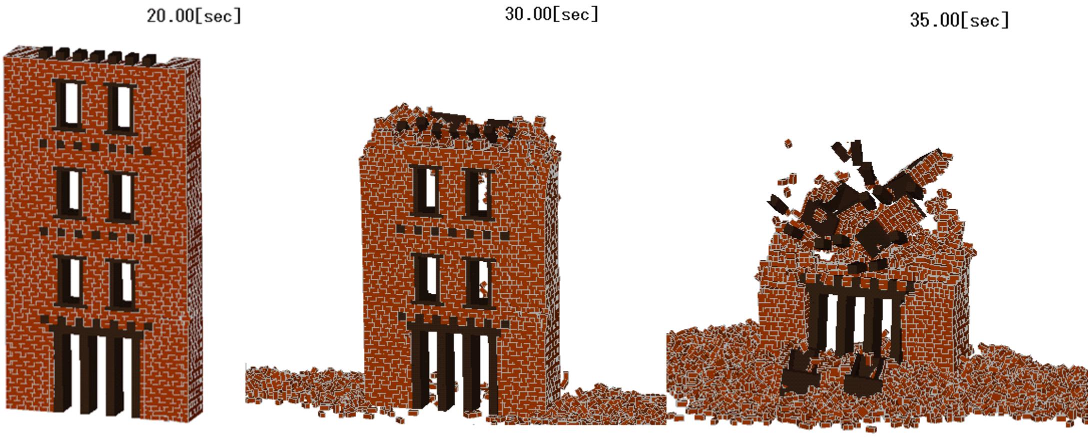
a) Estimated damage index

Three input ground motions with different occurrence probabilities are input to 7 models. The results are shown in Figs. 10-16.

Since the ground motion is input in the out-of-plane direction of the façade, the façade of the masonry buildings vibrated in the out-of-plane direction and failed. This made the side walls fall down and there are many fallen bricks around the buildings.

(a)40% (b)10% (c) 5%

Fig. 10 Seismic behavior of 3-story masonry building

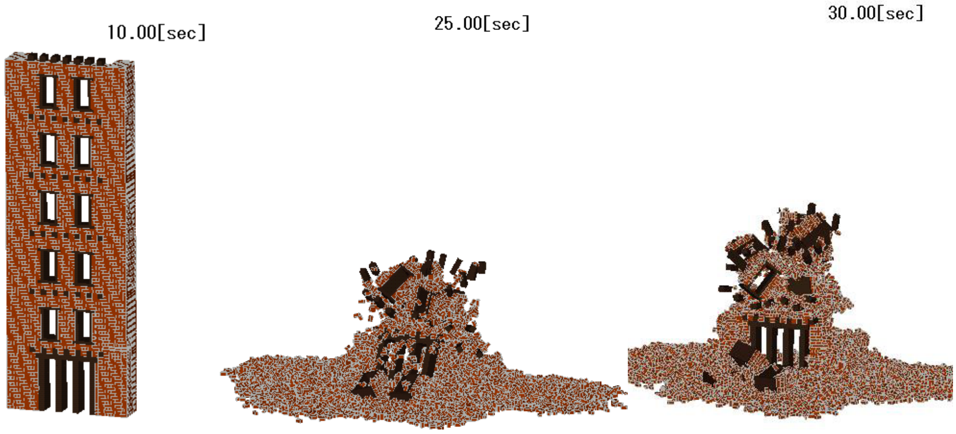


(a)40% (b)10% (c) 5%

Fig. 11 Seismic behavior of 4-story masonry building

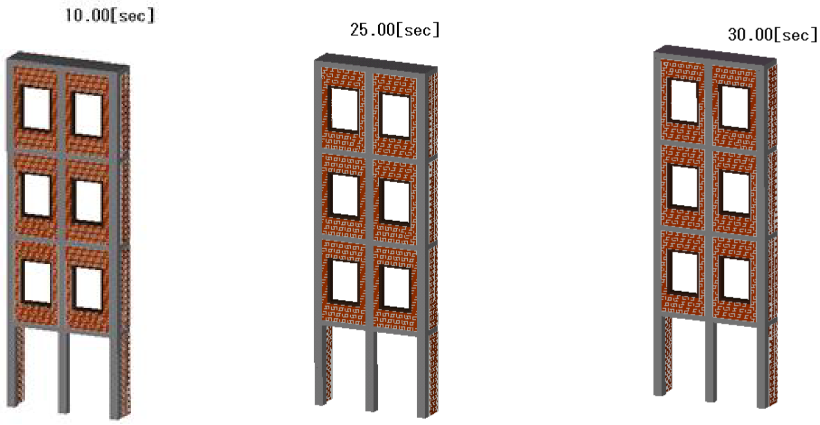
(a)40% (b)10% (c) 5%

Fig. 12 Seismic behavior of 5-story masonry building



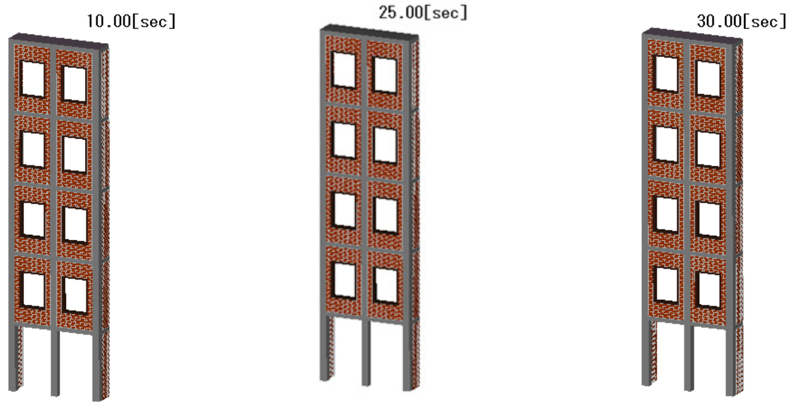
(a)40% (b)10% (c) 5%

Fig. 13 Seismic behavior of 6-story masonry building



(a)40% (b)10% (c) 5%

Fig. 14 Seismic behavior of 4-story confined masonry building

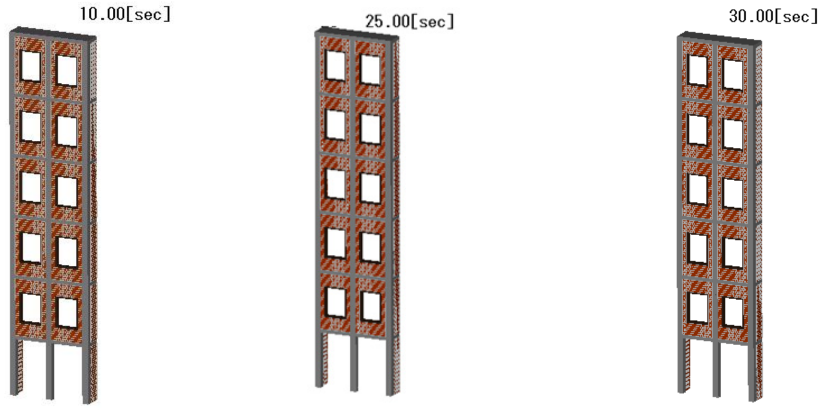


(a)40% (b)10% (c) 5%

Fig. 15 Seismic behavior of 5-story confined masonry building

By comparing the damage obtained by the DEM with the damage pattern in Fig.9, the damage index is visually evaluated. The damage index for 7 classified models are summarized in Table 4.

Based on the simulation, the damage index for three input ground motions are mapped in Fig. 17. The shaded buildings are 1-story buildings. From Fig. 17(a), the maximum damage index is 0.2 for the ground motion with occurrence probability of 40% in 50 years. The buildings in the target area are considered to have the strength to survive this scale of earthquake.



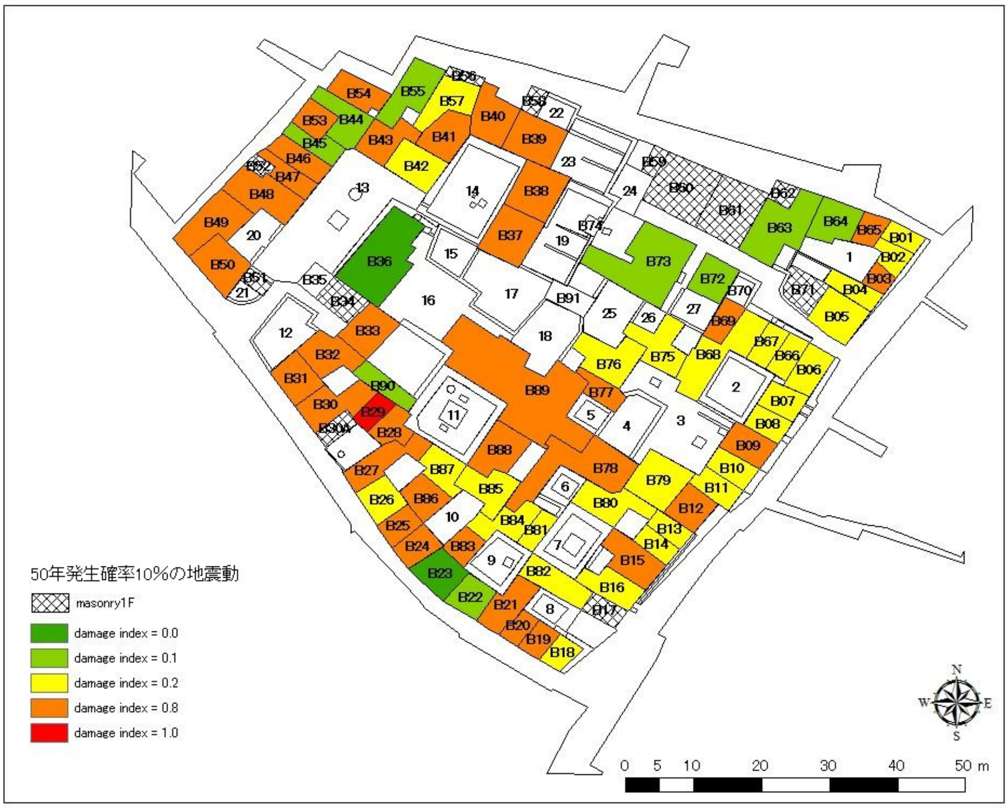
(a)40% (b)10% (c) 5%

Fig. 16 Seismic behavior of 6-story confined masonry building

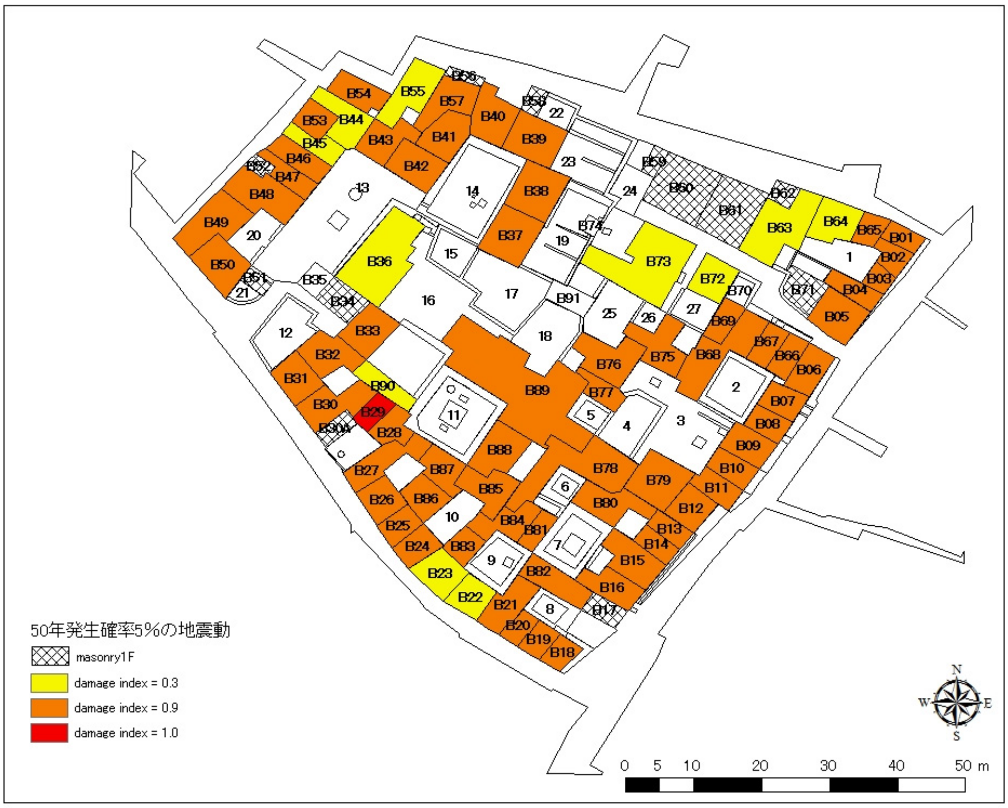
Table 4 Damage index

Damage index	Occurrence probability in 50 year		
	40%	10%	5%
Masonry 3F	0.2	0.8	0.8
Masonry 4F	0.2	0.8	0.9
Masonry 5F	0.2	0.8	0.9
Masonry 6F	0.2	1.0	1.0
Confined masonry (RCC) 4F	0.0	0.0	0.3
Confined masonry (RCC) 5F	0.0	0.1	0.3
Confined masonry (RCC) 6F	0.0	0.2	0.4

(a)Occurrence probability of 40% in 50 years



(b)Occurrence probability of 10% in 50 years



(c)Occurrence probability of 5% in 50 years

Fig. 17 Estimated damage index for target area

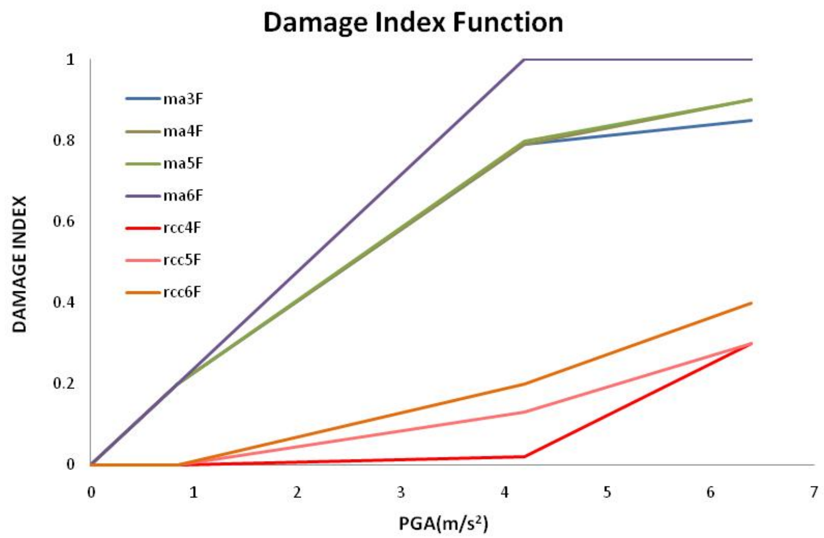


Fig. 18 Damage index function

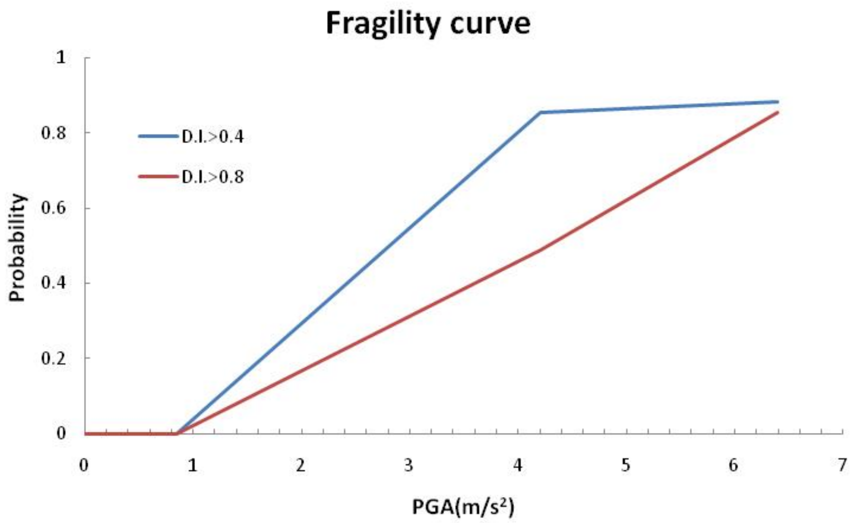


Fig.19 Fragility curve

b) Damage index function

From Table 4, the damage index function is estimated. The definition of damage index function in this study is the relationship between PGA (peak ground acceleration) and the damage index.

Fig.18 is an estimated damage index function for 7 classified buildings. The buildings with larger damage index have less strength against earthquakes. Line-up of buildings in order of increasing strength becomes masonry 6F < masonry 5F < masonry 4F < masonry 3F << confined masonry 6F < confined masonry 5F < confined masonry 4F. The most weak building is the 6-story masonry buildings. Confined masonry buildings have much strength than the masonry buildings. From this, the effectiveness of reinforcement using reinforcement concrete is clear.

c) Fragility curve

The fragility curve for all buildings except one-story buildings are also computed. The definition of fragility curve in this study is the relationship between PGA and the probability which exceeds each damage index. The probability is obtained by dividing the number of buildings whose damage index exceeds a certain value by the number of total buildings assumed.

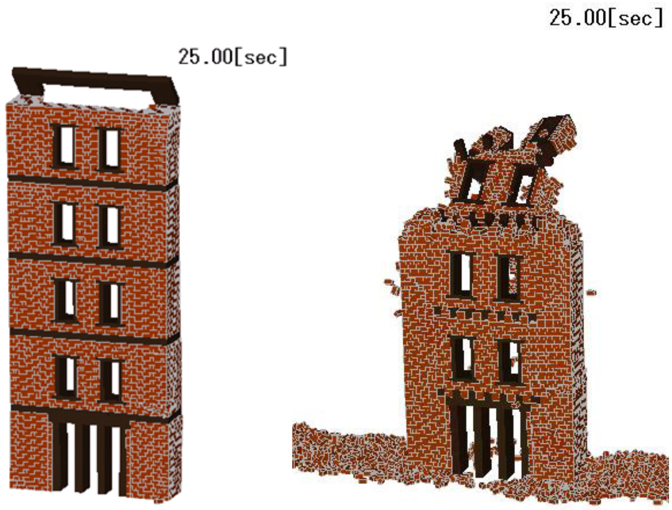
Fig.19 indicates fragility curves for DI>0.4 and DI>0.8.

About 85% of the buildings suffered damage (DI>0.4) at 420gal, and about 88.2% of the buildings suffered damage (DI>0.4) at 630gal. About 48.7% of the buildings suffered damage (DI>0.8) at 420gal, and about 85.5% of the buildings suffered damage (DI>0.8) at 630gal.

d) Reinforcement using ring beams

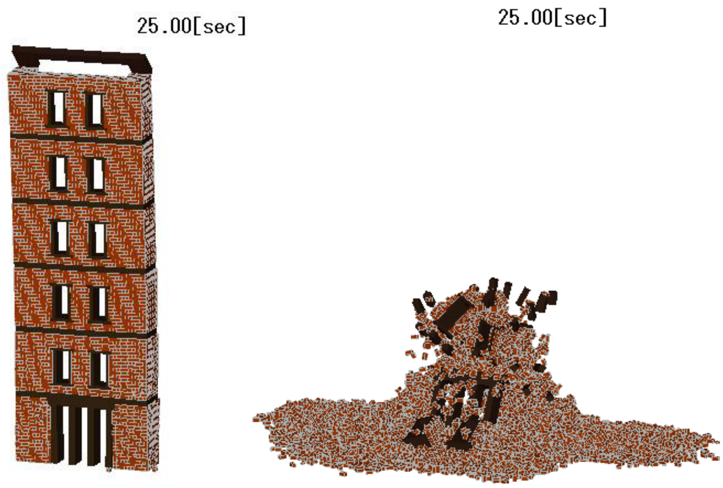
The effectiveness of reinforcement using timber ring beams is examined. By using decorated timber beams, the reinforcement is possible without reducing their historical values. The timber ring beams are placed on top of each floor to unite bricks in the horizontal direction.

Figs.20 and 21 compare the damage to the buildings with and without ring beams for 5 and 6-story masonry buildings when ground motion with the occurrence probability of 10% is input. The ring beams successfully avoided the building from collapse and the effectiveness is shown. However, for the ground motion with the occurrence probability of 5%, the ring beam could not avoid collapse. Therefore, this type of reinforcement is not effective for severe earthquake, and confined masonry is more stronger than the masonry with ring beams.



(a) with ring beams (b) without ring beams

Fig.20 Effect of introducing ring beams to 5-story masonry building (ground motion with occurrence probability of 10% in 50 years)



(a) with ring beams (b) without ring beams

Fig.21 Effect of introducing ring beams to 6-story masonry building (ground motion with occurrence probability of 10% in 50 years)

(5) Conclusions

The seismic risk of buildings in the Jhatapo area is evaluated by using the refined version of the DEM. The complete enumeration was done and the buildings are classified into 7 types with different building types and the number of stories. Only the façade is modeled for the analysis assuming that the vibration in the out-of-plane direction is dominant. The input ground motion with different occurrence probabilities are input in the out-of-plane direction of the models, and the structural damage is evaluated by using the damage index. It is found that the masonry buildings get severe damage but the confined masonry buildings using the reinforced concrete get less damage. As the number of stories increases, the structural damage becomes severer. The damage index function and the fragility curve for the Jhatapo area is estimated and the seismic risk for each building is shown on the map. The effectiveness of reinforcement using ring beams is also confirmed.

References

1) D'Ayala, D.: Correlation of fragility curves for vernacular building types: Houses in Lalitpur, Nepal and in Istanbul, Turkey. In: Proceedings of the 13th World Conference of Earthquake Engineering, Vancouver Canada, 2004.

2) Okada S, Takai N.: The basic framework for casualty modeling of victims of earthquake, (1) Classification of structural types and damage patterns of buildings, and damage index function, *Reports of the Tono Research Institute of Earthquake Science*, No. 2, pp.12-23 (in Japanese with English abstract), 1999.

3) Coburn, A.W.: Seismic vulnerability and risk reduction strategies for housing in eastern Turkey. Ph. D. Thesis, The Martin Centre for Architectural and Urban Studies, University of Cambridge, United Kingdom, 1987.

4) Grünthal, G. (ed.): European Macroseismic Scale 1998 (EMS-98), http://www.gfz-potsdam.de/portal/gfz/Struktur/Departments /Department+2/sec26/resources/documents/PDF/EMS-98_Original_englisch_pdf, 1998.

4. Analysis of Heritage Values, Vulnerabilities and Resources for Emergency Response

4-1 Reading Heritage Values of the Historic City of Patan in Kathmandu Valley

Sudarshan Raj Tiwari

4-1.1 Reading Urban Cultural Heritage of Patan

Patan Durbar Square Monument Zone (PMZ) is one of the seven monument zones of the Kathmandu Valley World Heritage Site (KVWHS). The KVWHS was inscribed in 1979 and its outstanding universal value (OUV) was assessed under criteria (iii), (iv) and (vi), clause 77, Operational Guidelines for the Implementation of the World Heritage Convention. These criteria note the attributes of KVWHS as outstanding architectural and technological ensemble of significance to human history (iv), an exceptional testimony to a living cultural tradition (iii) and a tangible artistic work of outstanding universal significance (vi). Since this is an urban cultural heritage, the reading of values of this heritage is not limited to its monuments but extended to the ensemble of building components, buildings, streets and other networks, crossings and urban spaces and the city. Also as it is noted as a living heritage, this study seeks to read livingness value along with heritage values in the elements, networks and spaces. To meet the objective of disaster mitigation planning particularly in rescue stage, a further reading of resource/ecological values in these aspects for access, space and supplies prospects is attempted. As per UNESCO's operational guidelines, heritage values

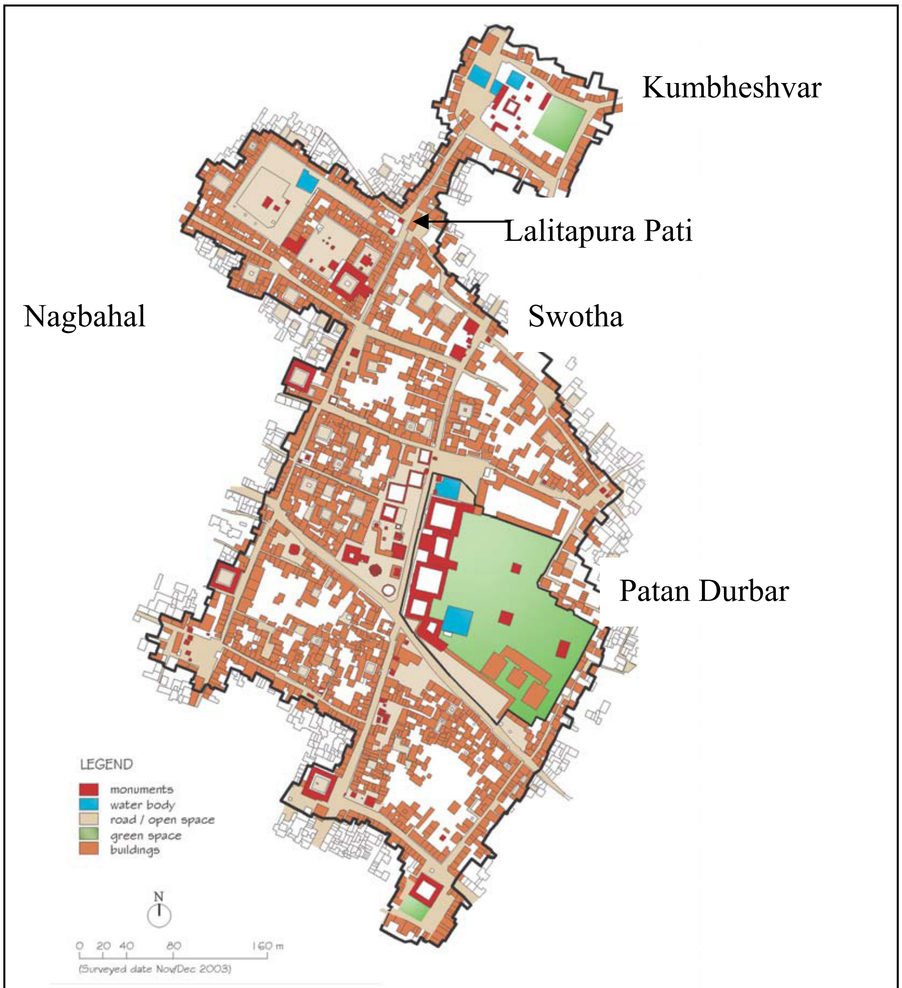


Fig. 1 Patan Monuments Zone of KV World Heritage Site

are read in terms of the indicators of historicity, architecture and artistry, authenticity and integrity, technology and ecology. Further values are in the characteristics that make and expound it as the recognized stage of development of human and Nepalese history, which in the case of PMZ is largely related to the later Malla period.

Ecological values associated with sensitive human interaction with environment, particularly the relation of settlement with the ground water and local aquifer as exhibited by the water works about Kumbheshvar, are worthy of note and have been taken as important resource values.

The key objective of this study for disaster risk mitigation of the said property is to manage vulnerabilities so as to preserve and continue the values, their authenticity and integrity mainly over earthquake and fire hazard events alongside the mitigation of disaster from the perspective of the human life and society and building structure.

Although it is usual to classify the cultural heritage in generic terms into tangibles (monuments, spaces, windows, musical instruments, handicrafts etc.) and intangibles (language, music, songs etc.), for urban cultural heritage in general and KVWHS in particular as the listing criteria specifically recognizes its exceptional testimony to a living culture, it would be important to seek its living linkages to people, their social organization (family, extended family, clan, neighborhood and citizenry) and cultural practices. Since tangible heritage assume a dominant position from the perspective of disaster mitigation, the consideration of intangible heritage are made within the study of values and vulnerabilities through the measure of their 'livingness'. The measure of values for the urban heritage has been considered in terms of value indicators (such as history, architecture, artistry, representation of a period, social symbolism, ecology) as weighed by the status of the heritage (studied in terms of authenticity, completeness, integrity and associated elements). Likewise measure of vulnerabilities are based on probability and propensity to damage and loss in the event of the hazard factored as the effect of the damage or collapse of the structure of the heritage building as well as effect of collapse of neighboring or associated objects and spaces and possibilities of recovery such as knowledge and skill availability for reconstruction etc. The classification uses a combination of social and typological differentiation criteria (e.g. temple, *pati*, residential building, *baha*, etc) and gives values to them depending upon their significance to the value indicators as well as the social units. The heritage of water (such as expressed in *hiti*, wells and ponds and their user social groups) and planning (such as streets, underpasses to courts and *nani* and open spaces of courtyards and crossings) has been considered as a being additionally and specifically significant as having resource value from disaster mitigation planning. Heritage of water is treated under a separate heading 4-1.3 below. The following description of the values of the city, built elements, networks and spaces making and constituting the urban heritage are specifically related to the study area of this research e.g. Jhatapol, Konti and Ikhache quarter at the north-western edge of Patan and generally to historic town of Patan and the Kathmandu Valley World Heritage Site.

(1) History, legends and symbolic values of the city of Patan

It is traditional in Nepal to attribute a fair share of symbolic values to the cultural heritage and these are particularly derived from association with legends of happenings of the past.

Known as 'Yala' to the Newar, it is believed that the city of Patan was founded by King Birdeva in 6th century and named Lalitpur in memory of a farmer named Lalita, whose leprous state was cured by the magical spring of Kumbheshvar. According to a popular legend, the king laid out the city of Lalitpur around the spring in the auspicious pattern of a *yantra* with nine jewels in the center, eight protective deities on the periphery (Astamatrika) and twenty four neighborhoods as instructed by Lord Kumbheshvar 1). The Lalitapur Pati and the upright stones of Jhatapol (which literally means 'wet land' in Newar language) keep memory of the myth alive. The origin of the town of Patan is shrouded in such and other legends and tales. Buddhist legends relate that the town was initially given a formal plan based on Dharma-Chakra, the Buddhist 'wheel of righteousness', by the Mauryan Emperor Ashok. The main arterial streets of Patan link the two pairs of Stupa (east-west and north-south) and intersect at the Durbar Square echoing the Dharma-Chakra layout. The Patan Durbar Monuments Zone of Kathmandu Valley World Heritage Site is not only the hub of Dharma-Chakra of this legend, it also houses the nine jewels that form the center of Lalitpur according to the Lalitapur legend - of these 'mani' jewels, Mani-hiti, Mani-Keshavanarayana and Mani-Mandap find place in the heritage list. The people of Patan find great value in establishing historicity through such legends. Many more of such legends, told and retold woven with nuances, to highlight events that suit each of the variety of faiths that share the urban space and the preferred paths of cultural development they love to retell, all add value to the spirit of the place. Legend of Matsendranath (Karunamaya), the founding of Ta-baha and the associated annual and twelve early chariot festivities or the legend of King Sarvananda and the founding of Guita Bahi make the place alive for the people. Even as many of such episodes may present a skewed sense of historical time when viewed against the 'established history', mythologies remain defining realities of indigenous cultural history. Hiranyaverna Mahabihar, Guita Bahi, Oku Bahal and the Mahaboudda temple have thus acquired greater historical, cultural and religious values among the heritage associated with Buddhist development of Patan.

Despite of difficulties of tying established history and mythological histories, one would rightly assert that 'Patan area was almost certainly settled in the Kirata period'. It was already a collection of vibrant settlements and activities several centuries before the earliest cultural marker of Gajalaxmi was left for posterity by the Lichchhavis at Chyasal-hiti. Popular folk memory in Patan is that the Kirata and the Lichchhavi fought their final war at Chyasal! Likewise Patuko Don and Patuko Dyochhe in Patan Durbar Square show legendary link to Kirata days. In the area under study, legends claim as early a link for the site of Pimbahal and its Stupa.

The recurrence of important role of farmers in the myths of Patan, whether related to its establishment or its progress in history, can be attributed to its agricultural origins. Irrigation water works such as reservoirs, canals and distribution outlets and their urban services counterparts of ponds and pit water conduits have formed important component of tangible cultural heritage of Kathmandu Valley towns and Patan in particular. The development of urban water supply system by as early as fifth century AD is unique to Kathmandu Valley. In the Patan Durbar Square Monuments Zone, the landmarks of Mani-hiti (inscription from 6th century AD), the Bhandarkhal pond, the Kumbheshvar-hiti, the wells and ponds of Kumbheshvar and Nagbaha-hiti tell the history of this development of urban water supply and irrigation services. The values of the water system development may be read in terms of historical, technological, aesthetic, as well as anthropological significance. These are elaborated separately under 4-1.3 and its subheading (3) below.

(2) Built Heritage Elements:

The traditional architecture of the ensemble of the multi-tiered temples, *chaitya*, palaces, *baha*, *bahi*, *sattal* and *pati*, the magnificent application art and craft of brickwork and woodwork in them, the design, proportioning and the human scale of the buildings and the resulting mesmerism of the urban landscape transcend the local living cultural value into the outstanding universal value of aesthetic, technological and ethnographic significance. An extensive inventory of the built heritage of Kathmandu Valley has been published in Kathmandu Valley – The Preservation of Physical Environment and Cultural Heritage – A Protective Inventory. It has categorized them into different classes (from class one to four) using indicators of historical-cultural antiquity, authenticity, artistic quality and architecture and uniqueness within the ensemble 2). Similarly the list of built heritage listed in the Kathmandu Valley World Heritage Site is available in publications of Department of Archeology. The following details are given only for some specific elements of concern to the current study and the study area.

a) Temples:

The public square in front of the Patan palace, the Patan Durbar Square is designated as one of the seven monument zones of KVWHS. The temples and palace courts of Patan Durbar Square, largely built in the sixteenth and seventeenth century AD, represent excellence of Malla period architecture. This square has a number of temples of national/town level significance such as the Charnarayan temple, the Krishna Mandir, the Bhimsen temple and the Viswanath temple. The oldest temple structure in Patan Durbar Square, the temple of Charnarayan was built in year 1566 AD. Kumbheshvar temple complex was designated into the PMZ because of high historical, symbolic, architectural and artistic value. As it is a major center of religious and ritual activities and pilgrimage, it holds great significance as a living heritage. The temple of Kumbheshvar is one of the only two five tiered temple of Nepal and the artistry of woodworks in its struts are remarkable and possibly some of them are original (from fourteenth century?). The antiquity of Kumbheshvar temple complex is reinforced by the fact that the temple precinct has the oldest medieval record of reconstruction for a tiered temple (1392AD) following the vandalism of the original temple by Sultan Samasuddin in 1349. Apart from the Kumbheshvar temple, the complex also has the temple of Bagalamukhi, temple of Vishnu, a temple with a well (which offers it an ecological value) and a number of smaller temples and images. A number of important deep pit conduits (*hiti*), a large ceremonial water tank and a garden space are other elements of import in the complex.

The Stupa and pond of Pimbaha, a little to the south of the study area is an ancient monument ensemble that was reconstructed in 1360 AD after it was also similarly vandalized at the hands of Samasuddin. Besides the pond on its north is situated the temple of Chandeswori built in 1663. Also in the Sulima square, a small but beautiful tiered temple of Siva Ratneshvor is located. All these buildings of religious usage have legendary and ecological association with the study area. The pond and the open spaces appear to play significant recharge role to the ground water resources of the Jhatapol study area through traditional system passing through Nhykachuka and Nagabahal. The palace square and the open spaces, like Nagabahal, could also play significant role as secondary spaces in disaster mitigation planning.

In every *Janaipurne* day (which fell on 13^{th} August in 2011), the people of the whole valley come in pilgrimage to the festival of Siva that is celebrated in the water tank of Kumbhesvar. Every

Saturday and Thursday, the shrine of Bagalamukhi in the Kumbheshvar temple complex comes alive with many devout visitors that come to offer worship, some with lighting of a one-hundred-thousand wick lights (*lakhbatti*). The temples and spaces of the Durbar Square also come alive with many festivals of national/town level such as the Chariot Festival of Machchhendranath, the procession of Krishna, the Mataya festival etc. It also becomes a center of pilgrimage for the whole valley during Krishna Astami days every year (21^{st} August in 2011). Kartik *Nach*, a classical dance drama, is performed in the space in front of Charnarayan temple with much cultural fervor in the lunar month of Kartik also.

b) Baha-Bahi:

There are no Buddist monasteries in the pilot study area. To the west on the other side of the street bordering the study site at Ikhachen Tole is located the Ikhachen Baha which has a totally reconstructed shrine of the Kwapadyo, the *chaitya* and the *dharmadhatu mandala*. Only the courtyard space lives while all the other three wings have been converted into slender five or six storey tall concrete housing blocks. Only the *chaitya* and the image of the Kwapadyo remain as heritage elements in Duntu Bahi also at Ikhachen Tole. The use of foreign materials and technology in reconstruction of the Kwapadyo shrine and the front wings have led to a significant loss of its heritage value. Next door, Pintu Bahi, an ancient foundation, has its original Bahi structure although in state of great disrepair. It has significant historical and architectural typological value. Its square court has a *chaitya* and a *mandala* as heritage resource. The chaitya, mandala and other images in the enclosure of Yochaken Baha are another set of heritage resources. Konti Cidhangu Bahi at Yokhachen Tole survives as an enclosed space with a chaitya at Konti Tole to the north of Konti Bahi. Konti Bahi is a well preserved *bahi* building and the courtyard has a Lichchhavi chaitya. A large chaitya situated in an open space to the east of Kumbheshvar complex is what remains of Konti Baha. To the south of the study area, the very important monastery of the Golden temple (Hiranyavarna Mahavihara) and Nagabahal is located. Hirnayaverna Mahavihara has significant historical, symbolic, artistic and socio-religious values. It's three-roofed Kwapadyo shrine as well as the shrine housing the Lichchhavi chaitya in the center of the courtyard are unique and exhibit marvelous artistry and craftsmanship. Being a monastery with a very large following, Hiranyavarna Mahavihara is unique in that it is a monastery for both Bajracharya and Sakya *sanga.* It is a very active living monastery.

c) Sattal:

Sattal is a two storied public rest house of history usually located in neighborhood squares formed at key crossing or on pathways. The sattal at the Jhatapol tole is of neighborhood level significance as indicated by its use in tandem with the Dabu, which is one of the 24 neighborhood Dabu of Patan 3). Making the entrance building to the Kumbheshvar complex is a large sattal with the excellent woodworks in the columns of the *dalan* space in ground floor and a long single Gajhya stretching all across the length of the building in first floor. It is used during large group worship and other community and *guthi* ritual functions.

d) Pati:

The *pati* (B51) located at the entry to courtyard 13 is in traditional form and is used by the *si-guthi* of the Maharjans living in the courtyard. The large number of small religious images installed in the row of niches in the walls carries the ancestral memory of the associated families. The newly constructed two storied community structure is a replacement of a *pati* used for singing *Dafa bhajan* – religious songs. It only retains a symbolic value to the Maharjan community. The upper storey is used by the *si-guthi.* The *pati* at Jhatapol crossing, although simple, retains its original form and is used as neighborhood activity logistics storage space. There are a number of *pati* of average architectural significance in and about the Kumbheshvar complex. area.

e) Chiba and Others:

Apart from the miniature chaitya in each of the *baha* and *bahi* noted under b) above, there are four *Chiba* in the study area. The *chiba* in Courtyard 1 is very well preserved and has excellent artistic quality. It carries an inscription and is over two hundred years old. Another *Chiba* in the nearby courtyard 3 is also of similar artistic and antique values. It has a inscribed stele attached to a side wall. Both the *Chiba* have formed part of the *Mataya* festival circuit of Patan since history. The *Chiba* in the courtyard 13 sited on a recently restored plinth structure in brick and stone has an above average artistic and antique significance. Although its association is with the immediate Maharjan community, like with all Buddhist religious entities in Patan, it is included in the town level circumambulatory procession of the *Mataya.* The *Chiba* in Chhenko courtyard 11 displays good aesthetic qualities but its restoration has not been done as sensitively as courtyard 13 and caused loss of its antique value. It is also visited during the *Mataya* festival 3). A temple of Krishna located in courtyard 19 is a small temple in Rana architectural tradition and has a family level association with the Amatya of the court. The small Narayan shrine and the *Math* located in courtyard 14 is of relevance to Talache house of the Amatya family. The artistic quality of images in both the courtyards of 14 and 19 are average.

f) Residential Buildings:

Residential buildings both as a street side and courtyard design make a significant component of the ensemble of urban heritage of Kathmandu Valley. It is also the type where the 'loss' of heritage has been extensive as the house accommodates the needs of living of the present generation as it changed to met the needs of the previous generations. In the study area, there is no residential building that totally and truly represents the architectural, aesthetic and technological achievement of the Malla period. Thus architectural value has been read as an aggregated state considering originality, integrity of structure, integrity of façade, elements of the component units such as roof, strut, wall, window, door and *dalan*, artistic embellishments and socio-religious expressions (sculptures and paintings on the exterior). The measure of heritage value of the residential buildings in the study area has been presented in detail under 4-2 below. The aggregated values show high heritage significance of B13, B14, B15, B16, B46, B48, B50, B81, B82 (see GIS plan, 4-2 Fig.9).

g) Inscriptions:

Tradition of leaving an inscription as a record of public activity has a long history in Nepal. The earliest inscriptions in Kathmandu valley date to the Lichchhavi society and start in profusion from the middle of the fifth century and they are on stone stele. These have great heritage value as a record as well as a historic object that gives a sense of time/history and relates the donor-builder and the users in the past and the present. The heritage value of social and cultural linkage that the inscriptions establish between the people in the past and the people at present through clan association and ancestral identity building is immense and present society is waking up to it. In the study area, we find several inscriptions in stone that establish local family, clan and neighborhood centered socio-cultural identity while providing significant historical records of acts, ritual activities and social institutions (such as *Guthi* – a traditional trust) specific to heritage elements, space and people. The location of these have been identified in the plan, Fig 2)

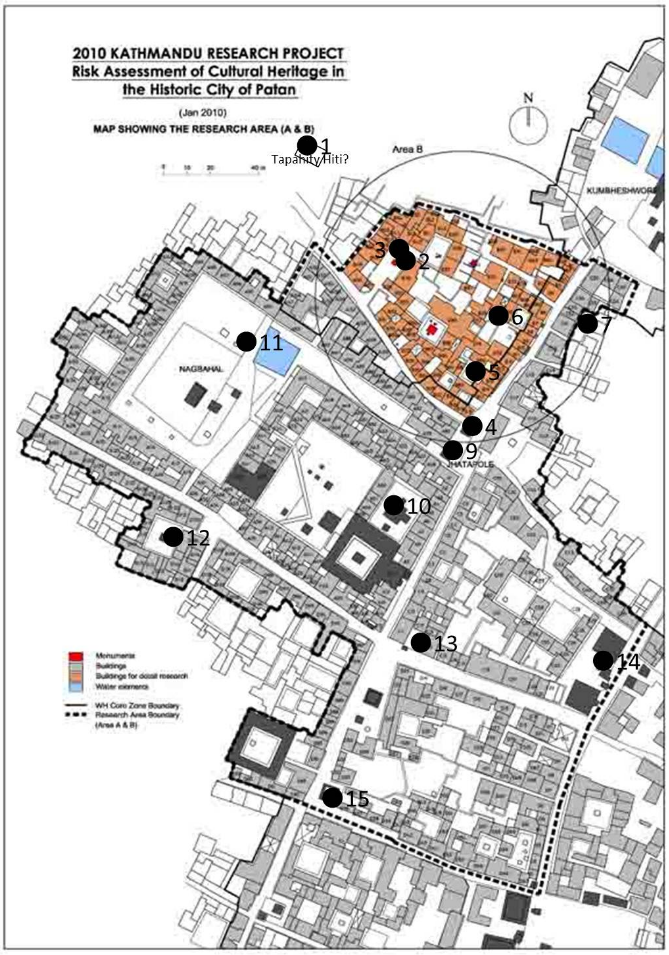


Fig.2 The location of Inscriptions

(3) Streets

Although, Patan's urban cultural heritage is dominated by the architectural monuments discussed above, value should be given to the urban spaces of the streets and the squares that provide the sites for the buildings and link them through varied festivities the monuments and their cultural occupants to the town and its residents. Festivals, the way they are held and managed by the community, tell a lot about open space management, apart from living out the seasonal play of gods to enliven, purify, renew and refresh, the spiritual and the mundane structure of the town.

a) Main Street:

The street bounding the study area to the east is part of the Mahapal-Konti high street of the Malla period organization of the city patterned as a 'nine-square *mandala* plan' with two parallel streets running north-south and two running east-west. For this reason, some city level processional festivals (like Ganesh and Bhimsen *jatra*) take place along the stretch of street between Jhatapol and Konti. The location of two important Chibaha in the area in courtyard 3 and 7 brings the Mataya festival circuit onto this street. Mataya circuit also links to the Chibaha in courtyard 11 and 13 and puts ritual life to the side streets on south and west the site.

b) Alleyways and Underpass:

These narrow lanes passing under residential building are valued as important urban planning tradition, offering both cultural and resource value and creating vulnerability all at once. The underpass of B28 and B9, which respectively lead to courtyard 11 and 3 are specifically of cultural note in the study area.

(4) Squares

Cultural practices that take place in spaces in and around the monuments, at crossings and its environment and link the heritage elements through pathways to the society are of specific value as they bring 'livingness' or contemporary life into traditions. Living heritages, thus, are unique combinations of tangible and intangible heritages that bring the present time and the present generation into play. Identity value and belongingness generated by the various traditional rituals and festivals that take place along the main street and bring the whole towns' population of Patan in cultural processions is significant. Such festivals bringing people in *pradakshina* or going around town travel through the north-south high street along Jhatapol pati in the study area and ritual festive interactions take place at the crossings as the two of the twenty four historical *dabali* of Patan, the ones at Jhatapol and Konti crossings fall in the study area. Likewise, the Mataya festival (in which a ceremonial circumambulatory procession is performed to each and every Stupa, Chaitya, the Buddhist monasteries of Baha and Bahi and the miniature Chibaha shrines that dot Patan) brings the Buddhist population of Patan to the study area to go round the Chibaha, Chaitya, Baha and Bahi located in the study area. The public square at the entry to Kumbheshvar complex becomes a scene of many a festive gatherings at specific days of lunar calendar. The large courtyards (numbered 13 and 14) are public at clan level and offer resource value as a primary rescue space.

(5) Society

Conservation, maintenance and operation of heritage monuments and associated socio-cultural activities, services and festivals, the institution of *Guthi* and its agriculture land-based financing, neighborhood and clan based organization and seasonal initiation actions, are themselves cultural heritages. The society organized as family, extended family and clan units and integrated in the community scale at clan, neighborhood and city/nation level is expressed in the streets and squares and associated cultural activities there in. Maharjan form the major *jaat* group in the study area.

4-1.2 Reading Traditional Urban Water Systems

Fig.3 A Restored Hiti in Nagbahal.

Buildings and structures of godly and royal/imperial importance and their ruins and remains dominate the built heritage passed on to posterity by most ancient civilizations; utilitarian structures are in general rare. Among such rarities may be listed the Great Bath of Moenjodaro, the Roman Aqueduct and the pit water conduits of Kathmandu Valley. But, with the glitter and charm of multiple-roofed temples, Mahachaitya and palaces of Kathmandu valley drawing the viewer's eye skywards, these ancient and unique pit conduits of as great architecture, artistry and technology are often missed and go unsung. Yet the heritage value of these monumental utilities should be perceived as even more since these were constructed for the use of the common man. The ancient urban water supply system consisting of ponds, canals, wells and artistic pit conduits is an urban utility heritage that has few equals in the world and still retains its utility value. For a building culture, which makes such a profuse use of wood in structural as well as architectural roles and whose monuments have been lost more often to fires than to earthquakes in history, such heritages have significant resource value from the perspective of fire-fighting.

Traditional Water Supply System of Kathmandu Valley has aroused contemporary interest primarily for its heritage value based on the artistry, craft and architecture of the stone conduit placed in deep terraced pits in fair numbers at road intersections and public places of the traditional settlements. These terraced sunken pit courts with conduits, locally called Hiti or Dhungedhara, are a class of monuments from Lichchhavi era, undoubtedly dated from as early as fifth century and possibly evolving out of the practices and traditions of the Kirata era half a millennium earlier, are as much a part and parcel of heritage of its unique historic urbanism and urban development as are the temples, Chaityas, palaces and residential neighborhoods popularly perceived to form the Kathmandu Valley World Heritage Site as listed in UNESCO. A recent inventory study recorded 400 Hiti in the valley, including 47 sites where the pits have been lost by filling in recent years.

Their rapid loss in the wake of recent urban development and the specter of water shortage and failing modern water supply systems have precipitated concern not just from heritage perspective but also from the urban services context. Since quite a section of the residents of the urban core have come to depend, for their daily needs, on water dispensed by these heritage utility outlets, added public concern has centered on the potable quality of water as well as on protection and preservation of the sources from pollution. Since such issues have propelled the available studies of traditional water supply system, the investigations have looked beyond the artistry and craft on to the architecture and engineering of sources, canals, reservoirs, outlet pits and their drainage. For some particular Dhungedhara which have been harnessed for collection and distribution of water through addition of modern storage and piping system at the tail of the traditional system, even the system 'regulators and filters' used in them have been opened and studied.

(1) History and Historicity:

The Hiti water structures are part of an elaborate urban water supply system that consisted of ponds (*pokhari*, *pukhu*), the terraced and sunken pit courts with conduits (*pranali*, Dhungedhara, Hiti), brick lined dug wells (*tu*, *inar*) and drinking spouts (*jaladroni*, *jadhu*) generously distributed in the settlements. The water to these system outlets were brought through canals (*tilamaka*, Rajkulo, *dehdha*) that reached out along a ridge from the urban reservoirs to its sources in the foothills of the valley, often with large intermediate storage ponds (*udaka*).

Of course, as an service utility of a living urban culture, much of the elements and the system have undergone continuous development over the centuries and if it was not for the Lichchhavi practice of leaving inscription in the stone conduit and the use of very durable stone, which has survived more than fifteen hundred years of use without needing a replacement, the antiquity of Hiti system would have been difficult to substantiate.

With an inscribed stone that describes its construction for public use and dating it to 550 CE, the sunken water supply pit system of Satyanarayan at Handigaun, Kathmandu, is the oldest Hiti extant in the Valley and comes from the Lichchhavi period. It was certainly not the first of its kind to be built and the Hiti and other associated elements making the utility structure were already in use for some time by then. We find inscriptions mentioning the networks and outlets for water supply and irrigation. For example, water canal (*paniyamarga*) is mentioned in an inscription of King Basantadeva located at Thankot and dated to 507 AD. The *jadhu* - a stone bowl with spigot for dispensing drinking water to passers by and devotee pilgrims to the religious complex, now located in Khapinchhe to the north-east of Patan Durbar, carries an inscription dated 530 AD. The Satyanarayan inscription records construction of water supply system as well as the objective water quality eg. testy, cool and clear water. An inscribed stone, located in Manihiti of Patan Durbar dated 571 AD mentions the pious deed of Bharavi, the same donor who had constructed the Handigaun conduit.

Water supply to Kirat ridge-top settlements was based on reservoir ponds located at a higher section of the settlement much like the ponds of the Malla period such as Rani Pokhari of Kathmandu, Siddhi Pokhari of Bhaktapur, etc. 4). In their early days, these reservoir ponds more mostly either rain-fed or spring sourced with a few others fed by canals bringing water from natural sources away from the settlements. The canals, called Tilamaka in Lichchhavi inscriptions, were also used to carry water to the distribution outlets in the sunken pit courts. The preferential use of the Kirat term Tilamaka instead of their Sanskrit equivalents such as *paniya marga* or *pranali-jaladroni* in Lichchhavi inscriptions would strongly support the inference that the system of canal and the canal fed conduit predates the arrival of the Lichchhavi in the Valley as much as the reservoir pond itself. The Newar term Hiti itself is derived from the Kirat terminology of Tilamaka 4).

Fig.4 The Chaitya behind Manihiti of Patan possibly sitting over the Hiti's filter and regulator bed

Management Trusts set up by the state for distribution of drinking water in public gatherings and festivities (*paniya gosthika*) and for maintenance of water systems (*pranali gosthika*) are noted in the 605 AD inscription at Lele, the area where we find the intake of Patan's traditional water supply and irrigation system. Another inscription of Sivadev (rule ca. 590-605 AD) located at Chapagaun, where we find the extant path of the canal, speaks of extensive fishery, possibly using ponds. A successful aquaculture is noted by Amshuverma in his 605 AD inscription at Bungamati, the site where the temple of Machhendranath is located.

Later Lichchhavi inscriptions provide evidence that this water work was extended and maintained several times. An in-situ stone slab inscription, with a image of fish in relief decorating the top of the stele, situated at the crossing of Mamadugalli lane and Mahapal-Konti street west of Patan Durbar, addressed to the people of that area (the then areas of Thambu, Gangul and Mulabatika) speaks of the restoration of a *Tilamaka* canal of earlier construction by the then ruler Jishnugupta in 624 AD. This canal appears later referred to in the Malla period as Lunkhusi, meaning a golden river as the canal water carried gold dust washed down from the roofs of many temples lining the main street there then.

Another stone slab inscription dated 726 AD, located in the pit courtyard of a water conduit near the temple of Minnath, close to Lagankhel, to the south of Patan Durbar, renews the instructions for administration and sharing of water from a *Tilamaka* canal into seven parts and among four localities. Thus we can see the large water-works associated with the festival of Matsendranath, both for irrigation and drinking water, were installed at various times in the middle Lichchhavi period. It had survived well up till recent times as Rajkulo and has gone into disuse and destruction only in last three decades.

(2) Elements of the urban water supply system:

It can be observed that the main canal of the extant Rajkulo had intakes at Lele Khola and Naldu Khola. The canal splits into several branches at Nakhipote leading to various reservoir ponds and shallow aquifers, which in turn charge and supply water to ponds, Hiti and wells generously distributed in the town of Patan with an astute consideration of the topography, geology, shallow aquifers and ground water table. During the Lichchhavi period itself, water as a urban utility appear to have undergone significant development and refinement in system, architecture and decor. And it is a tribute to their technological genius that the three natural elements of pond, river and spring were adapted into reservoir, canal and conduit and combined into the Hiti system.

a) Ponds:

The main reservoir ponds of Patan were located at Lagankhel and its ritual linkage with the Nagas associated with the festival of Rato Machchhendranath led them to be named Sapta-patal-pokhari. Remains of the system survive in parts at Lagenkhel area. Closer to the study area, we find the Pimbaha pond, La pukhu and Kamal pokhari forming intermediate reservoirs and recharge ponds of the larger system.

b) Well:

Wells are generously distributed in the living neighborhoods of Patan and served as service outlets in homogeneous social neighborhood. These were fed by ground recharged by ponds, shallow aquifers or natural water carrying veins and layers. Natural black cotton soil layers appear used to guide charge to required areas. Several wells are found in the study area. Wells have been noted in the courtyards 3, 11, 13 and 16 and the well of courtyard 13 displays an above average heritage value also.

c) Hiti:

The Hiti system used (i) canals to port water from the sources to local reservoirs and then on to conduits, (ii) regulating and filter mechanism, (iii) conduits with a small freefall for ease of use and 'ritual purity' in dispensing water and (iv) a deep channel to drain the pit. It displays historical, technological, architectural and artistic and cultural-anthropological values of as much significance as ecological and resource value. These are noted as distinct subheads below.

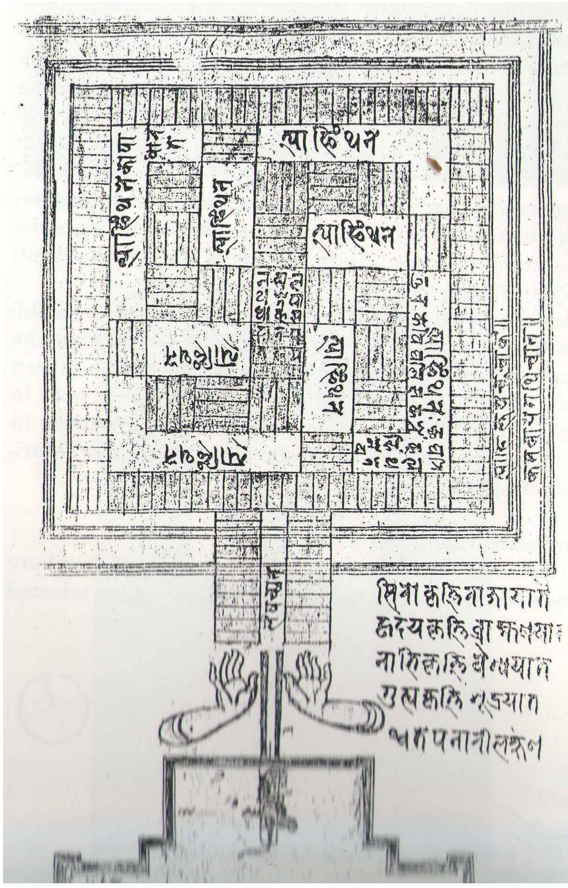


Fig.5 Manuscript drawing showing filter and regulating bed of Hiti system (Becher-Ritterspach, 1994).

(3) Water in Cultural Practices of Patan

A festive heritage of unique cultural and social significance for Kathmandu Valley in general and Patan in particular, is the Chariot festival of Machhendranath, a Jatra celebrated by both Buddhists and Hindus alike with great pomp and joy. This festival is as much a celebration of the god Avalokiteswora (for Buddhists) and Birinchinarayan (for Hindus) symbolized in the image of Machhendranath as of a large urban water supply cum irrigation system that has, since its creation, proved to be a lifeline for Patan town and its farming hinterland.

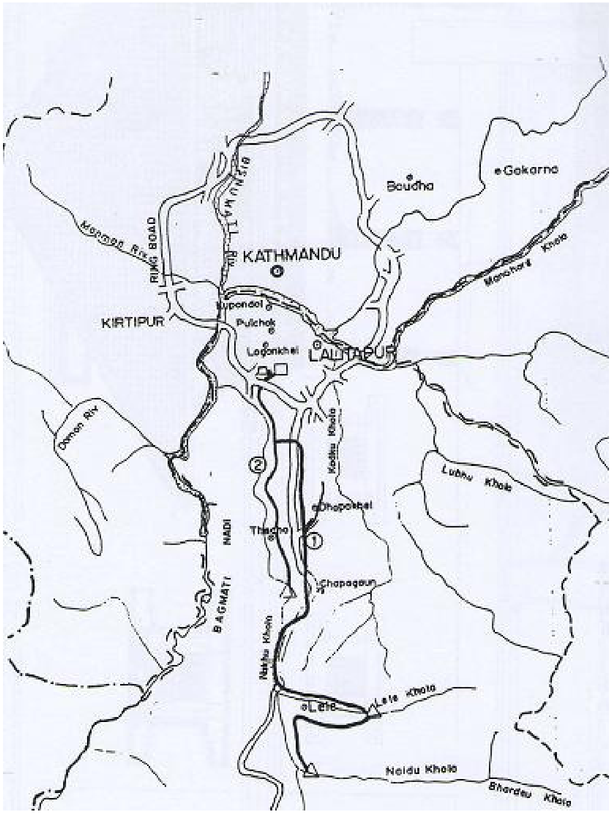


Fig.6 The Alignment of Rajkulo of Patan based on Extant Canal Sections and Ruins (Joshi,1993)

According to legends, an extensive festival of Sri Bungma Lokeswora, now popular as Rato Matsendranath, was initiated and popularized in the Lichchhavi period by King Narendradev following a failure of monsoon that had caused a long dry spell and famine then (Locke, 1975). The faithful were told that the reason for the dry spell was Guru Gorakhanath, who had imprisoned the Naga, the agents of rain. As a ritual solution Sri Lokeswora was to be brought to the valley. They were guided in this quest by serpent king Karkotak. As Sri Lokeswora arrived at Bungamati, Guru Gorakhnath got up to pay his respect to the new arrival and released the Naga, who instantly made rain and saved the valley. We can see that an extreme draught that adversely affected the agricultural output had invited a state intervention as a prelude to the festival. Although the legend is in divine weave, in reality, it was an extension and reconstruction of a large irrigation canal system for Patan.

There is little doubt that this festival is a annual socio-cultural rendering of periodic inspections, repair and operational activities necessary to sustain the water system, its intake, canal, town reservoir pond, distribution system and outlets and drainage. Indeed for the proper ritual conduct of the festival, water must be available at Lagankhel and Pulchowk ponds and the stone water conduit of Sundhara running and well. Even the town reservoir had to be overflowing!

The Janaipurnima festivals held at the site of Kumveswor temple in north Patan happens in a water pond that is fed by a spring source and a local aquifer. It is a celebration of Sivite Hindus and the faithful believe that the waters of the pond come from Gosainkund, a high Himalayan pond in Langtang ranges, which had relieved Lord Siva from the burning sensation of poison that he took in a Hindu creation myth.

From Malla period records available at ponds, Hiti and some wells, it can be concluded that the construction, extension, renovation and maintenance of the traditional system continued with great fervor not only at the royal level but also with community and neighborhood as well as family level participation during medieval times in Patan. As a matter of fact, the involvement of the latter groups in the development and extension of the traditional system of water supply in Patan appears to have been much larger than that of the state. The Newar society considered establishing water dispensing elements such as Jadhu, Hiti and well as pious acts of great value.

Fig. 7 Showing Linkage of Canal, Ponds, Aquifers and Hiti (Joshi, 1993)

It has been established that the Rajkulo feeds into at least eighteen out of the 39 traditional ponds (Pokhari, Pukhu) known to have existed in Patan (Joshi, 1993), quite a few of these branches and ponds were constructed in the Malla period. These ponds supplied water directly or indirectly to 51 extant Hiti including four which are dry (UN-HABITAT, 2008) and 220 dug wells. Most of these wells and Hiti are located within the urban core. The two ponds of great ritual significance (and therefore, functional relevance) in the Rato Machhendranath festival of Patan are the Lagankhel Pokhari (Saptapatal Pokhari of legends linked with ritual bath of Machhendranath, Bunganhavam) of Lagankhel and La Pokhari (water for wetting of cane used for tying timber members of the Chariot and starting of the Chariot festival) of Pulchowk. Both the ponds are fed by the Rajkulo by bifurcating it at Tikhel, the major portion of water going to the set of ponds at Lagankhel, which served as the general reservoir for Patan. A smaller branch Rajkulo taken from Tikhel goes north-west to feed La Pokhari. The overflow from these main reservoirs is again channeled to a series of ponds within the settlement. The overflow from La Pokhari goes to Kamalpokhari (Palesvanpukhu) and sequential overflow charge Podepukhu, Pimbaha Pokhari and possibly to Nhyakachuka and Nagbahal. Similarly the chain of ponds fed by overflow from Lagankhel are Pyapukhu, Bhandarkhal Pokhari, Extensive underground channels (in brick, terracotta or even wood) ran from these ponds to supply water to several nearby Hiti and to recharge ground water. The ponds and canals also charged the natural shallow aquifers as available in different localities of the town- this novel approach was particularly important to recharge ground water so that the dug wells were always replenished with water. Many Hiti are directly supplied from canals taken out of shallow aquifers. Three major shallow aquifers of Patan located at Nayokhyo to the southeast, Naricha east of Lagankhel and Khwyebahi south of Pimbaha are charged in this manner.

Northernmost Hiti of Patan yield more water as topography slopes from southwest to northeast of the town and where 'thick layers of black cotton soil at the outskirts of Patan protect the flow path from seeping into the Bagmati River, while creating potential aquifers in the north (Joshi, 1993). In such areas, where local aquifers get sufficient recharge naturally, such as at Ikhachhe, Kumbheswor, Chyasal and Guita, no pond overflow or canal linkage is provided. (See Figure 5 which details the branching of Rajkulo about Patan and linkage to ponds, aquifers and Hiti.)

Waste water from some Hiti is also at times collected into a pond and used for irrigation or ritual purposes, for example, water draining out of Sougal Hiti is collected in Khapichen Pokhari, Bya Hiti recharges Chyasah Pokhari, Subaha Hiti and Guita Hiti waste water is collected at Balkumari Pokhari.

Lagankhel Pokhari, Guita pokhari, Lapukhu and Pyapukhu may be the oldest among the ponds. Of the 39 historical ponds substantiated in popular memory, 16 are relatively in good condition from shape and size considerations, 9 are encroached and the remaining 14 have been lost as public utility (UN-HABITAT, 2008). Only one of them, Jawalakhel Pokhari, may be accepted as in a well maintained condition. It also has its own spring source of water.

(4) The Technology of Hiti

Although the application of the principle of gravity flow to transport water from the reservoir to the pit may not seem like ground-breaking technology, using the same principle to drain out of the deep pit and developing a design and construction technology that would work for more than a thousand year is. Brick arched drainage channels constructed at such depths demand deep trenching and/or tunneling techniques. The construction of a chain of conduit pits with successive conduits at a level lower than the previous ones demands the use of filters in between to assure the purity of water all along the chain. Indeed, a complex maze of brick-lined, tile-covered circulating contraption with sand beds in sections was constructed immediately behind the conduits. This system is capable of settling silt, filtering and, even more amazingly, controlling and regulating the flow of water as it comes out of the spout.

What seems to be an architectural feature of progressively reducing terraces leading down to the pit courtyard actually covers the filtration and regulating structure of the conduit system. Another marvel is the waterproofing technology applied to the sides and bottom of the conduit pit. Not only the conduit pits but also the sides and bottom of the reservoir ponds were made waterproof by the application of an almost foot-thick layer of a particular type of lake silt deposit of gray/black soil (Gathucha).

The small size of the drain-off channel and its use as a supply channel to pit conduits located at lower levels defy the possibility of maintenance and repair action and it is a wonder how these were maintained or kept running and operational for so long at all. Whereas indigenous cultural practices linked to the maintenance of the supply canal and well system are very distinct and articulated in the sithi festival, among others, no such practices are known about the hiti drainage and filter-bed maintenance. Local people mention the use of an unbelievable means of maintenance such as the drains being unclogged and cleared by snakes as they go into the system in pursuit of their prey, toads! Local people recall that it was traditional in the Alko-hiti community to let fish and snakes into the waters of the drainage pit. According to them, the residents of the neighbourhood were required to let any snake found in their land loose into the same pit. It seems the custom of raising fish in the shallow waters of the drainage pit helped in the removal of moss, which served as food for the fish. Likewise, the toads fed on the fish and snakes would enter the drainage channel and slide through it as they fed on the toads, keeping the system maintenance free, as it were. Could this be a case of a self-maintaining aqua-technology assisted by animals such as fish, toads and even snakes?" (from Tiwari, 2009)

(5) Defunct Status of the Traditional Water system in Patan

The traditional water supply system of Patan is almost totally defunct and only some of the elements such as Hiti, wells and ponds are still functional purely because some sections of the traditional system was based on natural shallow aquifers charged by rain and surface drainage. The Rajkulo was last maintained and made working for irrigation purposes about fifty years ago. By 1993, the Rajkulo was working from the intake to only down to Chapagaun about four kilometers from Patan. Since then, the scale and pace of urbanization of the area along the road that almost follows the canal alignment up to Tikabhairav has all but obliterated the mud lined canal. Thus the Rajkulo has not supplied any water to the traditional urban water distribution system of Patan for more than three decades already and it is only a matter of historical interest for the present. Proposals to rehabilitate the Rajkulo in order to supplement the water supply to Patan also through the traditional outlets have been made but have not been accepted presumably for practical reasons.

No more fed by the canal waters, most of ponds making the Lagankhel Pokhari reservoir system started on the drying process since the 1950's and have since been encroached, filled up and built up or downsized. Offices, schools and market functions occupy the buildings built over the original land of the ponds. In a smaller scale, Laganpukhu, Nhupukhu and Lagankhel Pokhari (3 three sections) remain as rain fed ponds. The La Pokhari has become a small tank and much of Kamalpokhari has been converted into garden and Patan Municipal Office building built over the rest of its land. Pyapukhu exists in a much reduced size encroached by a school building. With its sides and bottom sealed, it has turned into a tank! Only 16 of the 39 traditional ponds of Patan survive in respectful shape, size and condition. The replenishment of water in these ponds is largely through collection of rain and residual surface drainage, although some are charged also by existing natural spring sources.

Naricha aquifer remains well charged by rain and surface drainage of Lagankhel as it the area is still open with unsealed surfaces allowing absorption of rain water into it. Wisdom of the urban culture of yore to create open grassland (khyo) around the ponds and aquifers has been lost on the present day managers of the town and more and more of these open spaces are being put to other uses that seal the ground. Such unplanned building up around other aquifers of Patan has started causing them to dry. In 2009, even the almost perennial Kumveswor Hiti dried because of such reasons. In some aquifers, indiscriminate digging new shallow wells have also led to its water table falling low enough to cause drying of its Hiti and traditional wells, which tend to be shallow.

As the ponds which collected the rainwater and surface water from its khyo and as both the pond space and the khyo are lost 'because of ignorance, change in habits and economy and greed of people', the aquifers fail to recharge sufficiently. Consequently, while some of the Hiti and wells have a reduced discharge some others have completely dried up.

Since the Hiti gets its supply from ponds and aquifers through the medium of existing porous seam, shallow brick or terracotta or wood canals or trough drains, variety of unplanned and reckless public and private construction interventions along this line of passage have has destructive impact on the Hiti. Many Hiti have failed due to their supply lines being cut or destroyed by large public constructions such as laying of sewer and drain pipes. The new drainage system also diverted precious water that recharged aquifers into storm drains. Poor and careless introduction of these newer services have also led to impairment of water quality of some of the Hiti. Second negative intervention into the system is the construction of foundation for large buildings in private sites due to which the underground supply lines passing through such sites have been knowingly or unknowingly destroyed.

In still other cases, the failure of drainage channel of the pit court has been the cause leading to the collapse of the system. As we reported above, for quite a Hiti these drainage canals tend to be too deep and inaccessible for repairs. The clogging of these drains due to disposal of plastics or other materials with slow biodegradability has led to ponding in the pit, collapse of the structure and its filling up.

Hiti occupies a vantage location in the urban heritage of Kathmandu, not only as a historical, technological and artistic entity of great social and economic value but also spatially as it sits amid the other built heritage with significant use of timber. The threat of fire and earthquake hazard to the other built urban heritage of Kathmandu Valley has been often highlighted. Further studies are needed to seek to answer how the traditional water supply system may be used to reduce vulnerability of the heritage buildings from fire hazards and for fire fighting in the event of fire. In addition, the conservation of the Hiti as a urban cultural heritage by itself, particularly in the context of the greater public concern directed at its utility role and loss of historicity, authenticity and artistry that has been a common consequence of the utilitarian actions undertaken at such instances, is an added objective. It is also likely that the traditional supply system is less vulnerable to earthquake and these systems could be expected to remain operational when large section of the modern urban water supply system may actually be dysfunctional due to damage from the quake.

4-1.3 History and Traditional Knowledge of Disasters

Himalaya is one of the most seismically active zones in the world and makes Kathmandu valley highly prone to earthquakes. Geological investigations indicate that Kathmandu valley was a lake that dried about 12000 years ago. Thick deposits of soft clay that have so formed the valley floor magnify tremors many times over and make the buildings built on it highly vulnerable to earthquakes. The great earthquake of 1934, which occurred in the Bikram calendar year 1990 and affected 55000 houses destroying over 12000 completely in Kathmandu valley, was such a disaster that Nabbesaal (meaning year 90 in Nepali) has become a standard word to denote massively disastrous earthquakes. Nepal has been hit by such great earthquakes many times in history and experts read a return periodicity of 70 to 100 years for Nabbesaal like earthquakes. Such a periodicity of large earthquakes is not 'frequent' enough to be conducive to empirical learning and development of building technology as earthquake mitigation measures as the memory of the violent shakings get diluted to ineffectuality for experiments at structural risk reductions over four or five generations. Indeed, we find little record of structural damage or craftsmen's comments in accounts of earthquake in traditional historical treatises such as vamsabali and thyasaphu. Table 1 (following page) shows a list of earthquakes recorded in history and the scale of disaster noted. It also reveals a fifty year periodicity of moderate earthquakes.

However, earthquake resistant features in traditional buildings of Kathmandu valley have developed indigenously with little benefit from Vaastusashtra. These and other learnings on earthquake mitigation of Kathmandu valley relate more to experiences accruing from medium and small earthquakes that had shorter recurrence cycle. Since architectural design as well as construction (or for that matter any other trade or profession) was 'jaat' based and knowledge and experiences passed on orally or as practical skills from one generation to another, a continuity of innovations seems to have taken place leading to a significant accumulation of design, detail and practice of construction systems that offered a great strength to buildings caught in earth shakes. (Tiwari, 1998)

Nepali traditional architecture uses brick as a key building material and brick masonry laid in mud mortar is common practice. The vulnerability of such materials and methods to earthquakes and the strength and resistance offered by the use of timber in the structure must have been quickly understood by the traditional builders as the building materials natural to Kathmandu valley faced this other 'nature'.

Over time and with experience, further development of construction system and technology, such as the application of simple planning approaches of symmetrical or square plan configurations, use of dalan, strengthening elements such as use of double framing of doors and windows, detailing such as use of wedges to connect and hold together brick wall and joists, joists and wall plates or column and beams, rafters and eaves or struts, use of tie runners as string course, took place. These measures individually and together as a system infused a great amount of technical resilience against earthquakes. Such measures are found used in most building types, whether low rise or high. The use of literally thousands of wedges in the construction of traditional building enabled them to absorb great amount of energy out of the earthquakes by moving within the limits of tolerance of the structural materials and methods and attain safety against failure. Safety gains were enriched by the timber ties and cornices that helped divide brickworks into smaller masses and offer tensile strength.

Apart from the mitigating response of the design, detailing and construction technology of the traditional buildings, traditional knowledge and capacity for mitigation of earthquakes appear to have been developed also through a conscious spatial structure of open spaces and courts in town (dalan in buildings, chuka, layebhu and lachchhi, layechi) and through preservation of the agricultural economic base of the society achieved by building bounded towns in agriculturally fallow tar land and consciously delimiting expansion of towns unto agricultural land through ritual and faith based socio-cultural deterrents.

Remarkable community based mutual support system had been developed through the application and development of the Guthi system (a community managed system of perpetual trust institutions set up with publicly owned land resources) and mutually supporting and interdependent jaat (caste) based division of society. The urban space itself was divided into neighborhoods with distinct jaat specificity and Guthi specific to neighborhoods and supporting community activities specific to particular jaat became common practice in the Malla period.

Kathmandu Valley's urban cultural heritage of building and urban spaces, water supply system, festivals and street spaces, conservation practice and its institution of Guthi deserve to be researched specifically to further inform the mitigation measures and ways of their incorporating into the modern building, town planning and social institutions.

With the advent of modernism and modern administrative, technological and development processes have overwhelmed the traditional knowledge of earthquakes and disaster management and have all but wiped out the traditional knowledge system. The massive introduction of newer building technologies without serious adaptation for earthquake resistance in terms of science, technology and skill in building, the loss of the spatial structure brought about by the accelerated urbanization and new urban form and the loss of community organization of Guthi and social structure that facilitated mutual support have all come together to make contemporary Kathmandu valley greatly vulnerable to earthquake induced disasters socially, physically, spatially and institutionally.

Table 1: Historical Earthquakes of Nepal

Year (A.D.)	Deaths	Damages
Dec 24, 1223	Many people died	A lot of damages to residential buildings and temples
June 7, 1255	One third of the population of Kathmandu valley was affected. Many deaths including that of reigning king Abhaya Malla after eight days of tremor	A lot of damages to residential buildings and temples, people left their houses and lived outside for a period of a fortnight to a month after earthquake, while aftershocks were felt for the following four months.
1260	Many people died, famine after earthquake	A lot of damages to residential buildings and temples.
September 14, 1344	Many people died including the reigning king Ari Malla in Deupatan after one day of tremor	A lot of damages to residential buildings and temples
1408	Many people died	A lot of damage to temples, residential buildings, fissures developed in the ground. Machhendra Nath temple in Patan collapsed.
1681	Many people died	A lot of damages to residential buildings
June 4, 1808	Several people and animal perished	Houses collapsed
1810	Some people died, mostly in Bhaktapur	A lot of damage to buildings and temples
1823	No record of deaths	Some damage to houses
1833	Estimated magnitude 7.7, 414 people died in the vicinity of Kathmandu valley	4040 houses destroyed in Kathmandu, Bhaktapur and Patan and Banepa. 18,000 buildings damaged in the whole country
1834	No good record available	Many buildings collapsed
1837	No good record available	No damage recorded in Nepal but greatly affected Patna and other parts of Bihar, India
1917 (1918?)	No deaths recorded	No record on damage
January, 1934	Estimated magnitude 8.3 (epicenter, Eastern Nepal). Maximum Intensity X. 8519 people died of which 4296 died in Kathmandu valley alone	Over 200,000 buildings and temples damaged, nearly 81 thousand completely destroyed in the country. 55,000 buildings affected in Kathmandu (12, 397 completely destroyed)
1966	24 people died	1300 houses collapsed
1980	Magnitude 6.5 (epicenter far western Nepal). 103 people died	12, 817 buildings completely destroyed, 2,500 houses collapsed
1988	Magnitude 6.5 (epicenter SE Nepal). 721 people died	66,382 buildings collapsed or seriously damaged
1993	Epicenter near Jajarkot	40% of the buildings were estimated to be affected

(Based on Marahatta, 2011 sourced out of UNDP/UNCHS 1993, Pandey and Molnar, 1988, Bilham et al 1995, Pant, 2002)

References:

1) Gellner, David N., Monuments of Lalitpur (Patan) in Michel Hutt, 1995, Nepal, Shambhala, Boston.
2) HMGN, DHBPP, 1975. Kathmandu Valley – The Preservation of Physical Environment and Cultural Heritage – A Protective Inventory.
3) Gutschow, Niels 1982. Stradtrum und Ritual der newarischen Stadte im Kathmandu-Tal, Kohlhammer, Stuttgart pp 168.
4) Tiwari, Sudarshan Raj 2009. The Temples of the Nepal Valley, Himal Books, Kathmandu.

Gellner in Hutt, 1994)The following books and reports have been used in the preparation of this report:

Tiwari, S.R. 1998 Traditional Architecture of Kathmandu valley – responsiveness to earthquake through empiricism', available at www.kailashkut.com/publications.

Tiwari, S. R., Transforming Cultural Heritage into Sustainable Future: A Case Study of Patan, Nepal, in Urban Crisis, Culture and the Sustainability of Cities, 2007, eds. M. Nadarajah and Ann tomoko Yamamoto, United Nations University Press, Tokyo.

Marahatta, Punyasagar 2011, Draft Chapters of PhD Disssertation, Institute of Engineering, unpublished.

Jigyashu, Rohit 2002, Reducing Disaster Vulnerability through Local Knowledge and Capacity – the Case of Earthquake Prone Rural Communities in India and Nepal, PhD dissertation, NTNU, Trondheim.

UN-HABITAT, 2008. Water Movements in Patan with Reference to Traditional Stone Spouts, Water for Asian Cities Program Nepal, Kathmandu.

Becker-Ritterspach R.O.A. (1994) Water Conduits in the Kathmandu valley, Munshiram Manoharlal Publishers Pvt. Ltd. New Delhi.

Joshi, P.R. (1993). Feasibility Study of Rajkulo: Rehabilitation of Patan’s Traditional Water Supply Network, Final Report, Patan Conservation and Development Program, UDLE/GTZ.

Theophile, E. & Joshi, P.R. (1992), Historical Hiti and Pokhari: Traditional Solutions to Water Scarcity in Patan, Patan Conservation and Development Program, UDLE/GTZ.

Tiwari, S.R. (2009). The Temples of the Nepal Valley, Himal Association, Lalitpur.

Regmi, D.R. (1983). Inscriptions of Ancient Nepal, Vol I, Abhinav Publications, New Delhi.

MSc Urban Planning, Urban Planning Studio III, 2009 (Students Works)

4-2 Assessment of Heritage Values in Case Study Area

Sudarshan Raj Tiwari
Rohit Jigyasu
Naoko Itaya

4-2.1 Purpose of HeritageValue Assessment

The purpose of assessment of heritage values of the historic city of Patan is to identify and prioritize the protection of attributes with most important heritage values for disaster mitigation, response and recovery, especially those attributes that can serve as important resources for disaster mitigation and response.

4-2.2 The Target Area

The target area is a Jhatapo neighborhood in the historic city of Patan. This area includes core and the buffer zone of World Heritage Monument Zone of Patan, which consists of Durbar (palace) Square (Fig.1, 2).

The total population of the target area of Jhatapo is 507, which includes 68 children below age of 12 and 49 elderly people above 65 year. Only one person in the area is handicapped.

This area is inhabited by Newars; an ethnic group in Kathmandu Valley. Many of them belong to Maharjan caste. Most of them work in service sector that includes handicrafts, shops, hotels and other retail business. Also majority of them own their house from forefathers and have lived there since birth. Also most of them do not own any agricultural land.

Fig.1 Patan WHsite Coa Zone and Buffer Zone

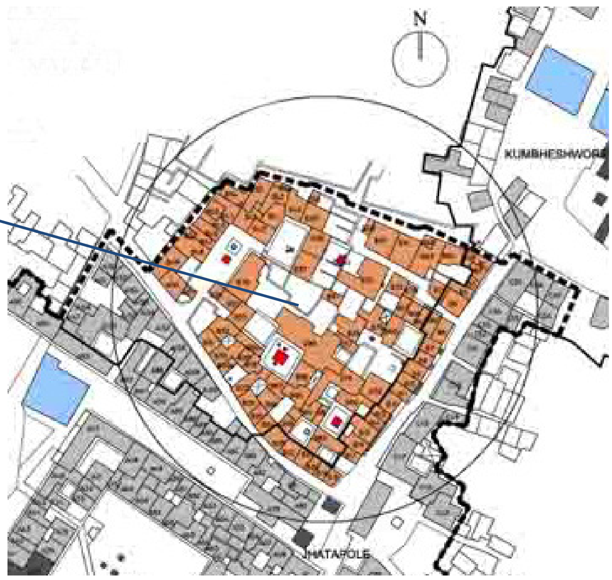


Fig.2 Jatapo, Research Area

4-2.3 Overall Methodology

Overall methodology can be divided into four main stages:-

First Stage: Preliminary Site Documentation and Community Diagnosis

The first stage involved collection of background material from various secondary sources on Patan's history, settlement pattern and cultural heritage. After selection of target area, a base map was prepared through field verification (from outside) and each building and open area was given a unique reference number. Next, specially designed inventory sheet was filled to gather data on each building and household related to architectural typology, ownership, usage, height, construction system, age and additions, structural and material condition on the basis of visual observations. Inventory forms to record other heritage components like temples, chaityas were also prepared.

Key informant interviews and a workshop with local residents of Jhatapol were also conducted with some residents to know their perceptions about heritage values of the area, memory of past disasters especially 1934 earthquake and current perceptions of risks in the area. Views of the local community demonstrated strong attachment of people to that area. It also showed low level of disaster preparedness.

All this data was subsequently analyzed and various thematic maps of target area were prepared.

Second Stage: Rapid Vulnerability Analysis

For all the heritage buildings/components identified during stage one, vulnerability analysis with respect to earthquake, fire and flooding was conducted through visual observations. The data for each building was recorded in data sheet and illustrated through photographs and three dimensional drawing.

Third Stage: Detailed Documentation and analysis of integrity and transformations.

Detailed documentation of the target areas in terms of interior walls and primary/secondary access points was undertaken to analyze the level of integrity and transformations in each structure.

Afterwards another round of survey was undertaken to visually record the integrity of the structures through analysis of transformations. In some cases, the structure appeared as one traditional house from outside but had been divided internally. In other instances, although two or more houses from outside were infact one structure, which could be confirmed by their uniform façade inside. However in both cases, transformations were linked to vertical division of ownership of single traditional house among one or more families. There were rare instances, where a traditional house was sliced to cut a new structure while retaining partial traditional structure.

On the basis of this analysis, original reference numbers were regrouped to arrive at a nomenclature specific to each traditional house that retained its architectural and/or structural integrity to a great extent.

Fourth Stage: Identification and analysis of Values

Various indicators were developed to analyze values for each attributes (heritage building and component) as identified during detailed inventory. Based on visual analysis, numerical weightage was assigned to each indicator from 4 (highest value) to 1 (lowest value). Comparison was drawn separately among traditional houses and chaityas for which scores from various indicators were aggregated for each building/component.

Other components like patis, trees, temples, wells, routes, courtyards were only assessed on the basis of qualitative indicators.

Those heritage attributes which have special value as resources for emergency response such as well and pati were identified separately.

A spreadsheet of all heritage attributes with their weighage scores for indicators was prepared to undertake comprehensive analysis for identifying attributes with highest values using Geographical Information System (GIS). The results were presented in various maps showing traditional buildings in various value scales, as well as other heritage components containing various values (individual and composite)

4-2.4 Attributes for Assessing Values:

Various attributes of the target area are being assessed for their values. These attributes are mainly physical but have intangible aspects embedded in them. These exist at area, building and component levels.

Area level Attributes (Fig.3) :

- Open Spaces
- Streets (Main streets, Alleways and Underpasses)
- Squares

Open Space Main Street Underpass

Fig.3 Area level Attributes

Building level Attributes (Fig.4):

- Traditional Houses
- Patis
- Temples

Traditional Houses Pati Temples

Fig.4 Building level Attributes

Component level Attributes (Fig.5):

- Chaityas/Shrine/Pillar/Tulsi Math
- Wells
- Hitis (not seen in target area)
- Inscriptions
- Sculptures/ Bells
- Trees (not seen in target area)

Chaitya Shrine/ Tulsi Math Well

Inscription Sculpture Pillar Bell

Fig.5 Component level Attributes

4-2.5 Indicators for Assessment of Heritage Values of Attributes

The values embodied in cultural heritage are identified in order to assess significance, prioritize resources, and inform conservation decision-making. Heritage values of the historic city of Patan have already been elaborated in section 4.1. Based on this, following values have been identified for detailed analysis of various attributes of the target area.

(1) Historic Value

Buildings and heritage components which are the product of human activities accommodate a wide spectrum of significance in our history. Principally their historic values result from their sheer life-span that reflect the corresponding times of construction in diverse periods as well as from multitudinous distinctions and qualities added to them in the passage of time. Time specific traditions and historic styles displayed in those buildings record and convey myriad fragments of the individual and collective memory of the aspirations of a society and a people.

For historic city of Patan, historic value for all built components was assigned on the basis of following indicators that would determine their weightage on the basis of the following period of original/first construction that might have subsequently been added/altered over time.

Criteria for Original Period of Construction	Score (Weightage)
More than 100 years	4 (highest)
Pre 1934 Earthquake	3
1934 to 1950 (until Rana Period)	2
Post 1950	1 (lowest)

For heritage components such as Chaityas (which are not buildings), historic value would also be determined on the basis of originality of material on the basis of following criteria:-

Criteria for Original Material	Score (Weightage)
Original Material	3(highest)
Added new material	2
Totally new material	1

(2) Aesthetic Values

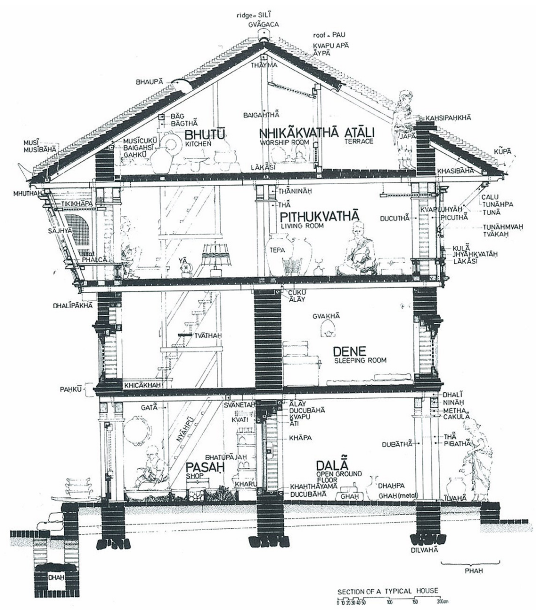
Aesthetic refers to the visual qualities of heritage. The many interpretations of beauty, of sublime, of ruins, and of the quality of formal relationships considered more broadly have long been among the most important criteria for labeling things and places as heritage. Aesthetic values of built heritage can be classified further as architectural and artistic values

(3) Architectural Value

Architectural value can be seen as part of aesthetic value that is embodied in the design and evolution of a building. It is embodied in its spatial planning, integrity of structure and façade as well as juxtaposition of individual elements that contribute towards its overall architectural character.

For historic city of Patan, architectural value for all built components was assigned on the basis of following indicators that would determine their weightage on the basis of following indicators:-

- Originality of traditional height
- Integrity of structure
- Integrity of façade
- Traditional architectural elements
- Traditional artistic elements



Originality of Traditional Height

Structure and Façade of traditional Houses

Fig. 6 Architectural Value of Traditional House in Patan

Criteria for Originality of Traditional Height	Score (Weightage)
Traditional height; 3 floors plus attic	3 (highest)
Added floor to the traditional height: Total 4 to 5 floors	2
Higher than 6 floors	1
Single floor	1 (lowest)

Integrity of structure and façade is based on the transformation processes in traditional houses affecting their vulnerability. These include:-

- Vertical subdivision by adding thin partition walls inside but keeping traditional configuration and structural system. In this case, the house retains its integrity from outside.
- Vertical subdivision by slicing the building and adding RCC framed structure while rest stays as load bearing. In this case the structure has lost its integrity to a great extent.
- Only skin façade is altered on either side facing the road or backyard or courtyard. In this case the buildings may appear contemporary from outside but retain overall structural integrity. This sometimes is confirmed by traditional façade seen from courtyard side.

Criteria for Integrity of Structure	Score (Weightage)
Overall Traditional Construction System (load bearing brick masonry). This would include minor additions and partitions.	3 (highest)
Hybrid (traditional construction with major addition of modern RCC frame walls, floors)	2
Concrete frame with brick infill (contemporary structure)	1

Two facades (entrance and secondary) are investigated for their integrity and traditional architectural and artistic elements.

Criteria for Integrity of Façade	Score (Weightage)
Original façade is symmetrical with traditional patterns and proportions between various elements	3 (highest)
Divided and/or partly changed original facade	2
Totally new façade	1

Traditional Architectural Elements in Façade		Yes/No (1/0)
Roof	Sloping	□
	Eaves	□
Wall in traditional brick/wood		□
Window	Large Malla Style Window	□
	Traditional Window with Lattice	□

(4) Artistic Value

Artistic value of built heritage (buildings) can be understood in terms of the works of art through painting or carving, which can be evaluated on their own merit although they may be part of the architecture.

Traditional Artistic Elements in Facades of Traditional Houses	Yes/No (1/0)
Carving in Wood / Brick	□
Painting on Wood / Brick	□
Sculptures (including those inserted in the walls of houses)	□

Painting on Brick Sculptures (inserted in the walls of houses)

Fig.7 Artistic Value

For heritage components (not buildings) such as Chaityas, artistic value can be understood in terms of following artistic elements of special values. These can be evaluated either on the basis of weightage or as simple checklist determining their presence/absence . Various indicators include

1. Degree/Quality of Carving (High-3/Medium-2/low-1)
2. Degree/Quality of Painting (High-3/Medium-2/low-1)
3. Presence /Absence of Sculpture (1/0)
4. Presence/Absence of Inscription (1/0)
5. Presence/Absence of Metal Canopy (1/0)

Some of the subcomponents like inscriptions can be evaluated in themselves for their historic or artistic values determined by another set of indicators.

(5) Socio-religious values of living heritage (livingness value)

As pointed out in section 4.1, one of the essential characteristics of urban cultural heritage in Kathmandu valley is its living nature i.e. it is not just about the past but also the present relationships that demonstrate continuity over time. This living nature is contiguous with the socio-religious values.

The concept of social value follows closely the notion of ‘social capital’, a widely used concept in the social science and development fields. The social values of heritage enable and facilitate social connections, networks, and other relations in a broad sense, one not necessarily related to central historical values of the heritage. The social values of a heritage site might include the use of a site for social gatherings such as celebrations and markets – activities that do not necessarily capitalize directly on the historical values of the site but, rather, on the public space, shared-space qualities. The kinds of social groups strengthened and enabled by these kinds values could include everything from families to neighbourhood groups to ethnic groups to special interest groups. Social values also include the ‘place attachment’ aspects of heritage value. Place attachment refers to the social cohesion, community identity, or other feelings of affiliation that social groups derive from the specific heritage.

Heritage sites are sometimes associated with religious or other sacred meaning, which in case of Kathmandu value are deeply related to the social values and together contribute to the living heritage. These attributes would include

1. Temples
2. Chaityas
3. Sculptures
4. Stones in the paving used for worship
5. Religious Paintings on door etc.

Criteria for determining socio-religious values of various attributes as living heritage	Yes/No (1/0)
City/National Level Association	□
Clan Level	□
Neigbourhood Level	□

City/National Level Association : Janaipurnima Festival at Kumveswor, Matsyendranath Jatra

Clan Level Association:

Neigbourhood Level Association:

Family Level Association:

Fig.8 Socio-religious values of living heritage (livingness value)

(6) Ecological Values

The ecological values are related to those attributes of natural environment that contribute towards sustainability. In the context of Patan these are represented by trees and water sources.

Attributes of natural environment	Yes/No (1/0)
Trees	□
Water Sources/catchment areas	□

(7) Resource Value for Disaster Mitigation and Response

Attributes with heritage values that have potential for disaster mitigation and emergency response such as Patis, wells etc. are especially identified for protection. This will be elaborated in section 4-3

(8) Assessing HeritageValues of Traditional Open Spaces and Routes

Open spaces are very important element of urban heritage of Patan. In fact, traditional open spaces and routes have retained the morphology of the city while built elements have been exposed to continuous transformations. Heritage Values of traditional open spaces and routes can be assessed on the basis of

1. Values of built heritage components in and around open space (historic architectural and artistic values)
2. Livingness Value demonstrating continuing relationship of open space at family, neighbourhood, clan or national/city levels.
3. Resource value for disaster mitigation and emergency response due to accessibility through traditional underpasses, presence of traditional water systems and public buildings such as Patis and Sattas that can be used for shelter and storage.

The relationship between attributes, indicators and values can be summarized as following:-

Attribute of the Site e.g. traditional building, Chaitya, temple etc.	⇨	Values to be assessed for each attribute e.g. historic, architectural, artistic etc.	⇨	Indicators for each values to be evaluated on the basis of various characteristics of attributes

4-2.6 Assessment of Heritage Values of Attributes

Geographical Information System (GIS) is used to prepare drawings of the area showing comparative heritage values of traditional houses (historical, architectural and artistic, resource value). These are aggregated to make comparison of composite values and represented in a drawing of the target area.

Comparative values (separate for each type of value and aggregate) are also assessed for Chaityas and other attributes and represented on area drawings

A composite value index map is prepared for all the attributes through GIS to identify the most important area from the perspective of heritage values.

It is very important to mention here that the results of the GIS analysis would be based on the weightage assigned to each value. In our case, we have assigned equal weightage to historic and aesthetic (architectural and artistic) values. However, if we were to give more weightage to either of these, the results might change significantly. Besides, results would also significantly alter if we were to give weightage to the livingness value. In such situations, the attribute may not be so old or aesthetically beautiful but would have higher value due to its continuous association with the community at city or national level.

Fig.9 Assessment of Heritage Values of Buildings using GIS

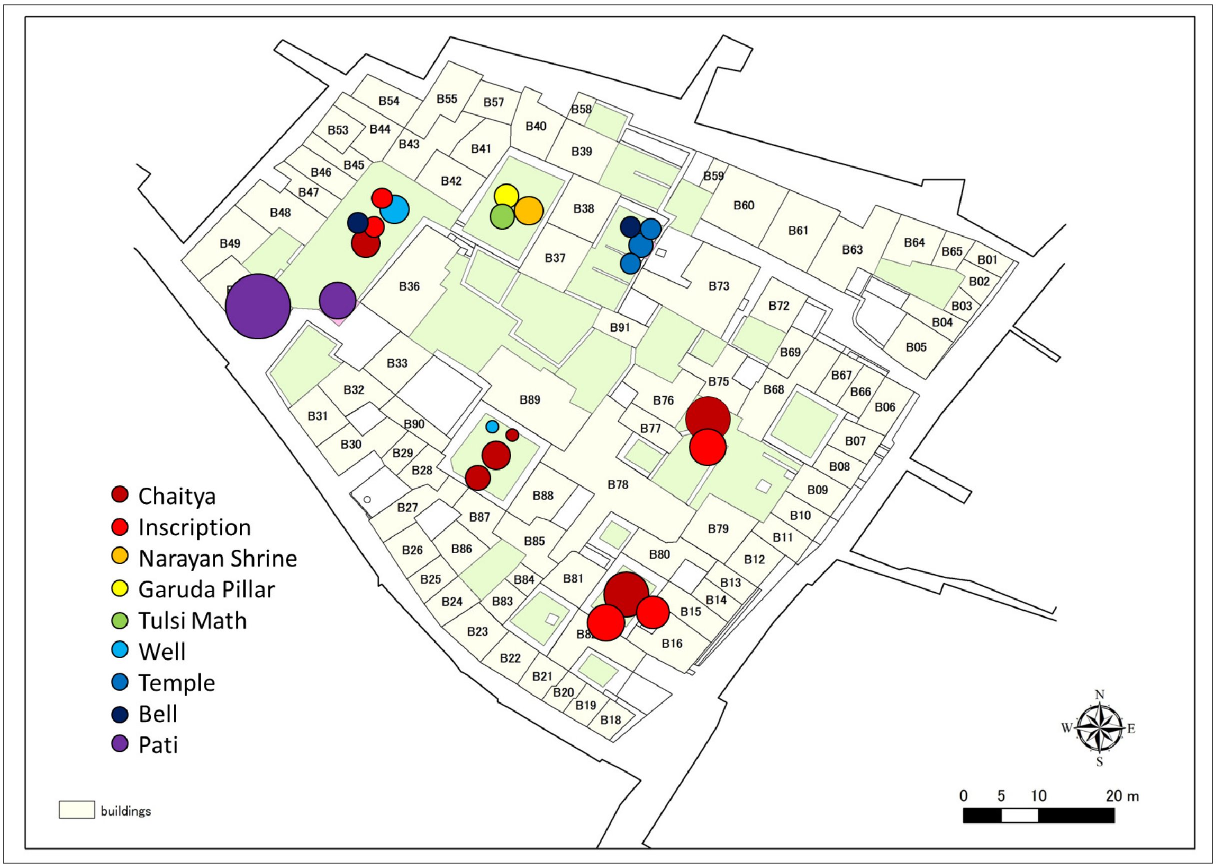


Fig.10 Assessment of Heritage Values of Components using GIS

Fig.11 Assessment of Heritage Values of Open Space using GIS

B82 B81 B15 B16

B16 B15 B14 B13 B50 B48 B46

Fig.12 High Value Buildings

Chaitya and Inscliption (Open Sspace 3) Chaitya and Inscliption (Open Sspace 7)

Fig.13 High Value Heritage Components

Well and Chaitya Patis

Fig.14 High Value Open Space (Open Sspace 13)

4-2.7 Indicators for Prioritization of Attributes for Disaster Mitigation.

The prioritization of each attribute for disaster mitigation is based on three factors:-

- Aggregate heritage value
- Resource Value for disaster mitigation and response
- Physical vulnerability (especially to earthquake) : This will be elaborated in section 3-3

Based on this prioritization, the most critical area for detailed intervention/proposals for disaster mitigation would be proposed.

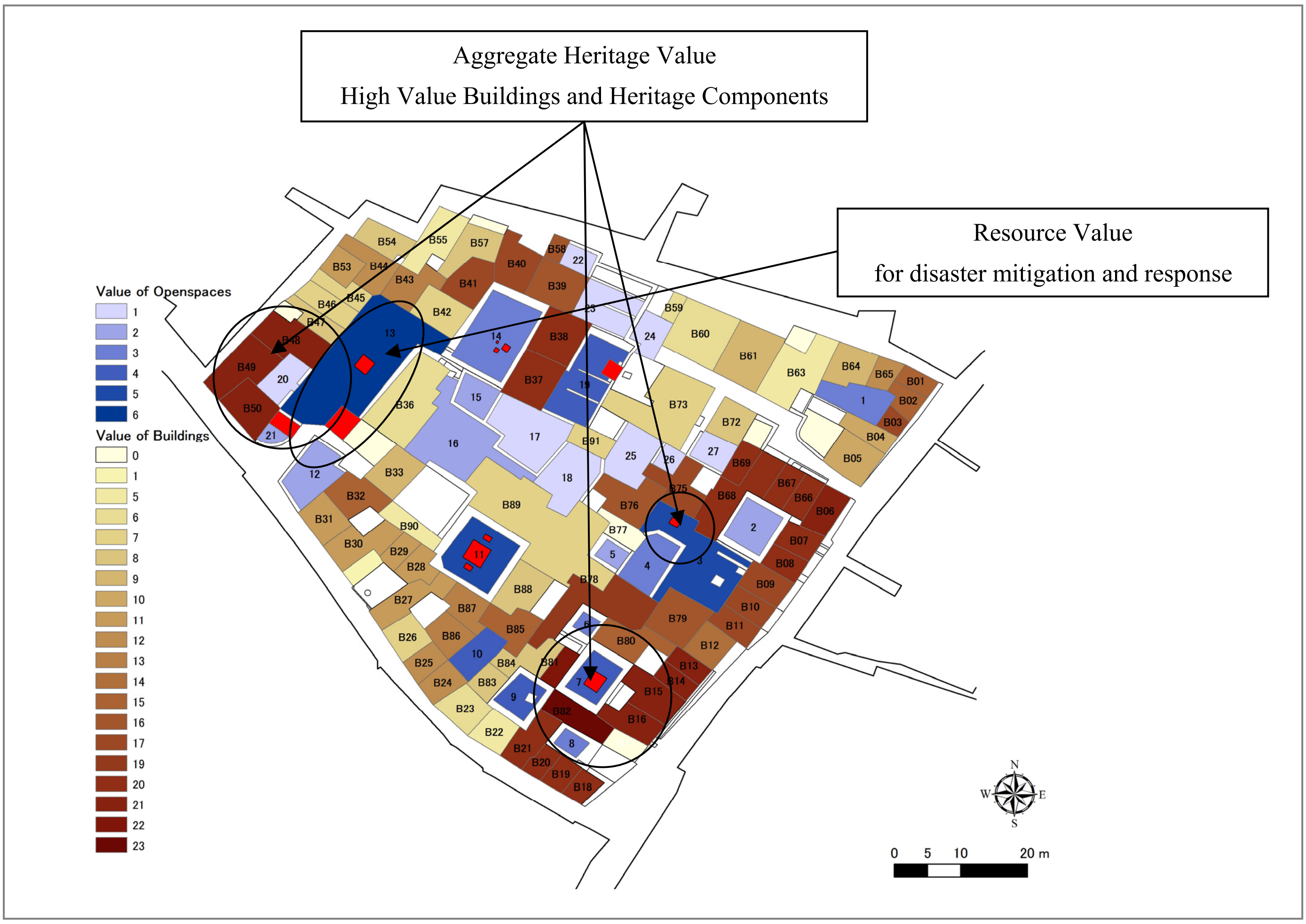


Fig.15 Indicators for Prioritization of Attributes for Disaster Mitigation

4-3 Assessment of Resources for Disaster Mitigation and Response

4-3.1. Survey for evacuation planning

Masahiro Yoshida
Takeyuki Okubo

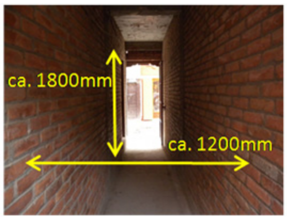
(1) Research of courtyard and courtyard entrance as evacuation route

a) Layout

Nepalese traditional urban structure in Patan, which was built by native Nepalese as Newari during Licchavis and Malla period, has the following features.

i A courtyard is enclosed by old houses (Pic. 1).
ii Each courtyard is linked with narrow courtyard entrances (Pic. 2)


Pic. 1 Courtyard



Pic.2 Courtyard Entrance

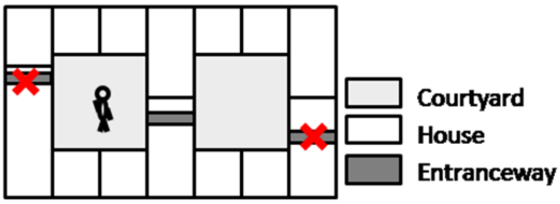


Fig. 1. Model of Unique structure

Therefore, in light of evacuation from earthquake disaster, Nepalese traditional urban structure is one of the elements of difficulties for people in the houses to evacuate to outside after the earthquake shock (Fig. 1).

Accordingly, the decision making of relative priorities for each element to be restored and reinforced within limited time and budget might be important.

In this research, at first, we conducted residents' workshop with Disaster Imagination Game method for understanding these risk from the residents view point, and calculated the quantitative importance of each route as evacuation route for residents (the possible number of people passing through each courtyard entrance was calculated).

b) Residents workshop on risk of courtyard entrance blockage with DIG

According to the results of DIG, residents indicated that the southern courtyard entrance should be kept safe for surviving earthquake disaster (fig.2). However, the results included a problem that not the all residents in Jhatapo participated in DIG workshop. In response to this challenge, the possible number of people was calculated to find which courtyard and courtyard entrance was relatively important than the others.

Evacuation routes were selected by the residents in consideration of location of water supply, location of open space, house of the vulnerable, location of community's meeting place.

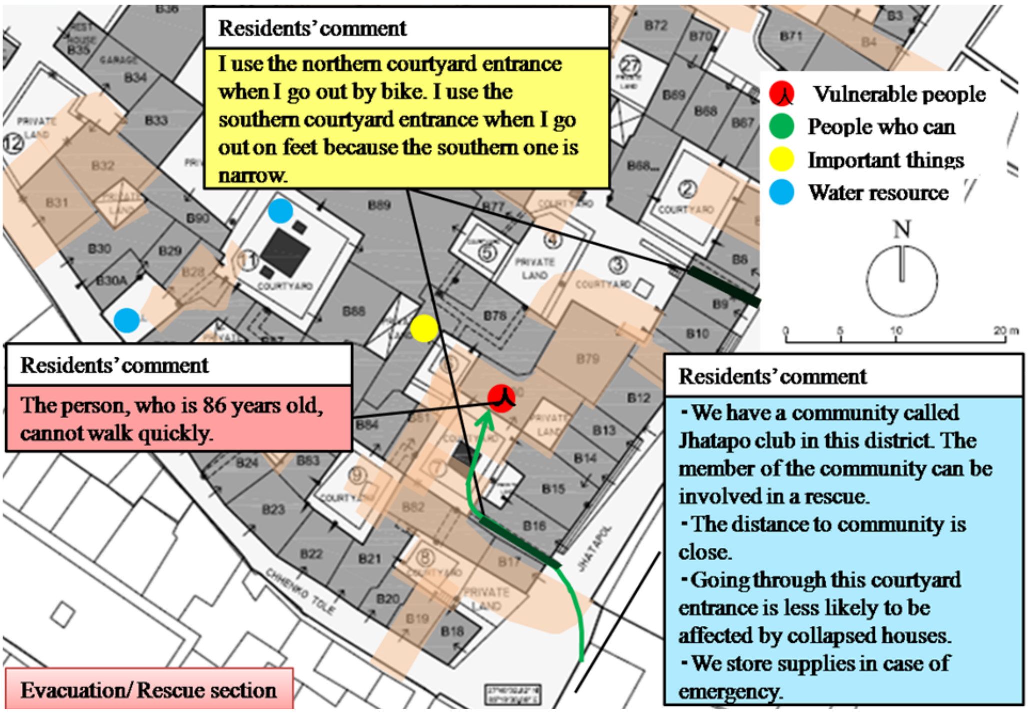


Fig.2. Disaster Imagination Game (DIG) in Jhatapo

c) Risk analysis of courtyard entrances

Evacuation routes in each neighborhood, which consist of courtyard entrances and courtyards, as well as buildings facing roads need to be restored and reinforced. Among these, reinforcement of courtyard entrances is the most inexpensive and feasible option. Access paths to be reinforced need to be prioritized for effective safety improvement with limited resources. This section describes the method to identify the access paths with number of evacuees (fig.3).

(a) Investigating the number of residents in the each building facing to the access paths.

(b) Dividing the number of residents in each building by the number of exits to calculate the average number of evacuees using each exit (In many cases, a building has exits both to road side and to courtyard side).

(c) Calculating the number of evacuees in each access path.

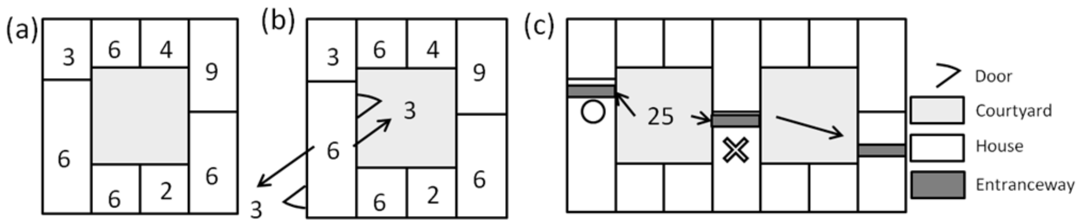


Fig. 3. Method of Calculation

d) Application to Jhatapo as a case study

The result of application to Jhatapo is shown in (fig.4). It also shows the risk of courtyard entrances in five levels. The number of people going through courtyard is shown in fig.5.

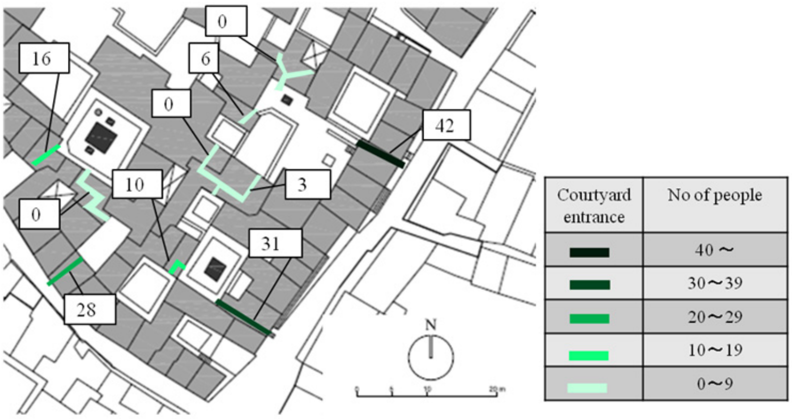


Fig.4. Trial calculation of numbers of evacuees through the courtyard entrances

Fig.5. Trial calculation of numbers of evacuees through the courtyards

(2) Challenges and Countermeasures for Disaster Mitigation

The workshop was conducted three times in all. Fifteen people, who abide in not only Jhatapol but also core and buffer zone of WHS, were gathered to hear residents' opinions in the first workshop. Their opinions could be considered as significant viewpoints for the further planning. The second workshop is called DIG (Disaster Imagination game). It was a kind of workshop method to extract challenges and countermeasures for disaster mitigation with a large scale map of target area. On Aug 13th 2011, this second workshop was participated by 7 people, who abide in Jhatapo and the surroundings. Seven people divided into 2 groups in DIG; group 1 with 3 people living in east side of Jhatapo, and group 2 with 4 people in west side and the surroundings. On Aug 20th 2011, last workshop as evacuation training was held. It was conducted to discuss more on challenges and countermeasures for disaster mitigation, and make sure of the result of DIG. The same 5 people (2 absentees) took part in evacuation training. The flow of DIG and evacuation training were shown in (fig.6) and (fig.7).

1	Taking a walking in target area
2	Writing a map
3	Pre-questionnaire
4	Self-introduction
5	Learning the past earthquake
6	Putting basic information on the map
7	Discussing fires
8	Discussing evacuation
9	Discussing the vulnerable
10	Post-questionnaire

Fig. 6. Flow of DIG

1	Firefighting training
2	Checking a flared house
3	Checking a narrow road
4	Checking a disused entrance
5	Simulating vulnerable people
6	Going to selected evacuation area
7	Questionnaire

Fig.7. Flow of evacuation

(3) Analysis of results

a) Analysis of Fires Section

Although there is no traditional water supply called Hiti in Jhatapo which is often seen in Kathmandu Valley, people living in Jhatapo can use water supply equipment even in case of fire disaster, connected with pipeline from Konti Hiti (pic.3) in neighbor district which Kumbeshole Club manages. They can help each other districts such as Jhatapo not having one's own water supply. However, to form relationship between communities, they need a sustainable opportunity to discuss on disaster.

Pic.3. Water supply from neighbor's community

b) Analysis of Evacuation Section

In evacuation section, choice of route was discussed with their unique urban structure taken into consideration. Group 1 said "we have to discuss the wall which would prevent from evacuating". Group 2 said "Nepalese government should purchase the courtyards including the wall as public domain and remove it". The wall as barriers for evacuation (pic.4) was seen as a problem by both groups.

Pic.4. Wall preventing from evacuation

c) Analysis of Rescue Section

In rescue section, system about their community was discussed to convey information about support of vulnerable people quickly. They have two communities. One is called Jhatapo Club and they have a chance to talk about to organize their festival and maintaining their residential road every month. The other is called LUZA consisting of only women and they save funds in case of emergency. They said "we would like to think about disaster prevention after this workshop now, because we did not create an occasion to discuss disaster countermeasure".

d) Analysis of evacuation training

In the evacuation training, the program was conducted as shown (fig.6). In the first section, participants experienced the first-aid firefighting without giving any information. However, it was seen that they tried to extinguish fire by themselves (pic.5). After that, they can get Japanese method to extinguish fire. This fact is found that conducting the workshop about fire disaster is effective in Nepal. In the sixth section, participants discussed about evacuation space after going to official evacuation area, shown in fig.8. Hearing agenda and its result are shown in table.9.


Pic.5. Firefighting training

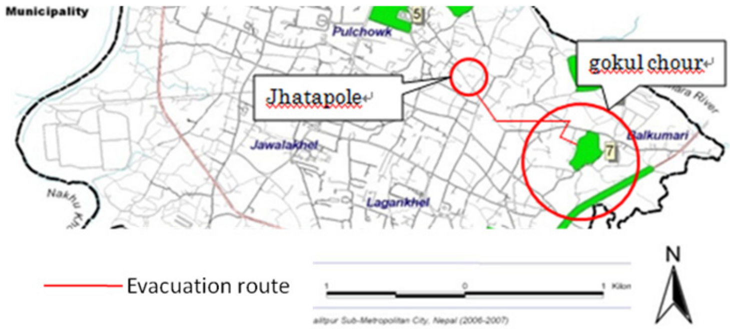


Fig.8. Route used in the evacuation training

Table.9. Hearing agenda and its result

Agenda	Question	Answer
Effectiveness of selected evacuation area	Do you know where selected evacuation area by government is?	Nobody knows the place of selected evacuation area.
recognition degree of selected evacuation area	Do you think you can go to selected evacuation area at the time of earthquakes?	First, Residents cannot go because of narrow road. Tourists cannot go due to recognition degree of selected evacuation area. Vulnerable people cannot go because of the length to the selected evacuation area.

According to the above result, current evacuation area could not be considered as sufficient evacuation space because of many tourists, the length of route and the urban structure (narrow road). Therefore, this area needs complimentary evacuation area, which can compensate for shortcomings.

4-3.2 Survey for Firefighting

Haruki Nagashima
Takeyuki Okubo

(1) Current condition of the public firefighting capacity

The investigation was conducted with interview to the officer in Lalitpur Fire Station on September 16, 2010. The purpose of the investigation was assessment of current condition of the public firefighting capacity. The research items were mainly about human resources, equipment, and water facilities for firefighting. The measurement of actual size of the fire engine was conducted, because the specification of size was not kept in the fire station.

According to the results of the investigation, Lalitpur Fire Station is in charge of fire disaster management in the world heritage core zone of Patan, with 11 officers and only one old fire engine. Several public fire hydrants are present, but they were installed more than 80 years ago during the Rana period, and therefore they are unusable now. Water for firefighting is only stored in the tank of the fire engine. The tank's capacity is 2,400 liters, which would run out after only 10 minutes of firefighting.

The investigation revealed the current condition that the public firefighting capacity is so insufficient, and sustainable firefighting activities would be even difficult in normal situation.

Chart. 1 Actual size and capacity of the fire engine

Chassis/body

Chassis made: Magirus Deutz   Chassis type: L1 Pruf-Nr.~~~S-48

Body model: 170 D 11   year (model): 1975 AD.   year (donated): 1975 AD.

Width×Length×Height: 2.25 m×6.25 m×2.85 m   Tire: 0.94 m   Wheel: 0.58 m

Wheelbase: 3.20 m   Engine model: LO2303   Engine power: 200 HP   Total weight: 60 t

Watersupply

Pump made: Magirus   Pump type: low / high pressure (Max 100 mWs.)

Water tank contents: 2.400 l

Water releasing capacity: 50m at full throttle / all water is released in 10 minutes at 1 hose

Spraying hosereels: 8×35 m / 7" Ø (jointable) / top : 1.8" Ø×1 / 1.6 Ø×2

Suction hosereels: 8×35 m / 7" Ø (jointable)

Cabin/superstructure

Number of seats in crew-cabin: 9 (7 members ride in usual)

Driver's cabin: 2 (driver and helper)   Number of breathing apparatus/ fastening units: nothing

Number of equipment lockers/roller shutter doors: 2×3 + 1 at rear

Roof

Equipment: ladder rack, ladder (small one : 0.3×3.6 m / large one : 0.3×6 m)

Accessories

Equipment: rip pipe (60 m) / 9 helmets

(2) Assessing the effectiveness of Hiti: traditional water resource in Kathmandu Valley

The measuring investigation was conducted with simple measurement of discharge from Hitis in the world heritage core zone of Patan on September 14 2010. The purpose of the investigation was assessing the effectiveness of Hiti as a water resource for firefighting. 3 Hitis in the target area: Konti Hiti, Nagabaha Hiti, Manga Hiti were surveyed.

a) Effective amount as a water resource for firefighting

The water releasing capacity of fire engine in Lalitpur Fire Station is 4 liters per second. Therefore, Hitis which have discharge more than 4 liters per second can be considered as water resource for firefighting

b) Method of the simple measurement

The time to fill up 1 liter measuring container with discharge from each tap of all Hitis was measured. Amount of discharge per second of each tap of Hitis was calculated, and total amount of discharge per second of each Hitis was calculated.

c) Results of the investigation

The results of measurement are shown in Chart. 2. According to the result of simple measurement, only Konti Hiti had discharge more than 4 liters per second, and the result shows Konti Hiti could function as a water resource for firefighting.

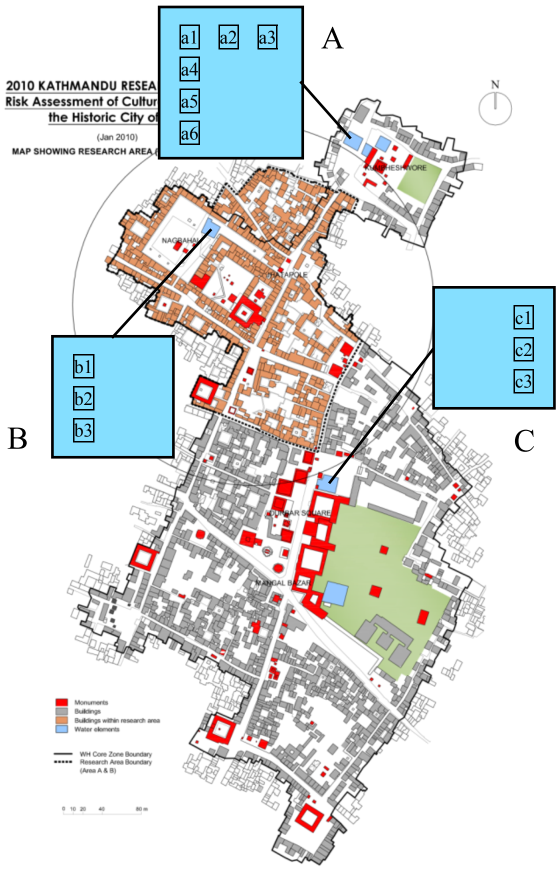


Fig. 1 Location of Hitis in the target area

Chart. 2 Results of simple measurement

Hiti	Tap	Filling time (sec. / lit.)	Amount of discharge (lit. / sec.)
Konti Hiti (A)	a1	2.10	0.48
	a2	1.35	0.74
	a3	1.50	0.67
	a4	11.90	0.08
	a5	1.35	0.74
	a6	0.70	1.42
Total			**4.13**
Nagbaha Hiti (B)	b1	0.95	1.05
	b2	1.00	1.00
	b3	2.95	0.34
Total			**2.39**
Mangah Hiti (C)	c1	1.00	1.00
	c2	0.85	1.18
	c3	1.10	0.91
Total			**3.09**

(3) Assessing the inaccessibility of the fire engine

The measuring investigation was conducted with measurement of the actual length that width, height, and the presence of obstacles on each street in the world heritage core zone of Patan. The purpose of the investigation was assessing the inaccessibility of the fire engine on each street in the target area.

a) Method of the measurement and the assessment

The actual length such as width and height on each street in the world heritage core zone of Patan were measured. The presence of obstacles was also surveyed. The actual size of the fire engine of Lalitpur Fire Station is 2.25m wide and 2.85m tall. Therefore, the fire engine cannot pass through either street narrower than 2.5m or underpasses lower than 3m, and the points on each street that the obstacles there are. Thus, such streets can be considered as inaccessible routes for the fire engine.

b) Results of the investigation

The results of assessment are shown in Fig. 2 as the map of inaccessible points for the fire engine. According to the result of assessment, the world heritage core zone of Patan has several narrow streets and the obstacles which the fire engine cannot pass through.

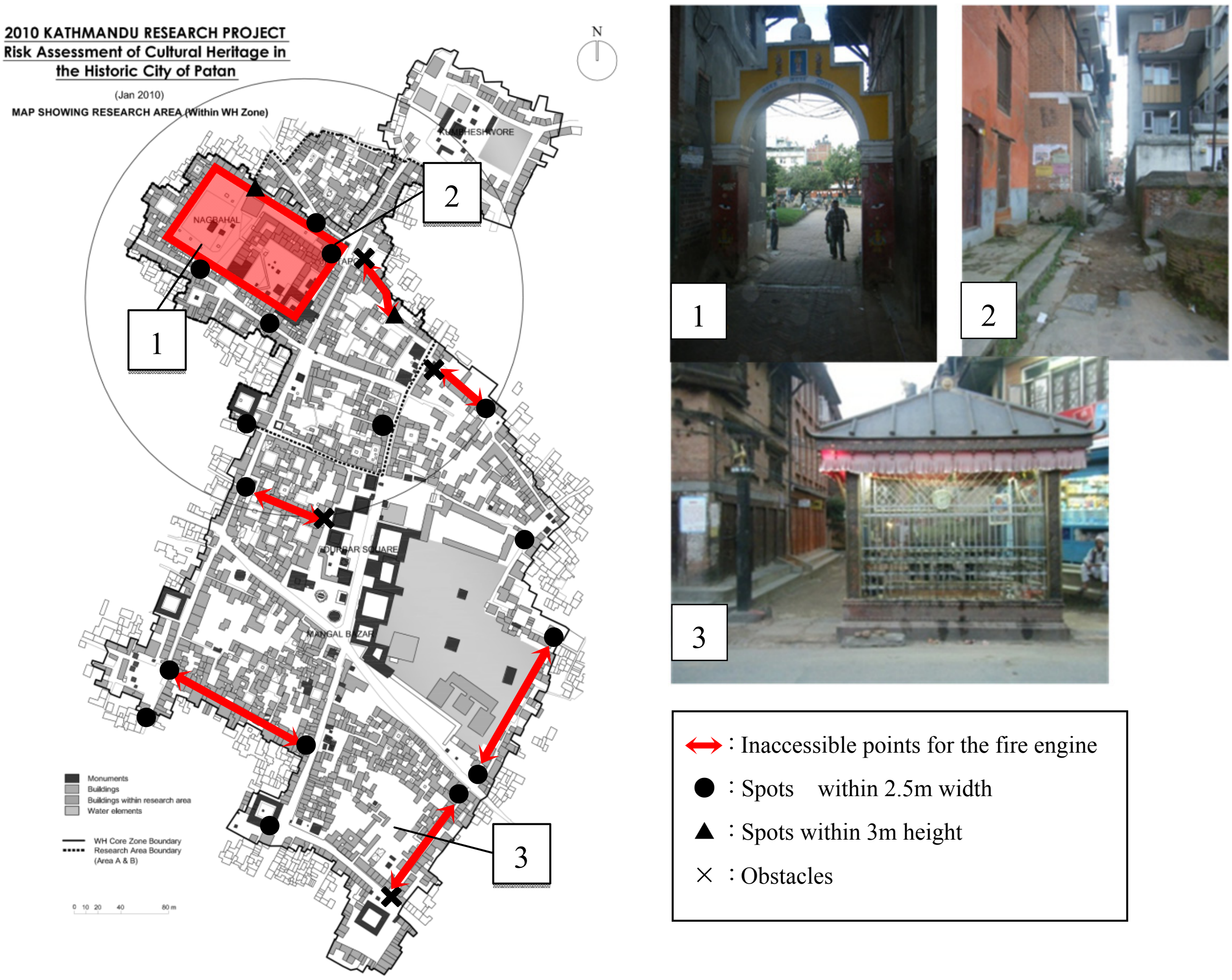


Fig. 2 Inaccessible points for the fire engine

(4) Assessing difficult areas for firefighting

a) Concept of difficult areas for firefighting

The difficult areas for firefighting are identified from two perspectives: lack of water resources and inaccessibility of the fire engine.

The concept model of difficult areas for firefighting is shown in Fig.3.

b) Method of assessment

Each of hoses equipped in the fire engine is 35m long. The fire engine with 8 hoses and maximum hose length is 280m with all of them connected. Where the streets meander, the maximum reach from a firefighting water resource such as Hiti is 200m, approximately $1/\sqrt{2}$ of 280m. Therefore, it can be assumed that water could be provided anywhere within a 200m radius from a water resource. In other words, water supply would be insufficient in the areas further than 200m from any water resources.

Konti Hiti can be considered as effective water resource for firefighting at a situation of fire spreading, because the capacity of tank on the fire engine is 2,400 liters, which would run out after only 10 minutes of firefighting.

c) Results of the risk identification

The results of the assessment are shown in Fig.4 - Fig.6 as the map of difficult areas for firefighting. Fig.4 indicated difficult areas for firefighting at normal situation. Fig.5 indicated difficult areas for firefighting at a situation of fire spreading. At the situation, water supply would be insufficient in the areas further than 200m from Konti Hiti. By utilizing all Hitis in the target area as water resources, sustainable firefighting activities would be possible. Fig.6 indicated difficult areas for firefighting at a situation in earthquake. At the situation, it is highly expected that the difficult areas for firefighting would be extended and isolated with all access routes blocked by collapsed buildings.

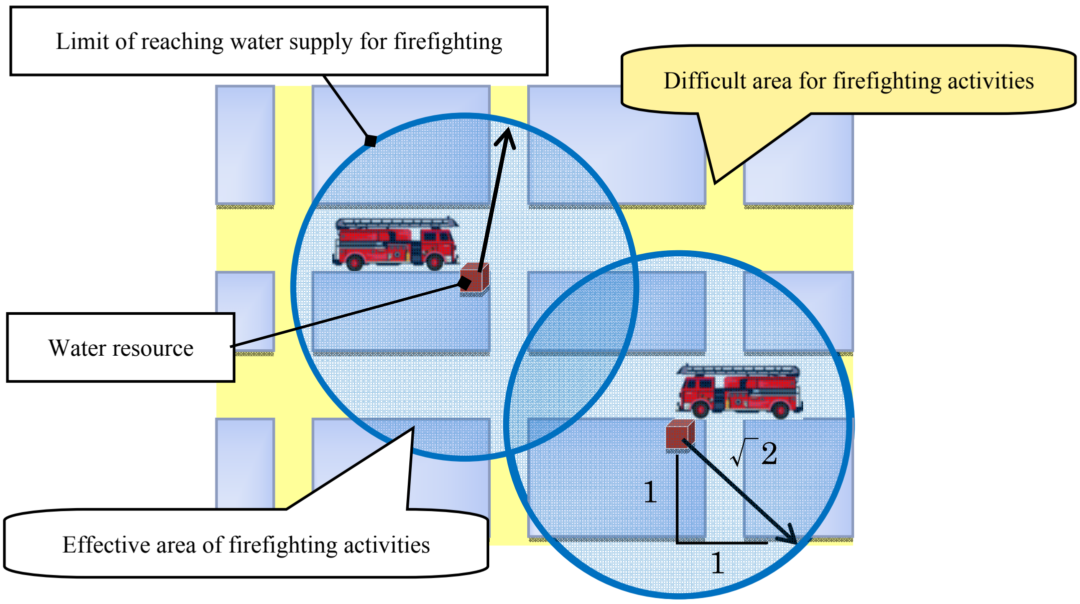


Fig.3 Concept of difficult areas for firefighting

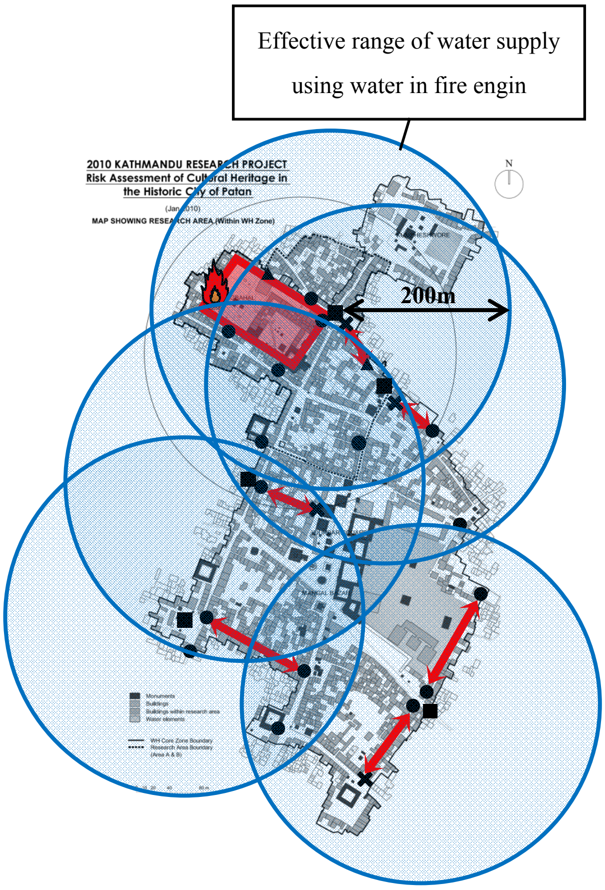


Fig.4 Difficult areas for firefighting at normal times

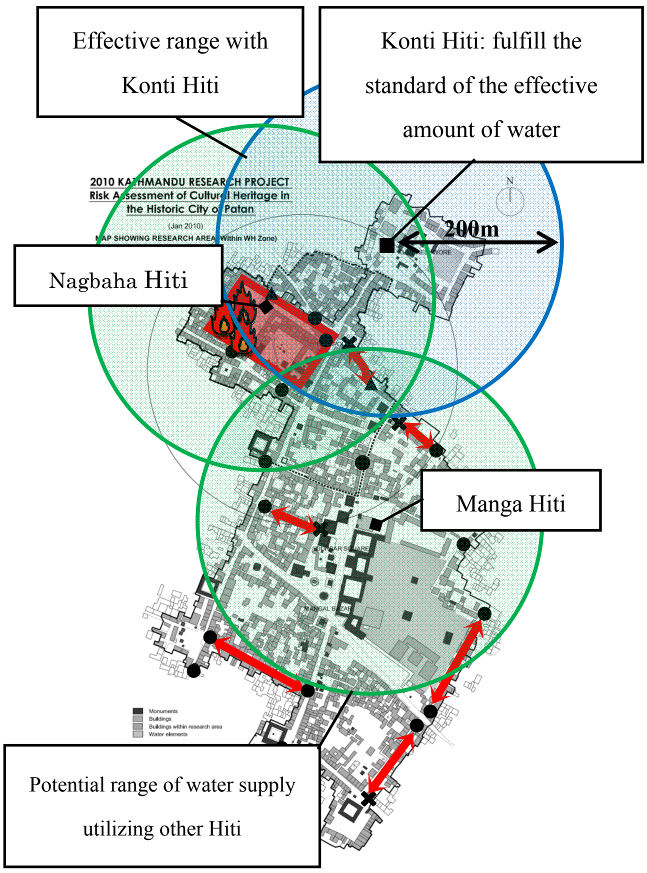
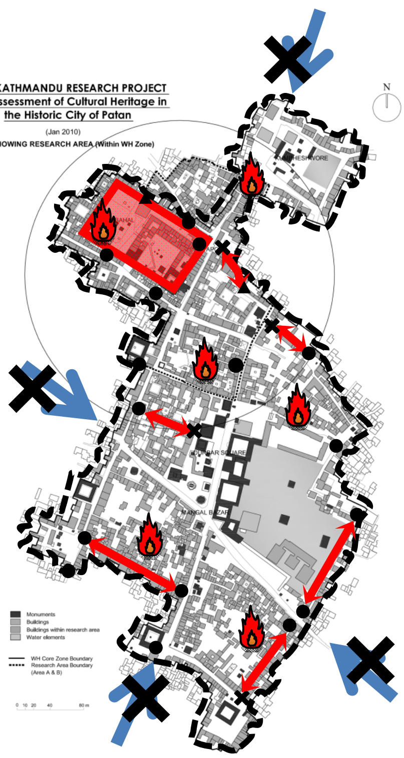


Fig.5 Difficult areas for firefighting at a situation of fire spreading



↔ : Inaccessible points for the fire engine

● : Spots within 2.5m width

▲ : Spots within 3m height

× : Obstacles

■ : Disposition of fire engine

← : Roads connecting to WH Core Zone

Fig.6 Difficult areas for firefighting at a situation in post earthquake fire

4-3.3. Survey for Community activities

Hiroki Akehara
Takeyuki Okubo
Masahiro Yoshida
Haruki Nagashima

(1) Research of Community around the Hiti

a) Purpose of Research

In order to mitigate the damage of disaster, there is a limitation only to rely on "the help by a public institution" in case of serious situation, and "to help oneself" and "to help each other" is required. The people's relationship in a local community is one of the major factors to determine the strength to mitigate the disaster.

61 traditional water supplies called “Hiti” are located in Patan area, and these water are used more than city water for daily life from ancient times. The values of the water system development may be read in historical, technological, aesthetic, as well as anthropological significance and value. In the world heritage core zone, there are 3 major Hitis: Manga Hiti, Konti Hiti, and Nagbaha Hiti. They tell the history of this development of urban water supply and irrigation services. Construction, extension, renovation and maintenance of the traditional Hiti system continued with community and neighborhood as well as family level participation from ancient times in Patan. Most of the residents' life has deep relationship with Hiti, and they are one of the centers of the residents' cooperative activities.

This research was conducted to investigate the activities of the each community around the Hiti.

b) Selection of 4 Hitis as research targets

Manga Hiti is located in the Patan Durbar Square Monuments Zone, Konti Hiti faces Kumbheshvar temple, and Nagbaha Hiti is located near to the area including Golden Temple. Thus they have historical and cultural value. On the other hand, they have applied by local residents in their daily life from ancient time. Therefore, they include the role as a collaboration space among local community. In terms of potential for emergency water supply, if rehabilitated, Konti Hiti should be the best but for the given site, Nagbaha Hiti offers advantage of gravity feed. Nagbaha Hiti however has a very low discharge and is already dry even after a recent rehabilitation. In that context, Manga Hiti which has a greater discharge but needs pumping and piping arrangements to feed to site may also be a credible option. Only Alok Hiti is located outside of core zone, but it provides most amount of water around Jhatapo area and also many amounts of consumption.[1)] There is community around Alok Hiti and the community members maintain it every day by themselves, and such activities are also conducted by the community. Although outside the world heritage core zone (Fig. 1), it was selected as a research target.

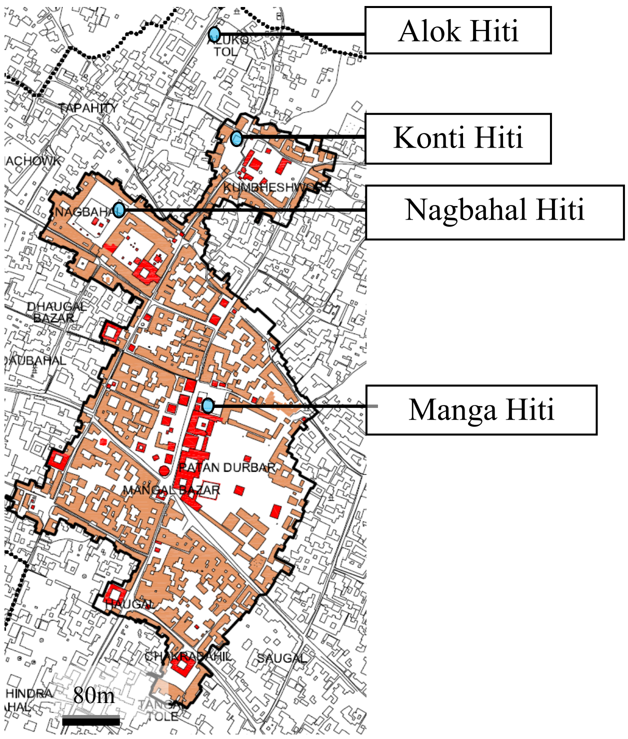


Fig. 1 World Heritage site and the Target Hiti

(2) Challenges and Countermeasures for Disaster Mitigation

a) Interview Research

The interview research was conducted to understand the community activities around each Hiti.

The name of the community around each Hiti, the name of the chairperson, and the date of research are shown in Chart. 1.

Chart. 1 Objects and dates of interview research

Name of community	Candidate	The researched date
Kubheswar Community (Konti Hiti)	Aloj Kumar Khadgis	2011/08/21
Mangal Tol Sudha Shangh (Manga Hiti)	Human Shrestha	2011/08/21
Young Stars Club (Nagbahal Hiti)	Ghanbahadull Shakya	2011/08/10
Alok Hiti Preventing and Water Supply Consume Community (Alok Hiti)	Naresh Shakya	2011/08/16

b) Outline of community around the Hiti

The summary of research outcomes are shown in Chart. 2.

Some communities raise funds from city office and international association. According to hearing investigation, after they apply to government for funds, governments decide which community should get funds by considering current condition of the community.

Chart. 2 Outline of community around the Hiti

Name of Hiti	Number of affiliation	Public support (Budget)	The purpose of activity
Konti Hiti	11 persons. (Community members are chosen by election from the 300 persons, except a chairperson and another one)	20%:City office(200000Rs) 20%:Community 60%:LGCDP (Local Government Community Development Project)	Religion and life are united. The water environment is improved for preservation of historical and traditional customs.
Manga hiti	112 persons (15 persons are chosen by vote)	None	Requests from residents, such as environmental preservation, health improvement, removing pollution, and water maintenance are solved.
Nagbahal Hiti	34 persons (3 of Chairman collect a budget and pay for 2 of cleaning persons for Hiti)	American Embassy paid the support money to build the Hiti	To improve the water quality of Hiti, new system introduced by their activities.
Alok Hiti	210 persons (Three sections with seven persons take charge)	The water tank was donated by the municipality.	Activities are conducted to keep the water environment for residents.

c) Introduction of water supply system with Hiti

The community around Alok Hiti has one of the advanced water supply system to get water on daily basis. They established the building with water strage tank (Pic. 1). The system supplies water from water taps (Pic. 2) of Hiti to each pipe for the houses (Pic. 3) by using electronic moter (Pic. 4).

Pic. 1 Building with water strage tank

Pic. 2 water taps of Alok Hiti

Pic. 3 Pipe for the houses

Pic. 4 Electronic moter

4-4. Current Status and Issues concerning disaster management of cultural heritage in Nepal

Rohit Jigyasu
Naoko Itaya

4-4.1 Current Status and Issues confronting Cultural Heritage protection and management in Nepal and Kathmandu valley in particular

(1) Legal provisions for the classification of monuments

The legal provisions for the conservation, protection and management of cultural property are based on the Ancient Monument Preservation Act (AMPA) 1956, its subsequent amendments (the fifth amendment in 1996) and the Ancient Monument Preservation Rules 1988. As per this act, Article 3(a) sub-article (i) states that 'From the viewpoint of ownership, the ancient monuments shall be classified In two categories as public ancient monuments and private ancient monuments". Sub-article (ii) states "From the view point of Importance, the ancient monuments shall be classified in three categories as of international importance, of national importance and of local importance". The monuments would need to be categorized as 'Classification 1', 'Classification II' and 'Classification III' respectively according to their antiquity, significance, nature, art and architecture etc. Specific criteria was developed for each classification. Refer appendix for details of these criteria for classification. However all these provisions have not been implemented yet.

Monuments / Monument Zones of international importance protected by the Department of Archaeology:

The AMPA gives the Department of Archaeology the legal provisions to declare a monument or area to be a Protected Monument Zone (PMZ). The Department of Archaeology is subsequently responsible for the protection of the site, including the prescription of building bylaws, approving requests for building permits and for any other construction activities within the zone. The Department of Archaeology is given the authority to stop inappropriate and/or illegal building activities and to request for the demolition of unauthorized constructions.

The seven Monument Zones of the Kathmandu Valley World Heritage property have been declared PMZs and the boundaries have been gazetted under the provisions of the AMPA. The Department of Archaeology is therefore responsible for the preservation of the areas comprising the property inscribed on the World Heritage List.

(2) Protection of urban cultural heritage besides monuments

The Local Self-Governance Act (LSGA) 1999 is the principle act for the decentralization of powers to the District Development Committees (DDC), the Municipalities and the Village

Development Committees (VDC). The Local Self Governance Act gives the elected local government bodies the function and duty – to varying degree - to record, maintain and preserve the tangible and intangible heritage within their area of jurisdiction.

Under the LSGA, the municipalities are given the mandatory function and duty to prepare an inventory of the culturally significant places and maintain and protect them. In respect to the physical development, the municipalities must prepare a land-use map and must approve design permits. The wards of the municipalities are given the functions, duties and powers to help preserve monuments and important sites within their ward. These functions and duties however need to be coordinated with the Department of Archaeology.

(3) Lalitpur Sub-Metropolitan City:

The municipal services in Lalitpur started with the establishment of Chhevdel Adda (Cleaning Office) in 1918, which was later incorporated in Municipality in 1996. On 1 February 1996, the city was declared a Sub-Metropolitan, one of only two in the country.

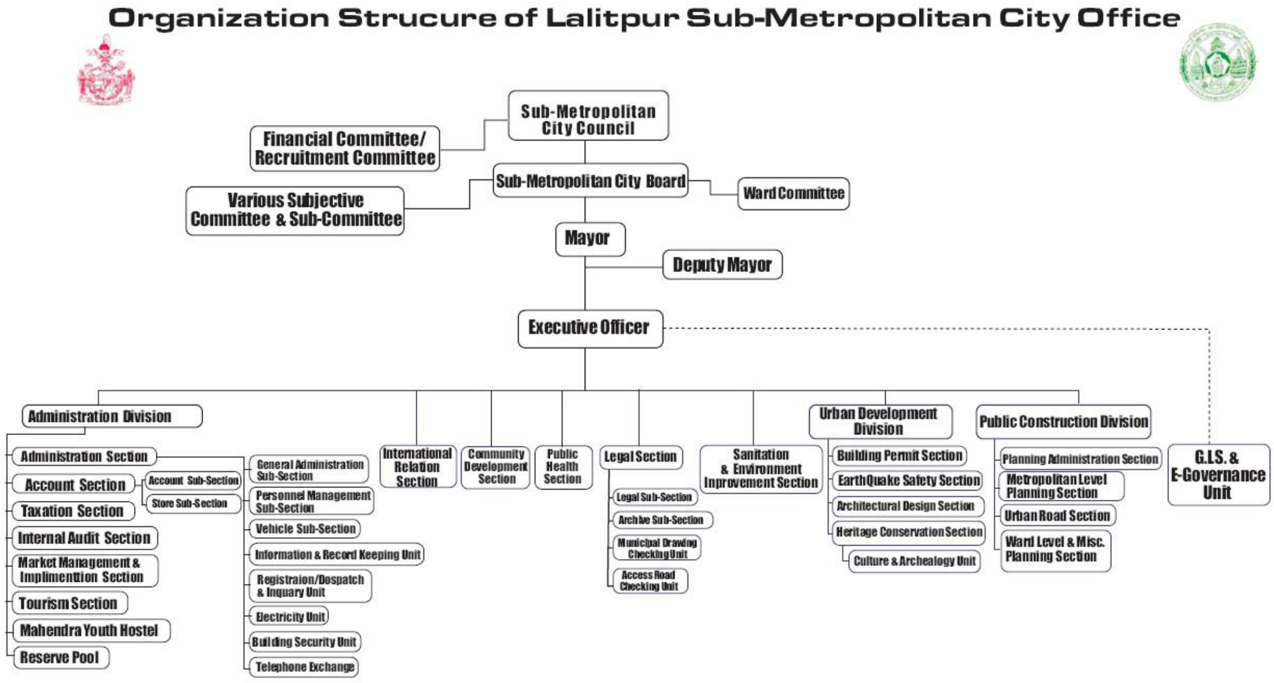
Lalitpur Sub-Metropolitan City (LSMC) is the sole agency for providing municipal services and carrying out urban development works in the city of Lalitpur. The Local Self Governance Act of 2055 (1999), through decentralization, has empowered the local bodies such as municipalities and VDCs to formulate and implement their own development plans and programs. As per the LSGA 1999, LSMC as the local government is a municipal service provider and engaged in public welfare, service and infrastructure development activities. As per the provision of LSGA 1999, the municipalities as the local government have to perform mandatory and optional municipal functions.

The municipal organization is constituted by a city council at the apex of its structural hierarchy. Municipal council constitute members that include mayor and deputy mayor, ward chairman, ward member of each ward committee, and prominent people of the society of different backgrounds.

Municipal board is constituted by mayor, deputy mayor and various committees and sub-committees. This also includes ward committee comprising one chairman and four ward members (including one female member) who are nominated through local election. The board convenes regularly to discuss and make important decisions on the day to day activities of the municipality.

Within the organization structure, there are various divisions and sections to carry out the day to day municipal functions. There are main three divisions in LSMC, Administrative, Urban Development and Public Construction. International relation, community development, public health, sanitation and environment are the sections. In addition to this there are several sections and sub-sections within the divisions as well. Recently, a unit called GIS and E-Governance has also been added which is a new concept in the municipal organization in Nepal.

[Source: City Profile, Lalitpur Sub-Metropolitan City, 2005]



[Source: City Profile, Lalitpur Sub-Metropolitan City, 2005]

4-4.2 Current Status and issues concerning the impact of existing Urban policies and planning on Core and Buffer zones of the Patan World Heritage Site

National Building Code (of Nepal) has direct bearing on urban policies and planning that impact core and buffer zones of Patan World Heritage Site.

The National Building Code, which was initially prepared in 1994, has recently come into effect and the municipalities have started enforcing the code. The code emphasises seismic stability, yet clearly makes provisions for load bearing masonry

As part of the comprehensive site management plan, special byelaws have been prepared for Patan Durbar Square Monument Zone and an integral part of the Municipal Building Bylaws that respect the heritage values. The responsibility for the enforcement of these bylaws lies with the heritage section of Lalitpur Sub-Metropolitan City.

The buffer zone encompasses already existing zones within the Municipal Zoning Plan, comprising of the "Conservation Sub-Zone" and the "Mixed Old Settlement Sub-Zone" which corresponds to the whole historic city of Lalitpur. The municipal bylaws for the "Conservation Sub-Zone" and the "Mixed Old Settlement Sub-Zone" are to be enforced for the area within the buffer zone.

However, these byelaws have still not been implemented by the responsible agency Lalitpur municipality.

Land use regulations and development are a function of both the city and natioanl government**. The Town Development Act (TDA)** mainly deals with the reconstruction, extension and development of towns. In the Kathmandu Valley, the Kathmandu Valley Town Development Committee (KVTDC) is responsible for implementing the Act.

The Guthi Corporation Act (GCA) was established in 1964, nationalizing all Guthis to a centrally organized unit, the Guthi Sansthan. The Guthi Sansthan is still the legal owner of many monuments and historic buildings within the PMZs.

4-4.3 The existing role of Central Government, army, municipality, owners and residents for disaster risk management of urban cultural heritage

(1) Central Government Level

The Ministry of Home Affairs through its department of Narcotics, Drug Control and Disaster Management, is the national agency responsible for disaster management in Nepal. Formulation of national policies and their implementation, preparedness and disaster mitigation, immediate rescue and relief works, data collection and dissemination, collection and distribution of funds and resources are the vital functions of the Ministry. Its network to cope with natural disasters is integrated by 75 Chief District Officers, one in each of the administrative districts, who act as the crisis manager in the event of natural disasters.

The main function of the department is to actively and efficiently co-ordinate and carry out emergency preparedness and disaster management activities with concerned agencies.

(2) Army and Police

The Royal Nepal Army and Nepal Police play important roles in rescue operations. Police officials collect first-hand information of a disaster and inform concerned officials. In the event of a catastrophic disaster, Nepal Police establish command posts to facilitate rescue operations. Moreover, Nepal Police Personnel collect most of the disaster data and information.

(3) Municipality

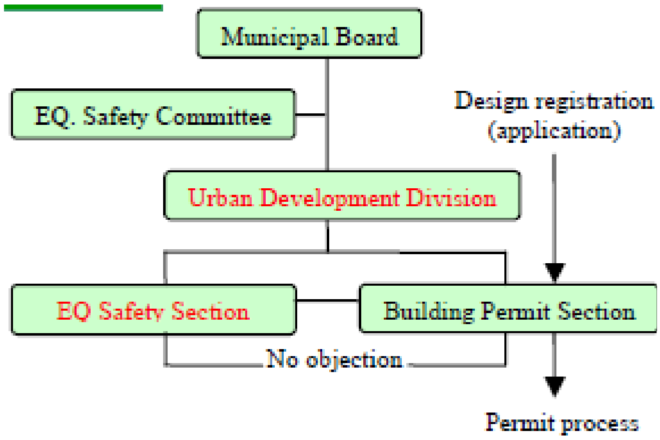
On the occasion of Earthquake Safety Day on January 16, 2003, LSMC announced its plan to implement NBC in all of its building permit process. LSMC became the first municipality inNepal to implement NBC and it was done before the implementation was made mandatory.

The decision was historic in the sense that it not only awakened the government to enforce NBC but also encouraged other municipalities on the necessity of building code implementation. It also proved that implementation of NBC can be done by determination irrespective of legal constraints.

(4) Earthquake Safety Section

Initially application of NBC was carried out by the Technical Cell formed under the Engineering Sub-Committee to look after regular building permit process. The cell was composed of a group of Municipal engineers, engineers from DUDBC, NSET, NESF and NEA and was functional for six months. The applications for building permit were first verified for conformity with building by-laws. Then, they were checked by the Technical Cell for conformity with NBC. However, it was soon realized that a separate section was needed in order to increase the efficiency and performance. On November 27, 2003, Earthquake Safety Section was established. It worked in consultation with the Earthquake Safety Committee (ESC), which was comprised of engineers from DUDBC, academics and other professionals to help LSMC in technical matters related to NBC.

The organization chart for initial arrangement and the current arrangement for checking compliance with NBC are shown schematically in the following figure



Following boxes explain the role/divisions among the Building Permit and Earthquake Safety Sections and House Owners

Building Permit Section:

- To check/verify architectural drawings/designs as per building by-laws and register the file.
- Notice to neighbors and field verification of plot/access roads and other legal documents;
- Submit Field Verification Report;
- Recommend for building permit to the Executive Officer/Mayor;
- To check and verify at Tie Beam level as per by-laws;
- To check and verify buildings for Completion Certificate as per by-laws
- To monitor construction fields regularly;
- To inform city dwellers about the permit and planning processes; and
- To formulate new systems/mechanisms for effective enforcement of building by-laws

Earthquake Safety Section

- To check/verify structural drawings/designs as per NBC and to recommend 'No Objection' for further process of Building Permit.
- To give suggestions to house owners and masons regarding earthquake safe technology in building constructions;
- To monitor construction fields regularly
- To conduct training/orientation programs to designers, technicians, contractors and house owners;
- To carry out awareness programs on earthquake safety to general public;
- To coordinate between ward level disaster management committees with Municipal Level Committee
- To work closely together with supporting organizations like DUDBC and UN agencies for earthquake risk reduction and preparedness; and
- To formulate new program proposals for effective implementation of NBC.

House Owners

- To prepare and submit structural design and drawings as per NBC;
- To follow the suggestions/comments given by the Earthquake Safety Section in design as well as in the construction.
- To apply earthquake safe techniques in construction field as per approved design;
- To use the quality construction materials in the field;
- To give special attention to quality of construction works and make it mandatory to use vibrator, mixtures, compactor etc.
- To carry out construction under the supervision of skilled technicians, and
- To employ trained masons in the construction

General guidelines on building permit and earthquake safety published by LSMC is the guiding document which has all the necessary guidelines and forms necessary for the building permit process. The document is informative and comprehensive as it not only outlines the necessary process but also provides information about relevant building codes, roles and responsibilities of different sections in the Municipality, designers and house owners.

(5) Challenges

- One of the main challenge is that the process of seeking building permit is very complicate with the reason that many house owners tend to take illegal shortcuts.
- There is no proper system of ground verification that construction at each step is according to approved drawings.
- There is lack of knowledge among masons and contractors on earthquake resistant building construction technologies.
- Residents tend to add stories over existing traditional structures thereby increasing their vulnerability to earthquakes.
- There is negligence and unwillingness of house owners to apply earthquake resistant

technologies due to perceived additional costs in construction.

- Because of unfair competition between local masons and contractors, they work at low costs by not following National Building Codes.
- MOST IMPORTANTLY, THESE BUILDING BYELAWS DO NOT TAKE INTO CONSIDERATION THE HERITAGE VALUES FOR NEW CONSTRUCTIONS AS WELL AS ASSESSMENT, REPAIR AND STRENGTHEINING OF TRADITIONAL HOUSES.
- IN PATAN MUNICIPALITY, THERE IS VIRTUALLY NO COORDINATION BETWEEN HERITAGE SECTION AND THE EARTHQUAKE SAFETY AND BUILDING PERMIT SECTIONS.
- ALTHOUGH COMPREHENSIVE MANAGEMENT PLAN FOR WORLD HERITAGE MONUMENT ZONES OF KATHMANDU VALLEY ADVOCATES RISK MANAGEMENT, THE PLAN HAS NOT BEEN PREPARED YET.

5. Disaster Risk Management Proposals

5.1 Structural Reinforcement of Traditional Buildings and Open Spaces

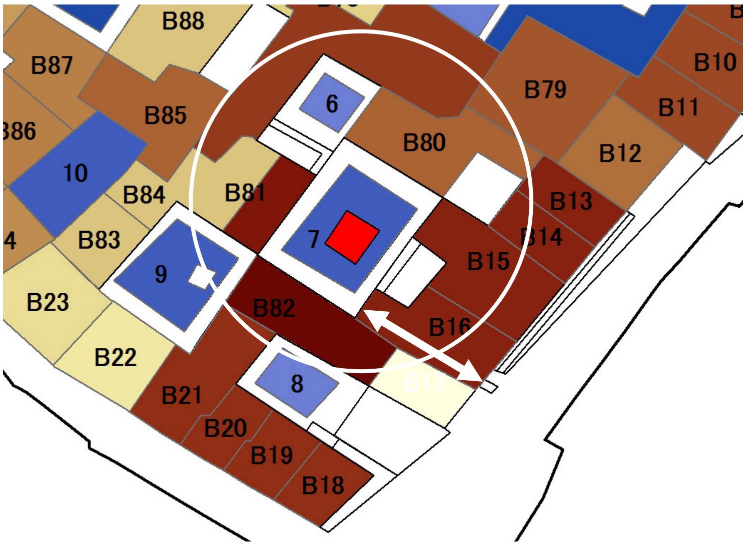
Hari Ram Parajuli
Prem Nath Maskey
Junji Kiyono

As a result of the seismic vulnerability assessment the damage index of each and every building, brick masonry as well as RC building, is determined and plotted in the base map as shown in Fig. 5-1.1. The value of the buildings, as determined for the buildings is presented in Fig. 5-1.2.

Fig. 5-1.1 Damage Index of the buildings

Fig. 5-1.2 Value of the buildings

The location of the open spaces and the underpasses, crucial for evacuation planning in case of earthquake Disaster is shown in Fig. 5-1.3.



High Value Chaitya located Courtyard 7

Surrounding Buildings are high value and vulnerable

Courtyard entrance	No of people
	40 ~
	30～39
	20～29
	10～19
	0～9

When Disaster occurred over 30 residents rush to this Underpass for evacuation

Fig. 5-1.3 Case study place for Disaster Risk Management

5-1.1 Strengthening of buildings

In reference with Fig. 5-1.1 and 5-.2, it is evident that the buildings B82, B81, B16 and B15. Around the open space 7, B16, B15, B14 and B13 by the side of the main road to Kumbheshwor Complex, and B50, B48 and B46 around the open space 20 have high values. The damage indexes of the buildings are the result of the vulnerability assessment including the structural analysis using the seismic input obtained from the seismic hazard analysis of the study area, and represent the future damage scenario of the study area in case of earthquakes. The values of the buildings are determined based on the location, social and heritage importance. These vulnerable buildings with high value need to be strengthened to mitigate the possible severe damages and loss during the disaster. The possible technique to be applied for strengthening of such buildings is suggested in reference with a building, B47. The typical plan and elevations of building B47 are shown in Fig. 5-1.4.

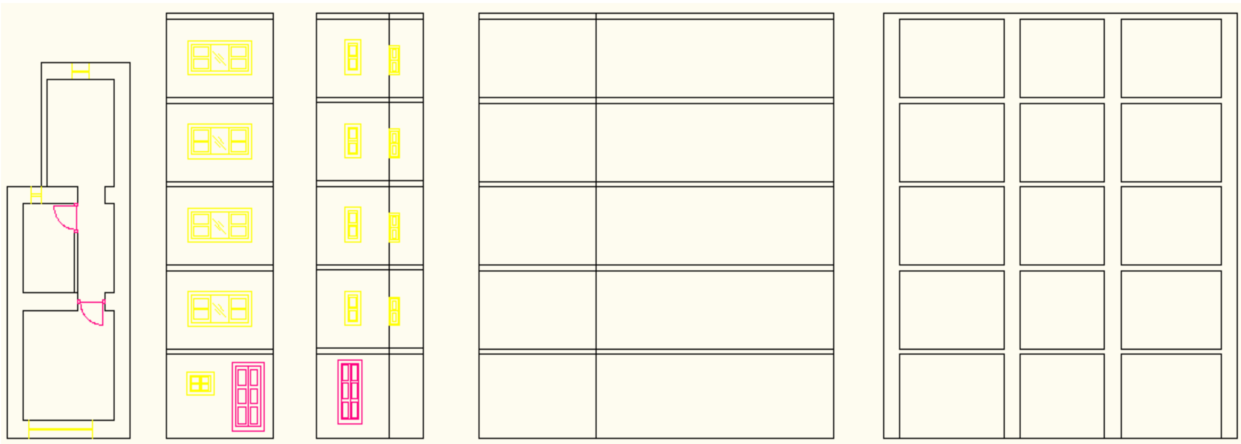


Fig. 5-1.4 Plan, Front elevation, back elevation, left side elevation and right side elevation of B47

For illustration, the vulnerability assessment of the building (B47) is carried out by modeling the building using finite element method (FEM) for the structural analysis. The stress contours in the buildings as the result of the analysis are shown in Fig. 5-1.5. The stresses in the wall panels at critical locations are found to exceed the permissible values of stress in brick masonry.

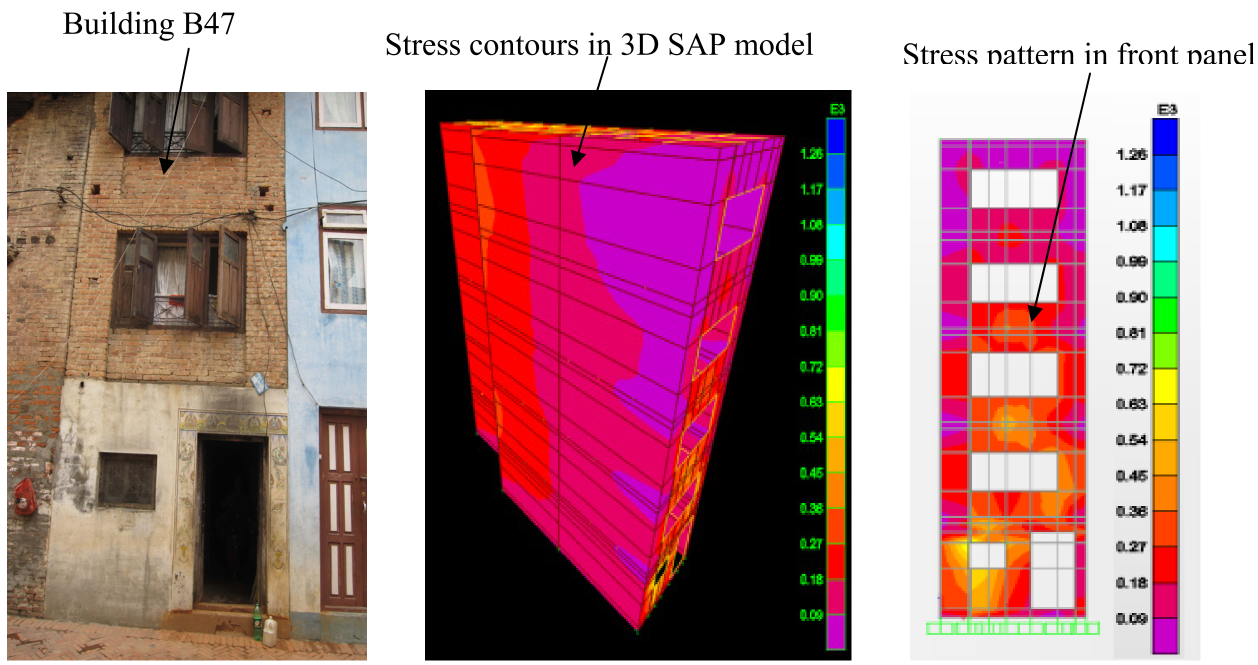


Fig. 5-1.5 Photo, stress contours and stress pattern in panel of Building B47

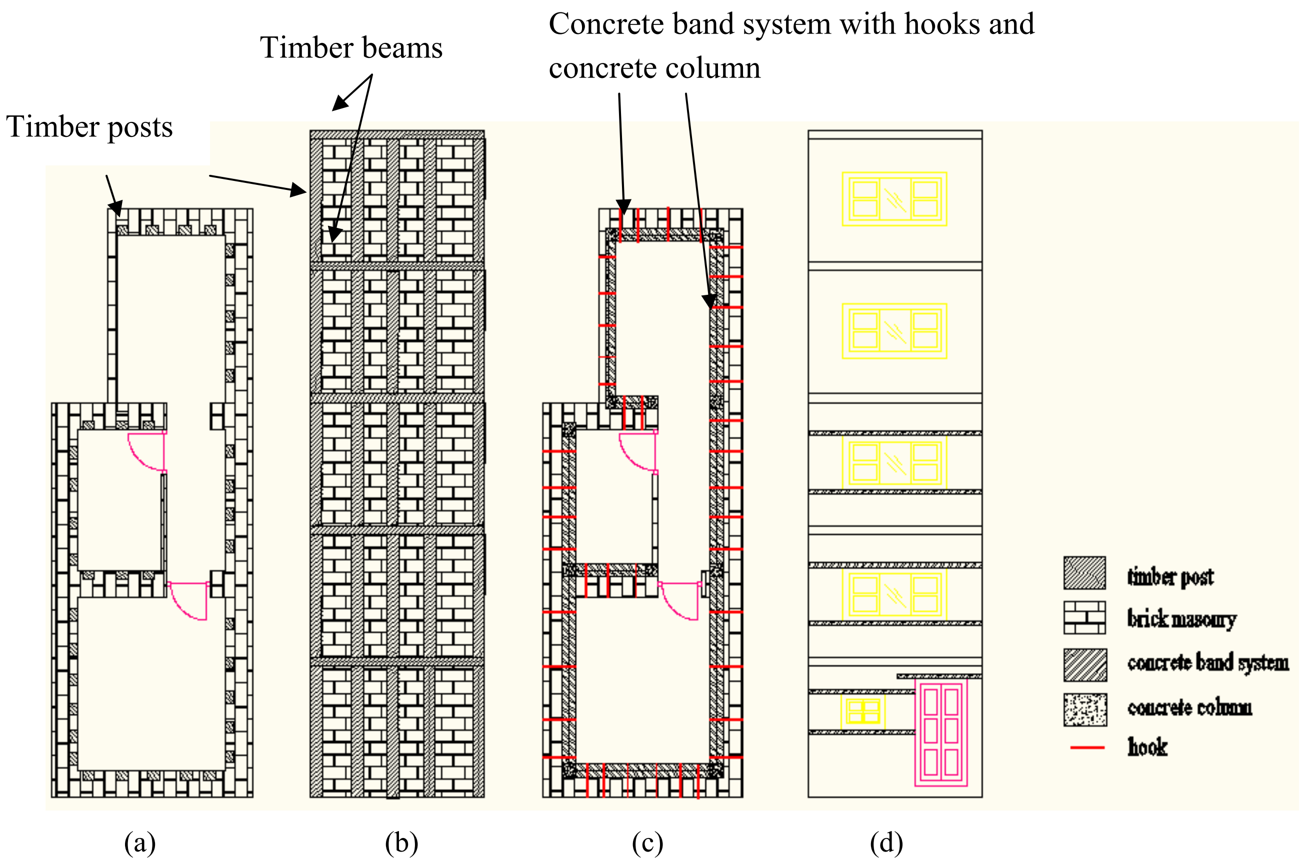


Fig. 5-1.6 Two simple techniques of strengthening the building B47 (a) plan showing the vertical timber upright posts (b) elevation showing the timber framing, (c) plan showing concrete columns and beams anchored with the existing walls, (d) elevation showing the concrete bands at the strategic levels.

The maximum stress values are about 0.55 MPa, which are greater than the permissible value of 0.3 MPa for brick masonry, showing the need for strengthening of the building. The strengthening of the building can be achieved by various methods depending upon factors, like importance of structure, cost, integrity to other structures etc. Out of many, two simple and possible methods are proposed. The first method consists of providing vertical timber upright posts as additional members to the wall along with the brick masonry at a regular spacing, and the posts are connected with the horizontal timber beams at floor level as shown in Fig. 5-1.6 (a) and (b). This method is in the line of the traditional technology with indigenous materials, easily accepted by the local inhabitants. The second method consists of providing horizontal concrete bands at strategic levels in each storey. Such concrete bands are inserted by cutting the wall partially and then the wall is tied with the band by providing hooks. The horizontal concrete band is framed with the additional concrete columns provided at the corner points as shown in Fig. 5-1.6 (c) and (d).

5-1.2 Strengthening of open space and underpass

The open spaces and the underpasses, typical elements of the local community of the Kathmandu Valley, are important in terms of disaster management. The open spaces are excellent places for shelter for people in case of damages of the houses, and underpasses are the only available thorough ways for exit from the traditional courtyards. In this regard, the strengthening of the open spaces and the old, narrow but long under pass becomes equally important. Two suitable techniques for strengthening of the open spaces and underpasses are suggested herein.

In case of earthquake hazard, the open spaces and underpasss are most important, because the people have a general tendency to run away to the nearest open space in courtyard, and pass through the underpasses from the courtyard to a larger open space or more safe places. Hence the strengthening of buildings to protect courtyard like A and strengthening of the underpasses as B1 and B2 (Fig. 5-1.7and 5-1.8), which are the only ways out from the inner parts is very important. The buildings which are exposed to the courtyard must be framed with addition of RC columns to strengthen them as shown in Fig. 5-1.9. The under pass could be protected by providing concrete shell throughout as a rigid structure as shown in figure Fig. 5-1.9 (b). As a second option the timber frame could be provided as shown in figure 5-1.9 (c).

The principles of the strengthening techniques for the buildings, open spaces and the underpasses are proposed in general as the possible method of application, and also and in the line of tradition. Details of the techniques and the quantitative values are not given here, limiting this section as a part of the proposal. Different techniques with use of new materials and technology are also possible; however, they should match the existing structures and traditional methods as far as possible.

Fig. 5-1.7 Building stock plan with underpass passage B1 and B2

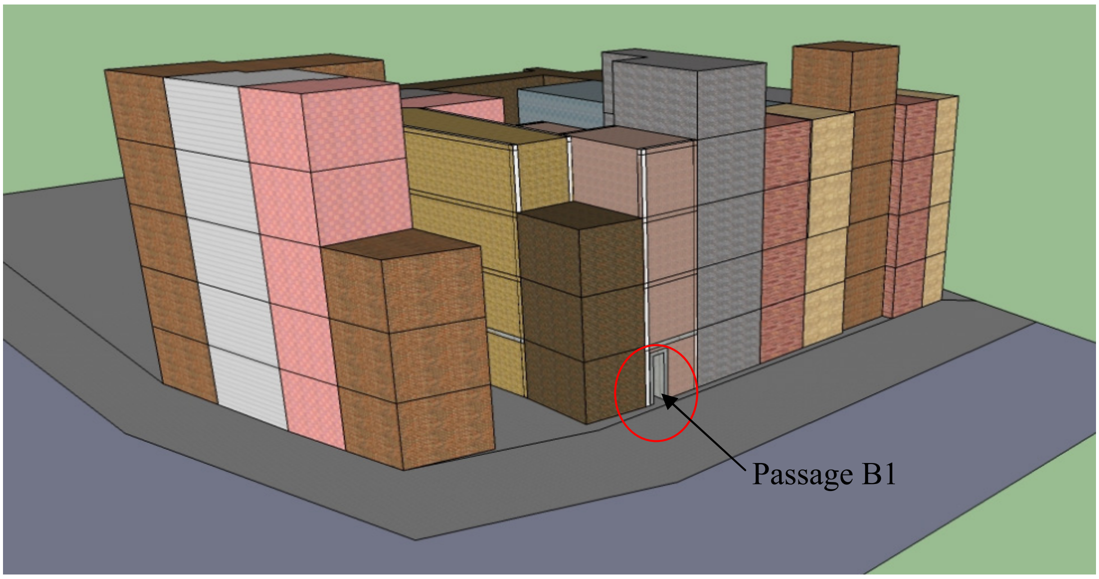
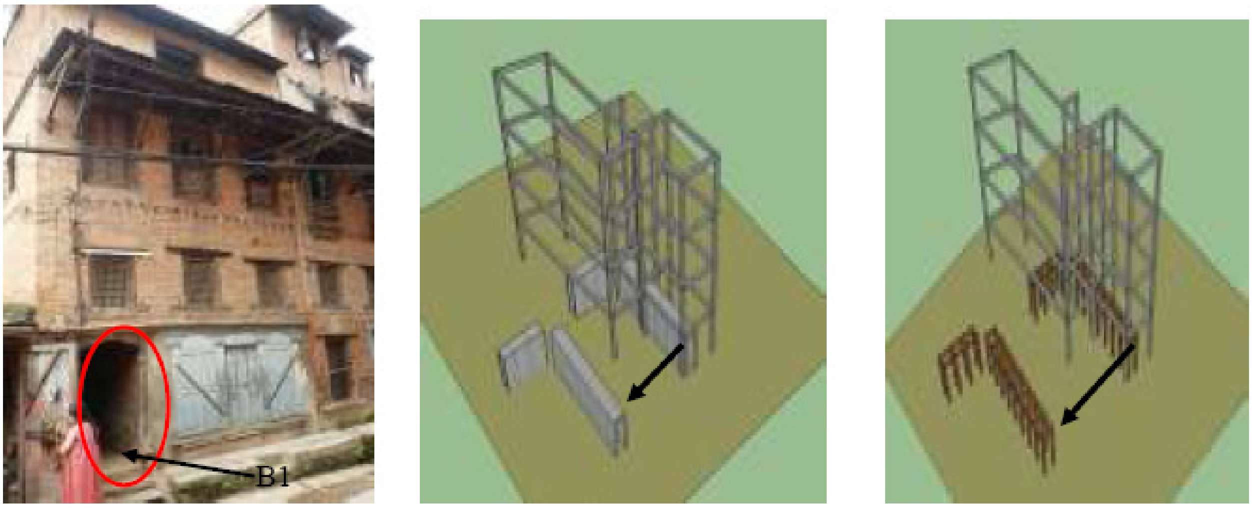


Fig. 5-1.8 Isometric view of the building stock with open space A and passage B1



(a) Photo showing underpass B1 (b) Concrete shell in under pass under pass (c) Timber frame in

Fig. 5-1.9 Strengthening techniques for under pass

5-2 Utilization of Cultural Heritage for Evacuation and Rescue

Takeyuki Okubo
Rohit Jigyasu
Naoko Itaya
Masahiro Yoshida

5-2.1 Summary

Many people would be affected in an event of earthquake in Kathmandu Valley. According to the record of the Nepal Bihar Earthquake in 1934, people ran into open spaces in panic. However, as large part of the old open spaces has been filled up with new buildings and walls surrounding private lands because of the increasing population, accessing them is not as easy as it used to be. A system for evacuation and rescue activities should beprepared as a large earthquake is anticipated in the near future.

Accordingly, designation of monasteries (Bahas and Bahis) and temples in the daily milieu as primary refuge areas, improvement of the safety of the access paths to these places, and utilization of cultural heritage components shared by the local communities (Patis, Sattels, wells, hitis etc.) for rescue activities, are recommended.

This article takes up evacuation plan from Jhatapo neighborhood as a case target area of investigation.

5-2.2 Preparedness for Evacuation

(1) Grounds of temples with functions as primary refuges

During the Hanshin Awaji Great Earthquake in January 1995, some people evacuated to the small parks (as large as 0.25 hectares used mainly by children and elderly people) whose service radius are approximately 250 meters. During the East Japan Great Earthquake in March 2011, shrines and temples in neighborhoods functioned as refuges. During emergent time after a disaster, safety of a city would be enhanced with preparation of primary refuge areas, where people could be immediately sheltered, as well as municipally designated city level refuge areas (Fig.1).

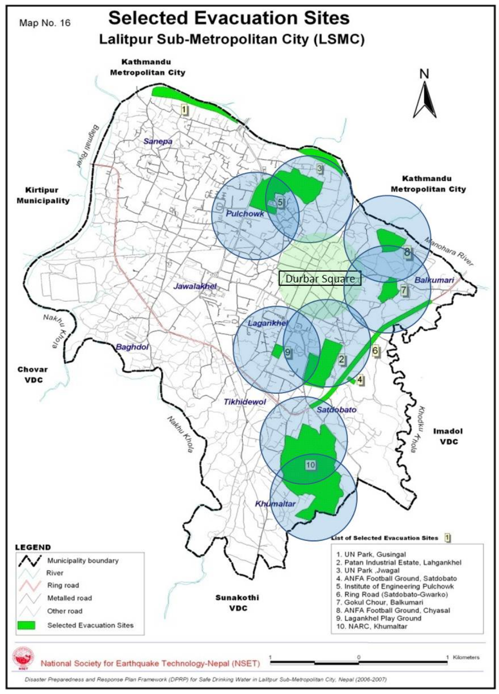


Fig.1 Municipally Designated City level Refuge Areas

The surroundings of the target area include several open spaces associated with temples that are closely related to local people's religious lives such as Kumbeshwar Complex and the open spaces around the Golden Temple as well as courtyards of Bahals and Bahils. These places may well have potentials for being utilized as primary refuge areas in times of disasters.

(2) Traditional urban structure that hinders the accessibility to primary refuge areas

Typically, Nepalese traditional urban structure has the following features.

- ✓ Each courtyard is surrounded by several houses.
- ✓ Courtyards are connected to one another through underpassages below houses.

When an earthquake occurs and these underpasses collapse, residents in buildings not facing the roads would not be able to evacuate. In this way the traditional urban structure of Nepal makes emergency evacuation difficult.

5-2.3 Preparation of evacuation plan "from Houses to Pattis and Sattels" (1st step)

(1) Disussion with local residents on available evacuation routes through Disaster Imagination Game (DIG)

In order to evacuate to primary refuge areas with relative ease during emergency, preparation of an evacuation plan and enforcement of daily drills are important. Local communities should actively participate in the preparation of an evacuation plan so that it is suitable to ground realities.

In order to have some clues for preparing an appropriate evacuation plan, a Disaster Imagination Game (DIG) was conducted in Jhatapo neighborhood in September 2010. The local residents who participated in the DIG stated their opinions as follows.

- ✓ Access paths for daily usage
  - ・ Motorcycles go through the northern access path leading to large courtyard.
  - ・ Pedestrians use the southern access path which is closer to the center of the area.
- ✓ What the residents think they should do in time of disaster
  - ・ The residents would wish to reach to the well as soon as possible so that they could immediately start fire fighting.
  - ・ The residents would want to immediately take refuge in the closest open spaces, anticipating the collapse of the buildings.
  - ・ Disabled, elderly and infantswould take more time for evacuation and need help from others.
- ✓ Evacuation routes selected by the residents
  - ・ Through the DIG, the residents selected the most appropriate routes out of various potential routes (Red arrow lines in Fig.2).

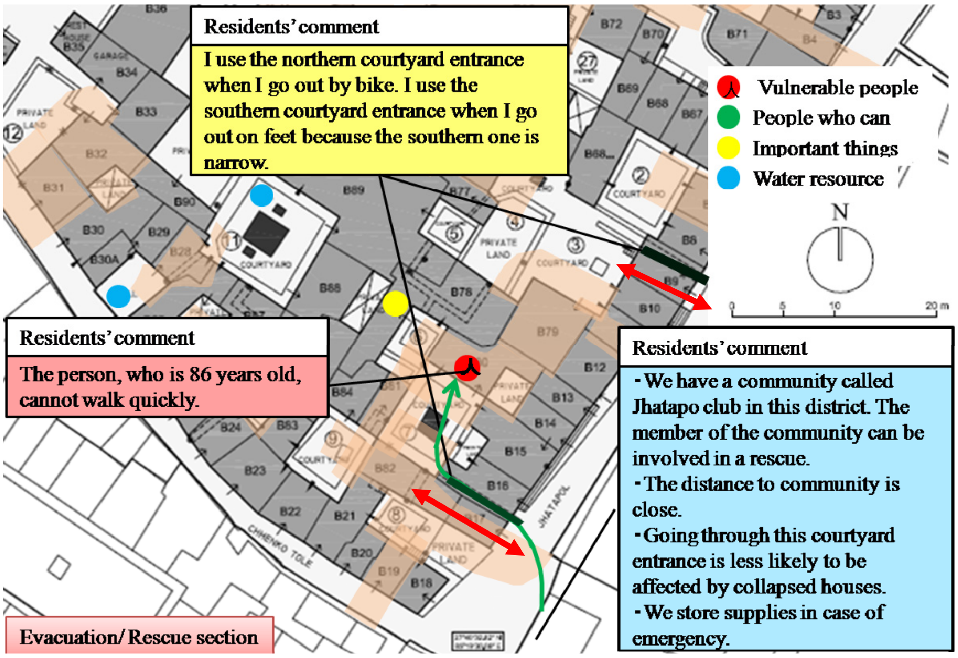


Fig.2 Disaster Imagination Game (DIG) in Jhatapo

DIG would be one of effective methods to prepare an evacuation plan with the participation of local communities .

(2) Calculation of the number of evacuees passing through the courtyards and access paths

Evacuation routes to the refuge areas in each neighborhood, which consist of access paths and courtyards, as well as buildings facing roads need to be reinforced. Among these, reinforcement of access paths is the least expensive and most feasible option. Access paths to be reinforced need to be prioritized to effectively improve safety with limited resources. This section describes the method to identify access paths where more number of evacuees are expected (Fig.3).

(a) Investigating the number of residents in the buildings facing the access paths.

(b) Dividing the number of residents in each building by the number of exits to calculate the average number of evacuees using an exit (In many cases, a building has exits on the road side and courtyard side).

(c) Calculating the number of evacuees who would use each access path, anticipating that the residents would evacuate to the refuge areas through the nearest courtyards.

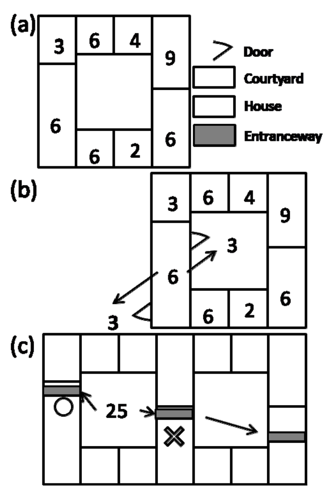


Fig.3

The number of expected evacuees who would use each of the access paths was calculated through a trial exercise (Fig.4). In this case, there are two important access paths indicated as red arrow lines in Fig.4, and these well correspond with routes of residents' selection in Fig.2.

The number of expected evacuees who would use each of the courtyard also was calculated through a trial exercise. There are two important courtyards indicated as red circles in Fig.4.

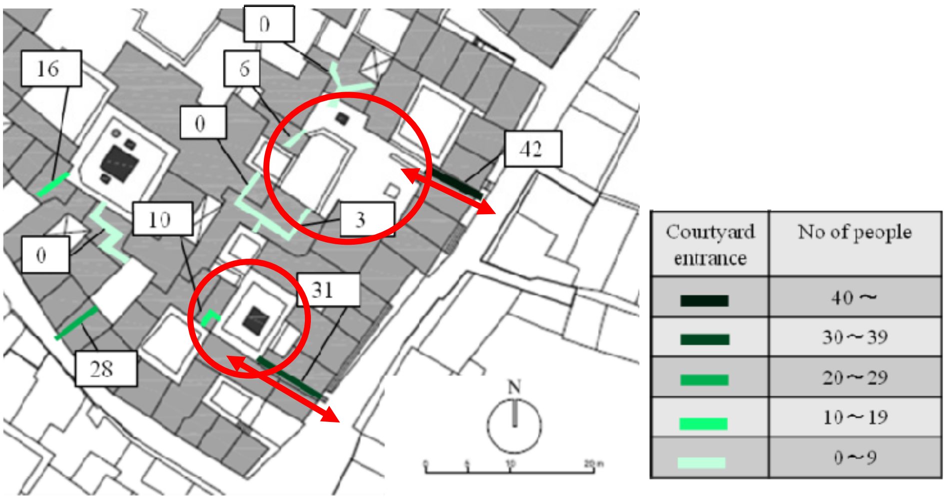


Fig.4 Trial calculation of numbers of evacuees through the access paths

(3) Selection of prioritized courtyard and access path to bereinforced for seismic safety

In the last phase of first step, we need to discuss the priority of courtyard and access path reinforced for seismic safety. For this purpose, maps based on research outcomes from “cultural value”, “structural vulnerability” and evacuation plan” are overlayed for reference.

5-2.4 Preparedness of “Pattis and Sattels” for Rescue Activities (2nd step)

(1) Establishing storage areas for rescue and salvage equipments in Patis and Sattels

During the Hanshin Awaji Great Earthquake, the sufferers had helped one another and worked on rescuing their neighbors from the underneath of debris until the public rescue teams arrived. In order for local residents to do rescue activities, there need to be rescue instruments such as shovels, bars, jacks and ropes. Moreover special equipments may be needed to salvage damaged/trapped cultural heritage components. It is effective to house the rescue and salvage instruments required in times of emergency at appropriate storage areas. On the street corners of Patan are Patis and Sattels; thecultural heritage components shared by the local communities. It is suggested that they could be utilized as storage areas. Wells and hitis are also valuable for supporting rescue activities with back-up activities such as water supply.

(2) Enhancing functions of Guthi(s)

Nepal has mutual aid organizations called Guthis. Although it is said that they have been weakening, they can be revitalized for the purpose of evacuation and rescue, and can be utilized for improving disaster preparedness of local communities.

Vulnerable people living in the city, such as infants, aged people and disabled. would need help from others for evacuation, information about their residences is required to be shared in advance. Moreover, wells and hitis should be better maintained for providing drinking water during emergency. Collective support functions of Guthis should be utilized for achieving these.

(3) Utilization of Cultural Heritage in the Daily Milieu for Supporting Evacuation and Rescue

The measures to be taken by residents, local communities and the municipality for utilizing cultural heritage components within the daily milieu to improve safety of the residents of the historic city are proposed as follows.

It is desirable to delegate cultural heritage components new roles to keep the historic city sustainable, and to revitalize the traditional functions of Guthis for improving disaster preparedness of local communities.

	Evacuation Preparedness	Rescue Preparedness
Municipality	Designation of primary refuges (Baha, Bahi, temple and other open spaces in each neighborhood) Maintenance of evacuation routes (securing the safety of the routes to the refuges by reinforcing access paths, courtyards and buildings)	Establishment of storage areas for rescue and salvage (at Patis and Sattels) Deployment of rescue instruments
	Support for preparation of an evacuation and rescue plan	
Community based organizations	Preparation of an evacuation and rescue plan、Practice of evacuation and rescue drills	
	Enhancement of the collective support functions of Guthis	
	Checking the residences of vulnerable people such as infants, elderly people and disabled.	Maintenance of the back up facilities for emergency supply of water(wells/hitis)
Residents	Participation in evacuation drills Preparation of survival kits (wears, foods, household medicines, radios, batteries and etc.)	Participation in rescue and salvage drills Preparation of rescue kits (work gloves, ointments, cloths and etc.)

5-2.5 Preparation of evacuation plan "from Paties and Sattels to Squares" (3rd step)

(1) Recent condition of Evacuation Sites

As the city levelevacuation sites designated by the municipality of Patan are at considerable distance from the target area, ,other large and safe areas are required for effective evacuation, especially for the elderly people and/or visitors who do not know the location of city levelevacuation sites. (Fig.5)

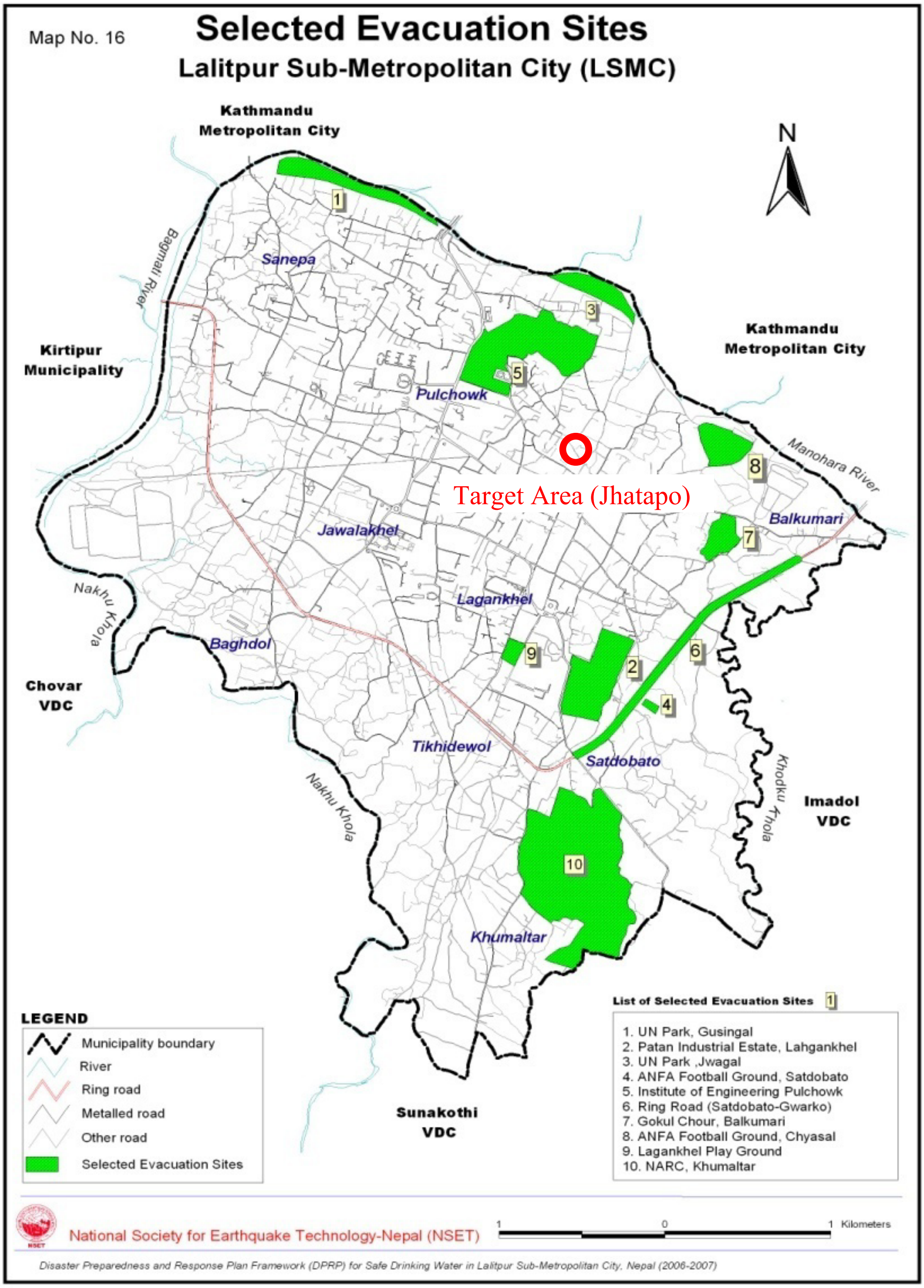


Fig. 5 Map of Existing Selected Evacuation Sites

(2) Strategy for Preparation of alternative evacuation site

a) Recomended complementary evacuation site: the Durbar Square

In the case of Hanshin Awaji Great Earthquake, most people took shelter in parks which were located at less than 500 meters distance from their houses. But in the case of historical city of Patan, there are not enough designated evacuation spaces for local residents.

Moreover, many tourists visit Patan, and most of them are unfamiliar with the local city layout and therefore do not know the location of designated evacuation sites. On the other side, most of them know the location of the Durbar Square; a famous heritage site. The Square is easy to reach even for tourists as well as for local residents.

The Durbar Square has both large open space and water resources as Hiti for immediate life support during emergency. Therefore it has high potential as a complementary evacuation space. (Fig.6)

For this reason, safety of the roads leading to the Square also needs to be ensured.

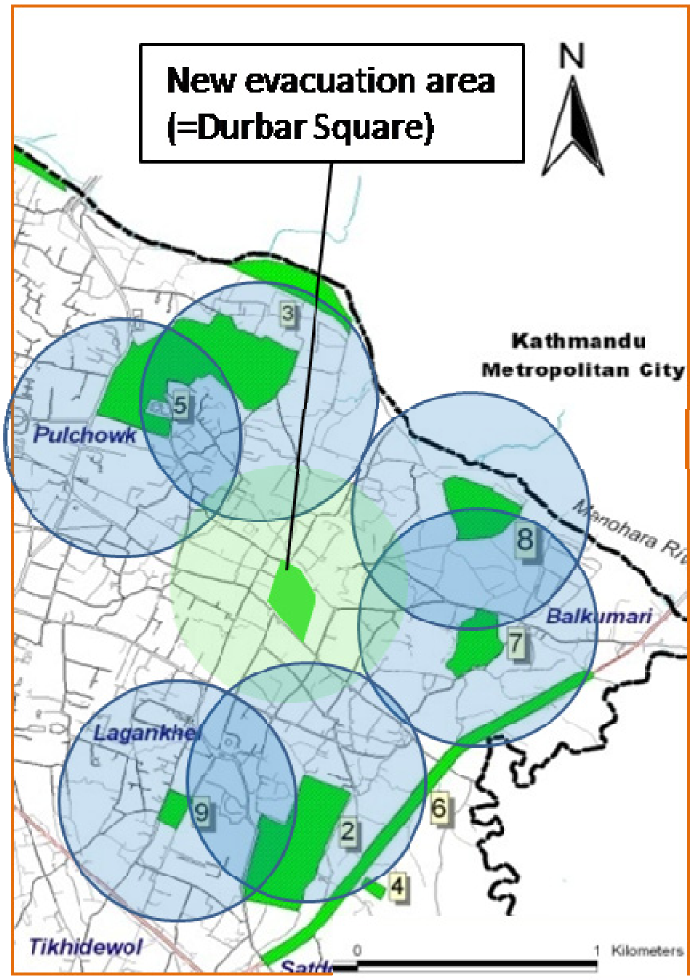


Fig. 6. Location of DS as the complementary evacuation site

b) Selection and repairment of evacuation routes to the Durbar Square

The risk of road blockage depends on the width of road. Therefore, recommended evacuation routes are to be chosen based on their widths.

According to the records of the Hanshin Awaji Great Earthquake, most of the roads which were narrower than 4 meters were completely blocked. The structure of buildings in Nepal is different from Japanese one, but width of almost all the roads around Durbar Square is narrower than 4 meters.

For keeping the relative safety, the widest road might be chosen as the main evacuation routes. And reinforcement of buildings which face to the road are required not only for keeping safety of evacuation route but also for keeping the value of traditional street scape, as much as possible.

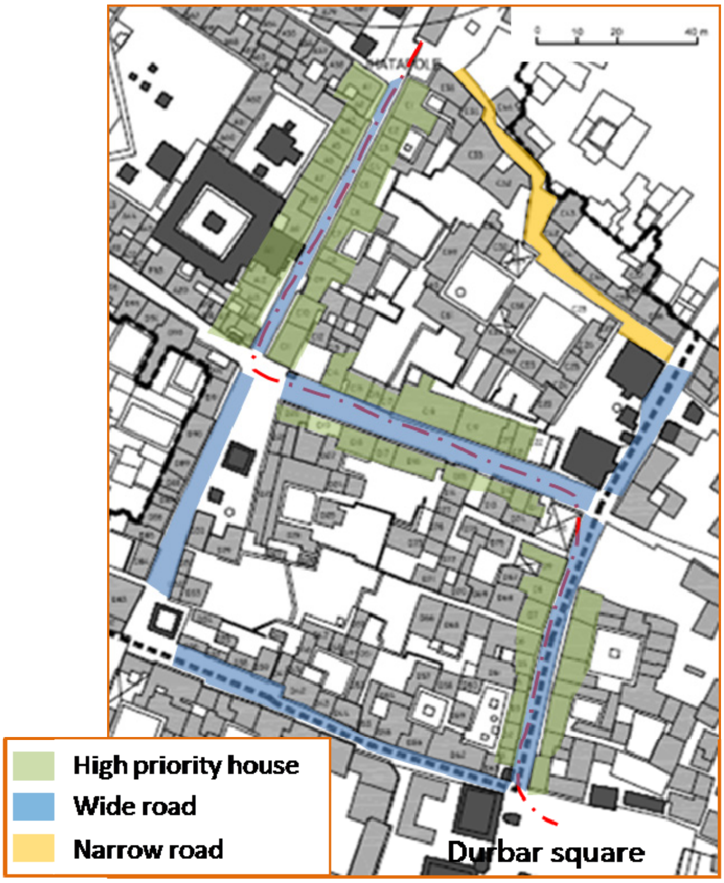


Fig. 7. Recomended evacuation routes from Paties and Sattels to the Durbar Square

c) Repair and reinforcement of the monuments in the Durbar Square

The Durbar Square is as large as 2.8 hectares. However, if all the residents within a 500 meters radius from the Durbar Square are to take refuge, there would be shortage of space for approximately 10,000 people. Refuge space for at least another 200 people would be lost if these monuments collapse.

Maintenance of the monuments in the Durbar Square is required not only for preservation of their heritage values but also for keeping safety of emergency space in case of disaster. And if the back=yards are opened as parks, the open space will drastically increase and most of people will be able to stay not only for sightseeing but also for evacuation.

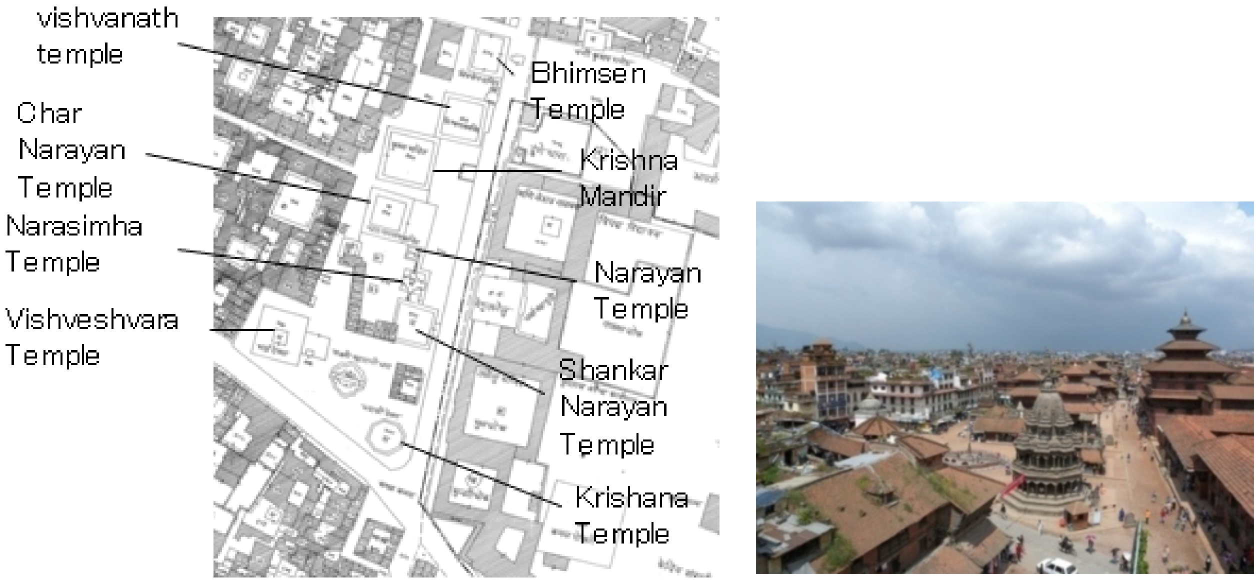


Fig. 8. The historic monuments in Durbar Square

5-3. Improvement of Firefighting Capacity by Utilizing Cultural Properties

Takeyuki Okubo
Rohit Jigyasu
Naoko Itaya
Haruki Nagashima

5-3.1 Summary

Although fires would very likely break out in the event of an earthquake, firefighting activities by public agencies would be difficult in the case of Patan. Lalitpur Fire Station, which is in charge of fire disaster management in the world heritage core zone of Patan, only owns one old fire engine. Several public fire hydrants are present, but they were installed more than 80 years ago during the Rana period, and therefore are not reliable now. The public firefighting capacity is grossly insufficient in current condition.

Thus, several measures to improve the firefighting capacity of local residents, communities and the municipality proposed.

This section focuses on the world heritage core and buffer zones as target of investigation and proposal (fig.1).

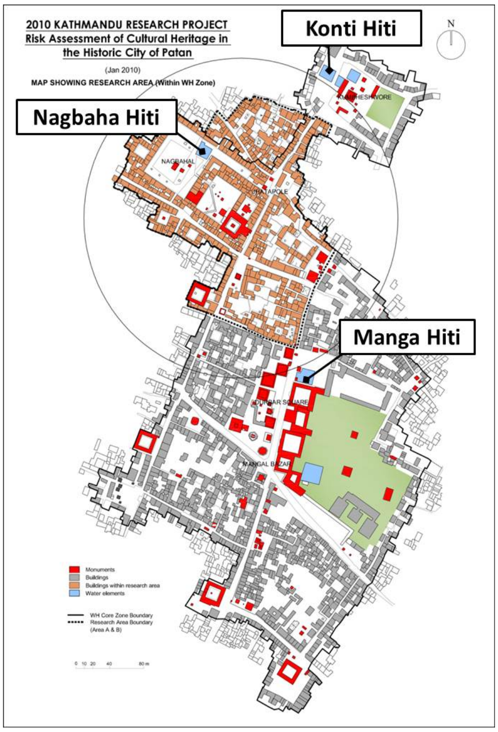


Fig.1 The world heritage core zone of Patan and Hitis

Pic.1 The burnt temple in Kumbeshwar Complex, 2009

5-3.2 Challenges for Firefighting in World Heritage core and buffer area of Patan

(1) Current condition of the public firefighting capacity

Lalitpur Fire Station is located is located 1.0 km to the west-northwest from the Patan Durbar Square, and approximately 15 minutes away from the target area by foot. The station has 11 officers and has only one fire engine. In average, 44 fires occured from July 2009 to June 2010 within the Fire Station's jurisdiction. Even when the fire engine tries to reach the scene of a fire, it cannot go through crowded narrow streets. Because of such condition, a fire burnt down an important temple (Pic.1) in Kumbeshwar Complex in 2009.

Moreover, water for firefighting is only stored in the tank of the fire engine. The tank's capacity is 2,400 litres, which would run out after only 10 minutes of firefighting. The public firefighting capacity is insufficient that sustainable firefighting activities would be even difficult in case of normal fire situations.

(2) Assessing the effectiveness of Hiti: traditional water network in Kathmandu Valley

Traditional water resources called Hiti(s) are closely connected to the lifestyle in Kathmandu Valley, and have been maintained by local communities since generations. The amount of water discharge was once abandunt, but has been gradually decreasing because of urbanization-induced exhaustion of underground water and plugging of the conduit pipes due to insufficient maintenance.

There are 3 Hitis in the target area: Kenti Hiti, Nagabaha Hiti and Manga Hiti. By utilizing them as backup for water resources, sustainable firefighting activities would be possible.

The water releasing capacity of the fire engine in Lalitpur Fire Station is 4 litters per second. Therefore, hitis that discharge more than 4 litters per second can be considered as effective water resources for firefighting.

According to the result of simple measurement, only Konti Hiti could be utilized as a water resource for fire engine (Chart.1).

Chart.1 Amounts of discharge of Hitis

Name of Hiti	Amount of water flow (lit/sec.)
(A) Konti Hiti	4.13
(B) Nagbaha Hiti	2.39
(C) Mangah Hiti	3.09

Pic.2 Fire engine of Lalitpur fire brigade office

(3) Narrow streets that make firefighting difficult

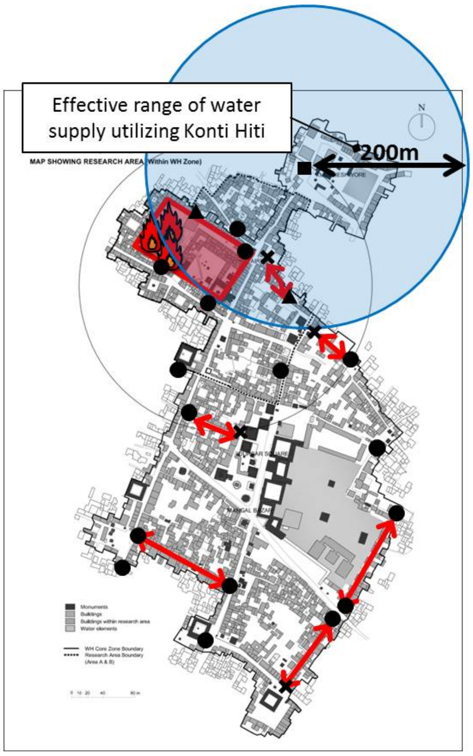
The world heritage core and buffer zones of Patan has dense built fabric of traditional houses, and narrow streets through which a fire engine cannot go through.

The fire engine that Lalitpur Fire Station owns (2.25m wide and 2.85m tall) cannot pass through either streets narrower than 2.5 meters or underpasses lower than 3 meters (Pic.2). Thus, the heritage zones located between such barriers cannot be reached by the fire engine. Moreover, many obstacles on the streets are regularly disturbing the traffic. It is expected that the streets would be blocked and difficult areas for firefighting would expand even more.

(4) Identification of difficult areas for firefighting

Firefighting difficult areas are identified from two perspectives: lack of water resources and inaccessibility of the fire engine.

Each of the hoses equipped in the fire engine is 35 meters long. The fire engine is equipped with 8 hoses, and the maximum hose length is 280 meters with all of



↔ : Challenged roads for fire engine
● : Spots within 2.5m width
▲ : Spots within 3m height
× : Obstructions to progress
■ : Disposition of fire engine

Fig.2 Difficult areas for firefighting

them connected. Where the streets meander, the maximum reach from a firefighting water resource such as Hiti is 200 meters, approximately 1/√2 of 280 meters. Therefore, it can be assumed that water could be provided anywhere within a 200 meters radius from a water resource. In other words, water supply would be insufficient in the areas further than 200 meters from Konti Hiti.

In the figure, the areas where the fire engine is hard to reach and where water supply would be insufficient are indicated (Fig.2).

It can be observed that difficult areas for firefighting include neighboring districts of Jhatapur Area with the Golden Temple, and are located around the designated world heritage.

5-3.3 Challenges for Eliminating Difficult Areas for Firefighting

The challenges are to secure water supply for firefighting and to secure accessibility for the fire engine.

(1) Security of water supply for firefighting

Currently, an effective amount of water discharge is confirmed only in Konti Hiti. However, the other Hitis also used to have sufficient water discharges, and it is desired to restore them. In addition, a water resource becomes effective only when channels for intake are secured. Recharge areas/pits for water intake need to be installed(Fig.3).

(2) Securing accessibility for the fire engine

It is highly expected that the identified difficult areas for firefighting would be isolated with all the access routes blocked by collapsed buildings after earthquake. Thus, the buildings along the streets within the areas need to be reinforced as far as possible.

On the other hand, introduction of a compact fire engine in the fire station is recommended (Pic.4). This would solve the problem of inaccessibility through narrow routes. These narrow routes should be restored even after the introduction of a small fire engine (narrower than 1.5 meters, lower than 2 meters) (Fig.4).

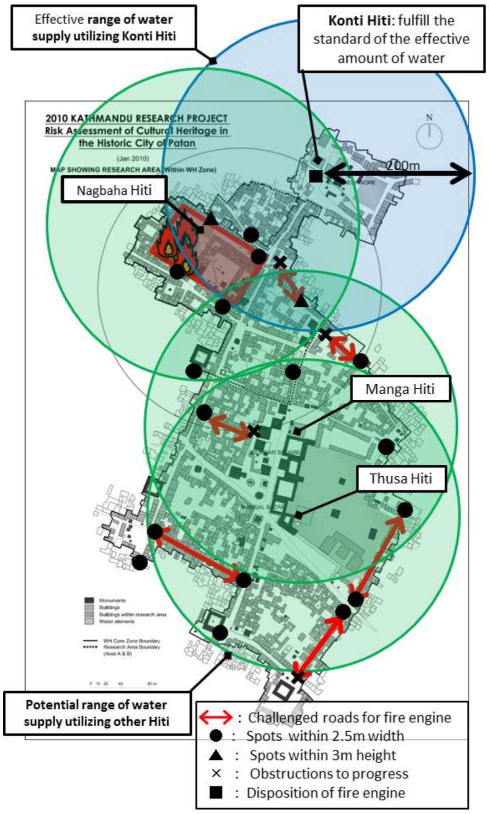


Fig.3 Effective or potential range of Water supply with Hiti

Pic.4 Compact Fire Engine

Pic.5 Repair of the Curbs of the Buildings

In order to expand the widths of such places without losing the value of the traditional townscape, repair of the curbs of the buildings (Pic.5) and transfers of monuments owned by individuals (Pic.6) are recommended.

Pic.6 Transfers of Monuments owned by Individuals

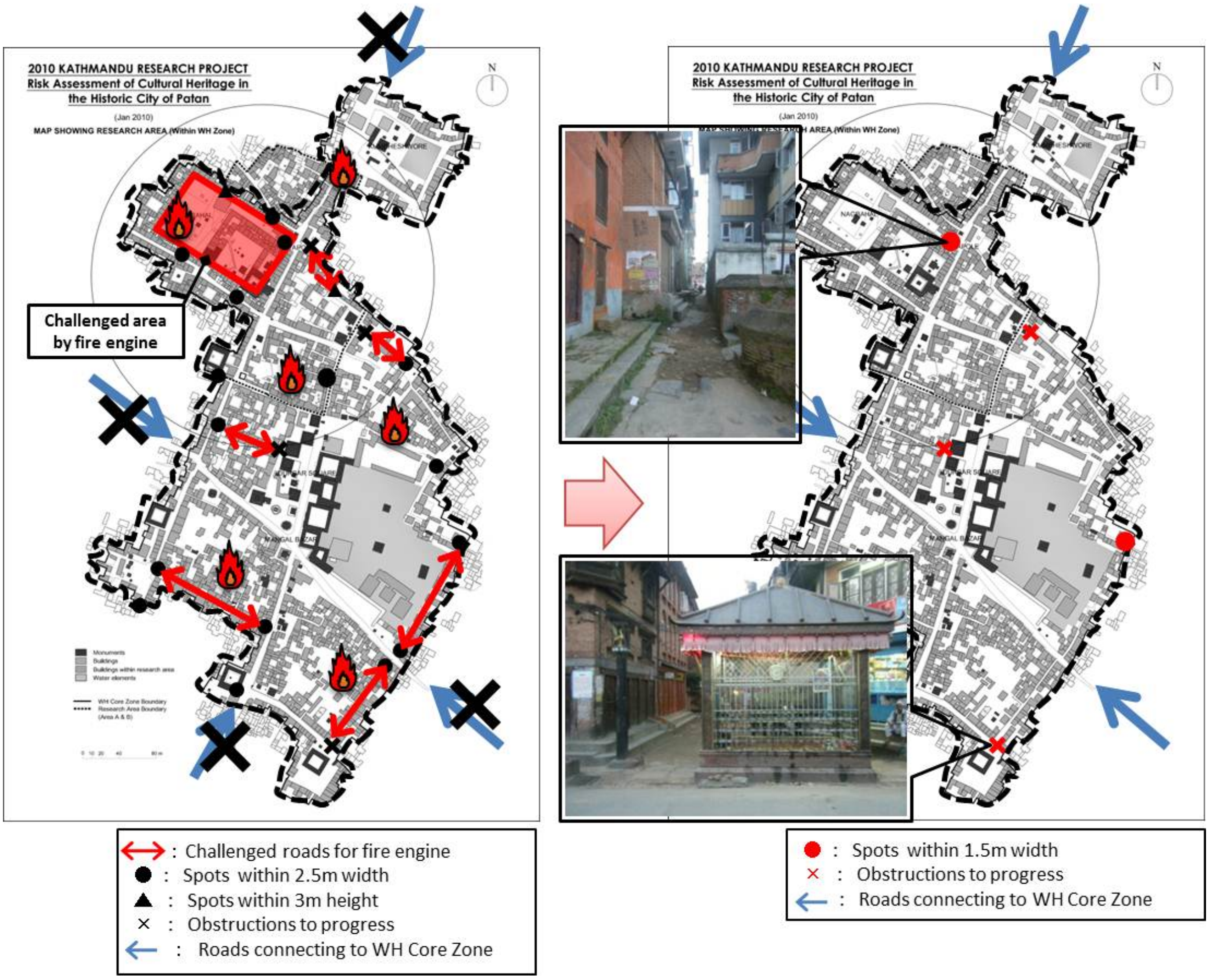


Fig.4 Difficult areas for firefighting (Left), After the introduction of a Compact Fire Engine (Right)

Right shows the place that prefer to be reinforced

5-3.4 Improvement of Disaster Risk Mitigation with Residents, Communities and the Municipality

Local residents, local communities and the municipality are recommended take the following measures to eliminate the firefighting difficult areas.

The key is to delegate to Hitis, the traditional water network, new roles to sustain the historic city, and to maintain the traditional urban structure.

	Security of water supply for firefighting (Recovery of Hitis)	Security of street widths (Not destroying the traditional urban structures)
Municipality	Installment of water supply channels. Installment of water intake channels.	Introduction of a small fire engine. Reinforcement for preventing the buildings from falling down.
Community based organizations	Management of water usage. Preparation of environment to utilize the Hitis.	Shifting of obstructive vehicles such as motorcycles. Formulation of parking spaces and rules.
Residents	Daily maintenance such as cleaning.	Maintenance of the facades of the residences including repair of the curbs.

Acknowledgment

Seismic Hazard Assessment and Structural Analysis of Traditional Brick Masonry Buildings Part;

We would like to express our sincere gratitude to Dr. Bhakta Bahadur Ale, Professor, Director, Center of Applied Research and Development, (CARD), Institute of Engineering (IOE), Tribhuvan University (TU) for valuable cooperation in the research work.

We would like to express our gratitude to Dr. Tomoki Shiotani, Associate Professor, Department of Urban Management, Kyoto University, Dr. Hisashi Umemura, Associate Professor, Department of Architecture, Nagoya Institute of Technology for their valuable contribution in Non-destructive testings of heritage structures of Patan

We also deeply appreciate the valuable assistance of Mr. Sujan Tripathi, Mr. Jhapat Bahadur Thapa, Mr. Govinda Raj Bhatta and Mr. Rakesh Dumaru, former IOE graduate students in in the tests at the site and in the laboratory, in survey of buildings and in analysis of the buildings.

Analysis of Heritage Values, Vulnerabilities and Resources for Emergency Response Part;

We would like to express our special gratitude to Mr. Kai Weise; the Architect of Planners' Alliance for the Himalayan & Allied Regions (PAHAR Nepal) and UNESCO Consultant for his immense help in implementing the field work especially with regards to documentation of the case study and interviews with key stakeholders. He was ably assisted by Mr. Manindra Shrestha and students from Khwopa Engineering College. The team also helped us in organizing workshop in the target area in 2010.

We also express our thanks to Dr. Rohit Ranjitkar, the Director of Kathmandu Valley Preservation Trust (KVPT). He supported us by introducing us to the area and the house owners. He enlightened us with his rich knowledge of the area.

Last but not the least, we wish to thank the residents of Jhatapo area. We went to their area several times for conducting our research and every time they greeted us with warm smile. It is only because of their kind cooperation that we could successfully accomplish our research. We sincerely hope they can continue preserving their rich heritage as part of their life.

Authors List

Seismic Hazard Assessment/ Structural Analysis of Traditional Brick Masonry Buildings:

Prem Nath Maskey
Professor,
Department of Civil Engineering, Institute of Engineering,
Tribhuvan University, Nepal
1-1, 1-2, 2, 3-1, 3-2.2, 3-3.2, 3-3.4, 3-3.5, 5-1

Hari Ram Parajuli
Post Doctoral Fellow,
Research Center for Disaster Mitigation of Urban Cultural Heritage,
Ristumeikan University, Japan
1-1, 2, 3-2.1, 3-2.3, 3-3.1, 5-1

Junji Kiyono
Professor,
Department of Urban Management,
Kyoto University, Japan
1-1, 2, 3-2.4, 3-3.6, 5-1

Simulation of Building Collapse/ Application of Risk Evaluation methods:

Aiko Furukawa
Associate Professor,
Department of Urban Management,
Kyoto University, Japan
3-2.4, 3-3.6

Kenzo Toki
Director, Professor,
Research Center for Disaster Mitigation of Urban Cultural Heritage,
Ritsumeikan University, Japan
3-2.4, 3-3.6

Hitoshi Taniguchi
Professor,
Research Center for Disaster Mitigation of Urban Cultural Heritage,
Ritsumeikan University, Japan
3-2.4, 3-3.6

Masatoshi Tatsumi
Master Course Student,
Department of Urban Management,
Kyoto University, Japan
3-2.4, 3-3.6

Heritage Values of the Historic City/Assessment of Heritage Values:

	**Sudarshan Raj Tiwari** Professor, Department of Architecture, Institute of Engineering Tribhuvan University, Nepal 4-1, 4-2
	**Rohit Jigyasu** Professor, President of ICORP, Research Center for Disaster Mitigation of Urban Cultural Heritage, Ritsumeikan University, Japan 4-2, 4-4, 5-2, 5-3
	**Naoko Itaya** Associate Professor, Research Center for Disaster Mitigation of Urban Cultural Heritage, Ritsumeikan University, Japan 4-2, 4-4, 5-2, 5-3

Assessment of Resources for Disaster Mitigation and Response:

	**Takeyuki Okubo** Professor, Department of Civil Engineering, College of Science and Engineering, Ritsumeikan University, Japan 4-3, 5-2, 5-3
	**Masahiro Yoshida** Master Course Student, Department of Civil Engineering, College of Science and Engineering, Ritsumeikan University, Japan 4-3.1, 4-3.3, 5-2
	**Haruki Nagashima** Master Course Student, Department of Civil Engineering, College of Science and Engineering, Ritsumeikan University, Japan 4-3.2, 4-3.3, 5-3
	**Hiroki Akehara** Student, Department of Civil Engineering, College of Science and Engineering, Ritsumeikan University, Japan 4-3.3