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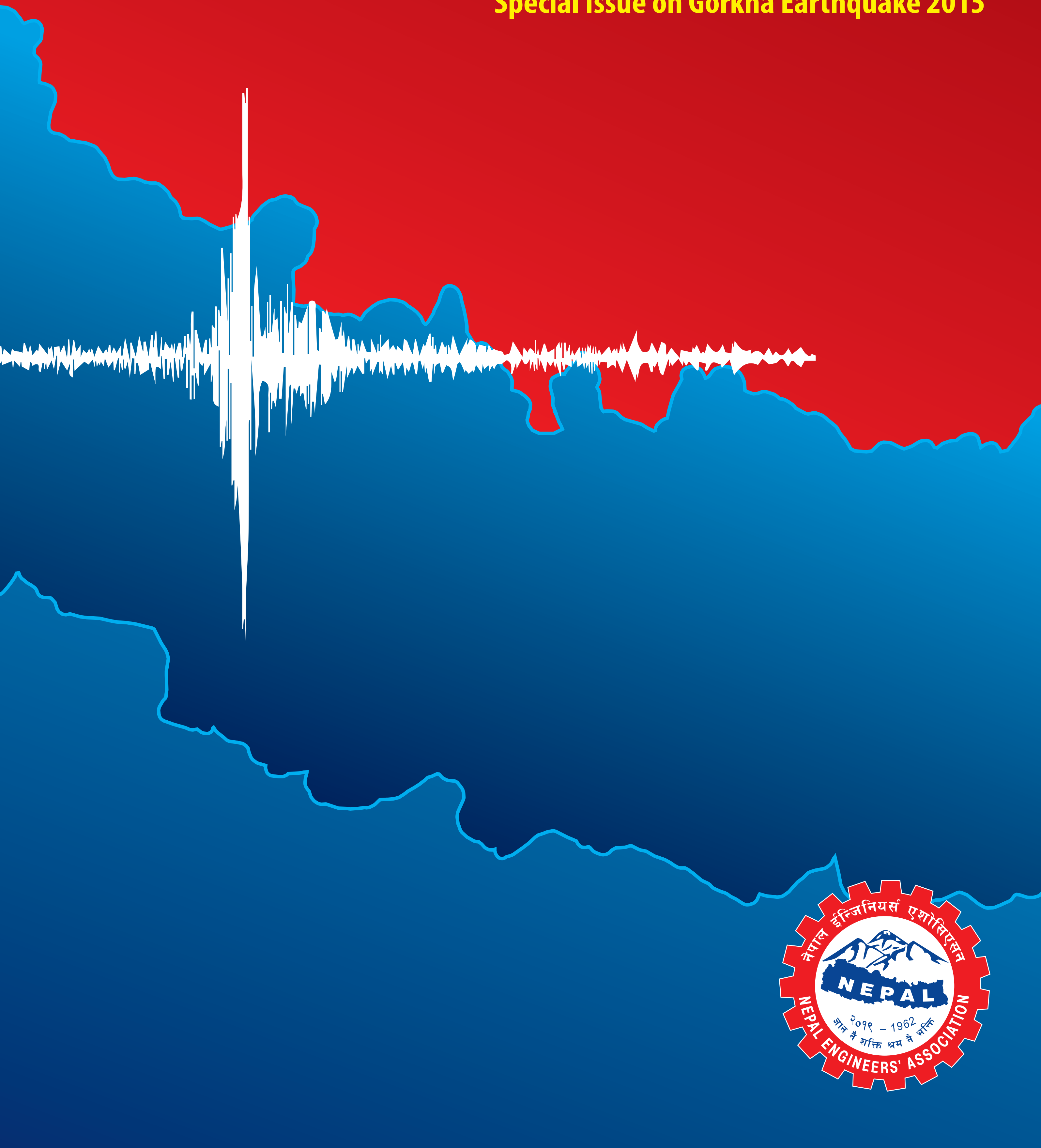

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Pulchowk, Lalitpur
G.P.O. Box: 604, Kathmandu, Nepal
Tel: 5010251/5010252, Fax: 977-15010253
E-mail: nea24@mail.com.np
www.neanepal.org.np

Message From the President

It gives me immense pleasure to commemorate the 54th year of establishment of NEA with the publication of this Technical Journal dedicated to the issues and challenges facing Gorkha 2015 Earthquake.

In addition to the historical initiative of NEA in undertaking RVDA of more than 65,000 houses in the Kathmandu Valley and more than 15,000 in other affected districts, NEA has extended remarkable support to the GON towards preparing for rehabilitation and reconstruction efforts, particularly with regards to training to engineers as well as masons and supporting with model design of rural housing through design competition. This Technical Journal shall also be seen as continuity to that spirit as it compiles research paper on almost all aspects of the earthquake science and engineering.

I am confident that research scholars and professionals all around the world and in Nepal shall greatly benefit from the information intended to be disseminated by this Journal.

I congratulate the Editorial Team of NEA including the secretariat staff for their success in making this publication happen. I extend my heartfelt gratitude to the eminent scholar of this country for their painstaking efforts in reviewing and enhancing the papers.

Wish you all the very best on the occasion of 54th Engineers' Day

Er. Dhruba Raj Thapa
President

From the Desk of Chief Editor cum General Secretary

Publication of Technical Journal has been a ritual followed by almost each of the past executive councils of NEA. But the events that followed the disastrous earthquake of April 25, 2015, were of particular enquiry to the engineering community. Subsequent to historic initiatives of the Nepal Engineers' Association in leading the way to Rapid Visual Damage Assessment of affected private buildings together with providing the platform for earthquake related talks and knowledge sharing at the NEA premise from experts all around the world, necessitated technical documentation and its standardized publication. Apropos, the 30th Executive Council has decided to publish a special issue of Technical Journal with ISSN 2091-0592, dedicated to "Gorkha 2015 Earthquake". Call for papers in the prescribed format was invited in February and a panel of eminent scholars was formed in March to review the technical suitability of the papers. This journal in its present form is the outcome of painstaking efforts by the peer reviewers, authors and the staffs of NEA secretariat that followed subsequently.

Altogether 21 papers finally qualified to be included in the journal. The topics ranges from geodetic datum and seismic response estimation under seismic geology to damage analysis repair and retrofitting belonging to the field of structural engineering. A few other group of papers focuses on build back better efforts in the remote hilly region. In this respect, papers incorporating research findings related to earth mortar stone masonry and random rubble masonry along with earth bag technology are worth consideration.. Papers deliberating on impacts of earthquake to transport sector and use of ICT based tools for earthquake management is also covered together with one on emphasizing cultural continuity in the post earthquake rehabilitation. Recent experience of Pakistan with reconstruction and rehabilitation is followed by deliberation on challenges faced by Nepal for reconstruction. But the research paper by ETH Zurich entitled "Vulnerability of the Nepalese Building Stock during The 2015 Gorkha Earthquake" that is built on RVDA data base created by NEA, calls for special attention.

Immediately after the magnitude 7.8 earthquake that struck Nepal, NEA in collaboration with several professional and academic institutions offered voluntary services of engineers and architects to residential households for rapid damage assessment of buildings and thereby to advise whether the house is fit for occupation or not. Altogether, 3500 volunteers were mobilized, trained and assigned to affected area in a group of 5-7. A total of more than 65,000 houses were inspected within a month and half. This campaign was later known as "जनतासंग ईन्जिनीयर", meaning Engineers with People. The campaign had added significance as eventually it went down in the history of NEA as the sole occurrence that established the social credentials of engineers to its optimum. Subsequently, with the support from the ETH Zurich, an engineering, science, technology, mathematics and management university in the city of Zürich, Switzerland the RVDA information in the paper form were digitized and a data base of more than 37,000 inspected houses was created, and finally the data is being used for the said research paper.

It gives us immense pleasure to note that the launching of this Technical Journal on July 18, 2016 (Shrawan 3, 2073 BS) coincides with the celebration of NEA's 54th years of existence.

Finally, I express my sincere gratitude to all my colleague engineers at NEA, secretariat staffs, Times Creation and Magnum Printing Press, without whose help it would not have been possible to publish this Technical Journal in its present form.
Happy Engineers' Day!!

Kishore Kumar Jha
Editor in Chief/General Secretary, NEA
Date: July 18, 2016

Message from the publication committee

I feel honored to write a few messages in this "Earthquake Special Technical Journal" published by NEA as a chair person of publication committee as well as member of the review committee of this journal. I have feeling that this is one of the excellent jobs done by this executive committee during their tenure. I must congratulate all the executive members of the 30th Executive Council of NEA for their excellent job.

During the earthquake April-May 2015, NEA has done tremendous job to the victims of earthquake. Upon the request made by NEA, hundreds of engineers visited the earthquake victim sites and provided real time technical support and suggestion. No matter what was the situation, young energetic as well as seniors engineers provided their technical supports to the victims of earthquake. The efforts then made by NEA was highly acknowledged by the society.

This journal is the continuation of the knowledge sharing by the professionals related to earthquake field. All the articles in this journal are focusing on this issue. This journal provides very specific as well as general information in the field of earthquake. Senior international experts have shared their experiences with their papers; however places are also given for the young engineers to express their views too. The idea behind is that to encourage young engineers for writing and publishing papers. This journal will also be useful as a source for all researchers dealing with this field.

I would like to thank NEA President, General Secretary and other executive members for providing me this opportunity to serve as a chair person of the publication committee. I really appreciate the support received by the members of publication committee. I would like thank all the members of publication committee.

Durga P. Sangroula, PhD

Professor
Department of Civil Engineering, Pulchowk Campus
Institute of Engineering

A brief introduction of the reviewers

Dr. Durga Sangraula

Dr. Sangroula is currently a Professor at Pulchowk Campus, Institute of Engineering, Tribhuwan University, Nepal. He has worked in Norwegian University of Science and Technology (NTNU) University , Norway from February 2002 – February 2006 (4 years 1 month). He received Master of Science (MSc) and Dr. eng, Hydrology and Water Resources Science and Sedimentation Engineering from the same university. He also has to his credit Master of Science (MSc), in Water Resources Engineering (1983 – 1989) from the Leningrad Polytechnical Institute, former USSR. .

Dr. Sangroula's interests includes lecturing, writing books as well as delivering special lectures on the field of reservoir sedimentation. He has to his credit 3 books, 4 research papers and numerous scientific papers published to date. Dr. Sangroula has been actively involved in the affairs of NEA since more than a decade, and currently is the Chairman of the Publication Committee.

Dr. jibgar Joshi

Dr. Joshi is an eminent urban and regional planner. He got the degree of bachelor of civil engineering with honors in 1971 from Jadavpur University, Calcutta. He received his PhD in 1981 from Tribhuwan University; the title of his dissertation being regional planning with special reference to Nepal. He was a Humphrey/SPURS fellow at the Department of Urban Studies and Planning, MIT during the year 1987/88. He received P. G. diploma in housing, planning and building from IHS, Rotterdam in 1983.

Dr. Joshi offers courses to M.Sc. students at SchEMS and Urban Planning Program, IoE. He also supervises thesis works of master's students. He is a freelance consultant and is currently engaged as Team Leader for the preparation of HABITAT III - Country Report. His current research interests include governance structure and service delivery. He is a life member and former executive member of NEA. He is a life member and Immediate Past President of RUPSON. He is also a life member and Former President of Nepal Association of Humphrey Fellows. He is the author of more than eight books and many research papers. His books include: Housing and Urban Development in Nepal 2013, Managing Environment and Cities for Sustainable Development 2011, Regional Strategy for Development: A case study of Nepal 1985.

Dr. Gokarna Bahadur Motra

Dr. Motra has Ph.D. in Civil Engineering Structures (2013) from the Indian Institute of Technology Bombay, India with thesis title "Optimal Control of Seismically Excited Buildings Connected by Magnetorheological Dampers". He has M.E. in Industrial and Civil Engineering with Honours (Distinction) from the Yerevan Architectural and Constructional Engineering Institute, Republic of Armenia, USSR.

Dr. Motra is a Professor at Tribhuwan University, Institute of Engineering, Pulchowk Campus Department of Civil Engineering, Pulchowk, Nepal since 2012. He was Associate Professor between 2005 – 2012 in the same institute as above. Previously, he was lecturer (1991-2005) there. He has guided Master's Theses to around 10 master students on Structural Engineering. He teaches Structural Mechanics, Structural Analysis, Structural Dynamics, Earthquake Resistant Design of Structures, Computational Techniques, and others. He has to his credit number of publications in international journals; international conferences and national journals.

Dr. Ramesh Maskey

Dr. Maskey got an intermediate in Electrical Engineering from Institute of Engineering, Tribhuwan University, Nepal in 1979. He did MSc in Civil Engineering with specialization in Hydropower in 1980-1987 from USSR, and MSc in Resources Engineering in 1994-96 through DAAD (German Academic Exchange Service) scholarship at University of Karlsruhe, Germany. He taught as lecturer at Institute of Engineering in Nepal during 1987-1988. Then joined Nepal Electricity Authority (NEA) as civil hydraulic engineer for Arun-III hydropower project during 1988-1994. During that period, he was one of the core group members in drafting National Environmental Impact Assessment Guidelines 1992.

Dr. Maskey worked as a short-term consultant in the Water and Power Development Authority, Lahore, Pakistan for the Institute of Water Resources Management, Hydraulic and Rural Engineering (IWK) at University of Karlsruhe in 1998. He joined the same institute as Associate Scientist for teaching post-graduate students and continue my research for a period of 5 years. He received Doctoral Degree in Civil Engineering in 2004. He left the University in 2005 and worked as founding executive director of a private firm in Germany till February 2006. He joined Kathmandu University (KU) as professor of Civil Engineering in March 2006. Since 2013, he became the Associate Dean for Academic and Administrative Affairs of School of Engineering. He has almost 30 years of experience in my profession. At present, he is leading a team of experts on determining ecological flow in Gandak River Basin, a joint research project of KU and WWF since May 2016. he supervised undergraduate and graduate students' project works, produced two PhD scholars and supervising another one at present. He has authored many scientific articles and worked as peer reviewer of internationally renowned journals. Recently, the German Academic Exchange Service Center (DAAD) has honored him as Research Ambassador in September 2014. Dr. Maskey's research interest encompasses water resources engineering and management, hydropower, renewable energy technology, distributed power system, robotics, hydraulic structures and river engineering etc.

Er. Kishore K. Jha

Mr. Jha is the General Secretary of the 30th Executive Council (2014-2016) of Nepal Engineers' Association. He holds B.E.(Civil) degree from American University of Beirut and M.Sc. (Urban Planning) from Institute of Engineering, Tribhuwan University. He is a Fellow of Nepal Engineers' Association. Mr. Jha has extensive professional experience of over 31 years covering research and investigation, planning, project formulation, feasibility studies, design and implementation of particularly the urban and rural infrastructure projects. He had also been involved in conducting social research, policy analysis as well as monitoring and evaluation. Besides, being involved as working director of a consulting firm, Mr. Jha have had the opportunity to execute more than fourty consulting assignments in the capacity of Project Director/Coordinator, and thus has been adequately exposed to corporate financial and personnel management activities. These wide ranges of diverse experiences have helped Mr. Jha develop a unique blend of professional expertise in the fields of civil engineering, development planning, social analysis and research, monitoring and evaluation along with corporate and project management.

Mr. Jha has to his credit wide range of publications in the national and international journals, and has been involved in several publications of the NEA and other professional societies.

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Towards a Modernized Geodetic Datum for Nepal: Options for Developing an Accurate Terrestrial Reference Frame Following the April 25, 2015 MW7.8 Gorkha Earthquake

Chris PEARSON

(School of Surveying, University of Otago), Chris.pearson@otago.ac.nz

Niraj MANANDHA

Survey Department Nepal

ABSTRACT

Along with the damage to buildings and infrastructure, the April 25, 2015 Mw7.8 Gorkha earthquake caused quite significant deformation over a large area in central Nepal with displacements of over 2 m recorded in the vicinity of Kathmandu. Correcting for this will require a national deformation mode (NDM) that will have the capacity to correct for the earthquake displacements and ongoing tectonic deformation associated with Nepal's location on the India/ Eurasian plate boundary. The NDM discussed here contains models of the velocity field and co-seismic deformation. The velocity model for Nepal is based on a compilation of published velocity measurements used to study the boundary between the Indian plate to the south and the overriding Eurasian plate to the north. The co-seismic deformation associated with the Gorkha earthquake and its 12th May Mw7.3 aftershock was modeled using published dislocation models. By combing the velocity and co-seismic models we have developed an NDM that can correct coordinate for both the effect of the earthquakes and continuous deformation associated with Indian / Eurasian plate boundary. Preliminary tests of the model demonstrate that applying the NDM makes a significant improvement when adjusting survey data sets that were acquired both before and after the earthquakes.

Keywords: *Gorkha earthquake, deformation, national deformation mode (NDM), Indian/Eurasian plate, co-seismic deformation, adjusting survey data.*

I. INTRODUCTION

The current Nepal-Everest datum is a classical datum developed in 1984 by the Military Survey branch of the Royal (UK) Engineers in collaboration with the Nepal Survey Department. However, Nepal is located at the conjoint of two converging plates: the Indian plate to the south and the overriding Eurasian plate to the north. Due to the regular convergence of these plates the existing passive geodetic control network has become distorted with time. This combined with the effect of the April 25, 2015 Mw7.8 Gorkha earthquake, which caused significant deformation over a large area centered at Kathmandu means that the integrity of the current Nepal-Everest datum cannot be assured. In this paper we consider options for a modernized geodetic datum for Nepal that will have the capacity to correct for the earthquake displacements and ongoing tectonic deformation associated with Nepal's location on the India/Asia plate boundary.

Semi-dynamic vs conventional datums

Because of the effect of plate tectonic motions, the actual position of points on the earth change continuously. However nearly all users find it difficult to deal with continuous coordinate change. There are two quite different ways in which geodetic datums can deal with tectonic motion. Modern semi-dynamic datums are based on a version of the International Terrestrial Reference Frame. Stable coordinates are produced by projecting each coordinate to its position at a common date called the reference epoch. In order to make this technique work we need a model of how the earth is moving due to plate tectonics. In stable areas, the effect of earthquakes will be small and the motion of the points will follow the motion of the tectonic plates. In areas that are located on the boundaries of tectonic plates, the motion is more complicated because the points are deforming or moving relative to each other in complex ways.

In this case a mathematical model, usually called a National Deformation Model, is used to calculate the trajectory of points. This usually includes a way of estimating the constant or secular velocity of each point and a way of calculating the effect of any earthquakes that may have occurred between the time that the coordinates were measured (epoch of observation) and the reference epoch. The effect of earthquakes is an instantaneous offset while the effect of the velocity increases linearly with time. The total motion is just the sum of the earthquake and constant velocity terms. In practice both the velocity and earthquake shifts are stored as a series of grid files which are used to estimate the appropriate values for an arbitrary point by linear interpolation. The basic idea of a National Deformation Model is illustrated in Figure 1, which shows the trajectory of a point affected by a constant velocity and two earthquake shifts.

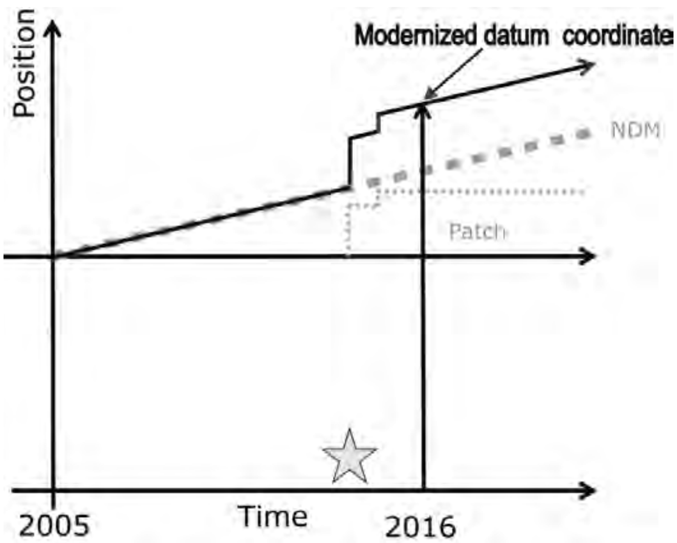

Figure 1 : Schematic diagram of a dynamic datum. Heavy dashed gray line shows the secular velocity and thin gray dotted line co-seismic contribution to the deformation model. The solid black line shows the deformation model with both contributions combined.

In contrast, older classical datums, which were usually established before the reality of plate tectonics was widely accepted, establish fixed coordinates for a network of control points with no mechanism to correct for the tectonic motion. As a result the marks will drift off their true positions. However, the relative position of points will not change significantly with time in stable areas such as the interiors of plates since the entire region is moving rigidly. For regions on plate boundaries such as Nepal, ongoing deformation means that the relative position of points will change with time due to an non-homogeneous velocity field and earthquakes. Thus the datum will become distorted as the bearings and distances between marks calculated from their coordinates become increasingly different from what we would measure on the ground.

II. SEMI-DYNAMIC DATUM FOR NEPAL

Nepal is located at the conjoint of two converging plates: the Indian plate to the south and the overriding Eurasian plate to the north. A significant amount of the convergent component of plate motion is accommodated within Nepal resulting in the crustal velocities changing from a northeast trend in Northern India to an east-northeast trend in southern Tibet. Due to the regular convergence of these plates Nepal has been subjected to a series of great earthquakes including the 25th April 2015 Mw7.8 Gorkha earthquake.

The Deformation model is the tool that allows coordinates to be projected either backward or forward in time to the reference epoch. Typically, a deformation model contains two distinctly different elements. The first is a model of the variation of the long term (or secular) crustal velocity across the country and the second is a model or models of the co-seismic deformation associated with any large earthquakes that have occurred since the datum was introduced. Both the velocity model and the co-seismic deformation models are grid files so that the estimates of the velocity or co-seismic shifts can be determined by linear interpolation (Stanaway et al. 2012).

Our model of the velocity field for Nepal was developed by combining published velocities for Nepal and adjacent parts of China and India from four geodetic studies in the Nepal region (see Pearson et al 2016 for a detailed discussion). The velocities are shown in Figure 2.

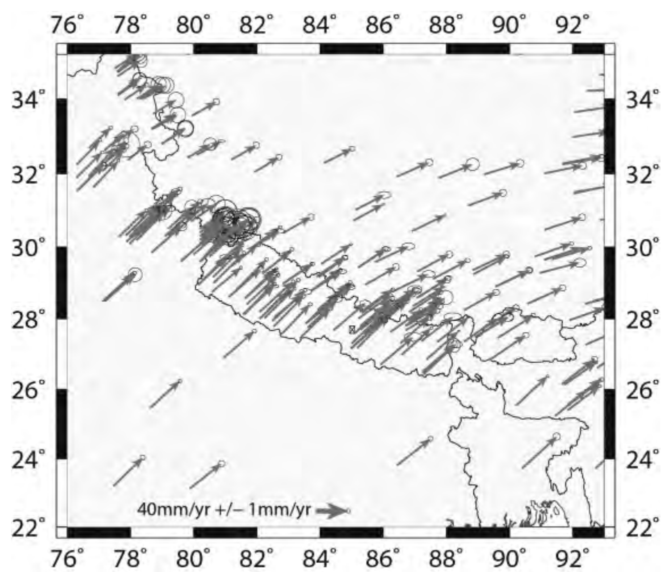

Figure 2 : *Compilation of velocity measurements for Nepal and surrounding parts of India and China. (Pearson et al 2016).*

Using these velocities we developed a grid file that covers the region from 80°E to 89°E and 26°N to 31°N (Figure 3). While Figure 3 shows velocity vectors on a half degree spacing the actual gridded velocities have a spacing of 20 points/degree (0.05°).

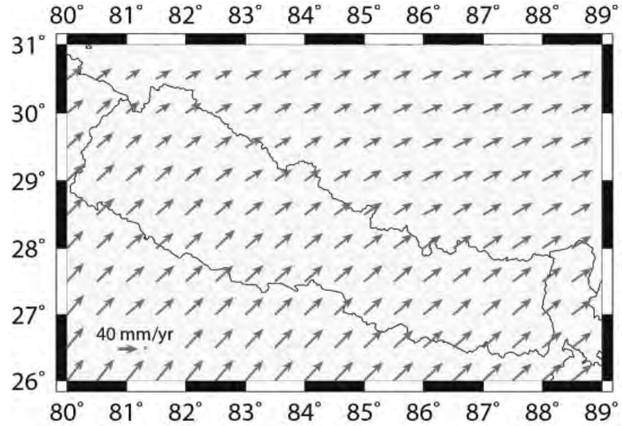

Figure 3 : *Velocity grid for Nepal. (Pearson et al 2016).*

The NDM must also include patches or gird files that can be used to predict the earthquake deformation at any point. We are currently working on developing patches for the co-seismic part of the 25th April 2015 Gorkha Earthquake and the 12th May Mw7.3 aftershock using published dislocation models (Galetzka et al., 2015). Figure 4 shows the co-seismic slip from the 25th April 2015 Gorkha Earthquake. Note that the Kathmandu Valley has moved by up to 1.9 m.

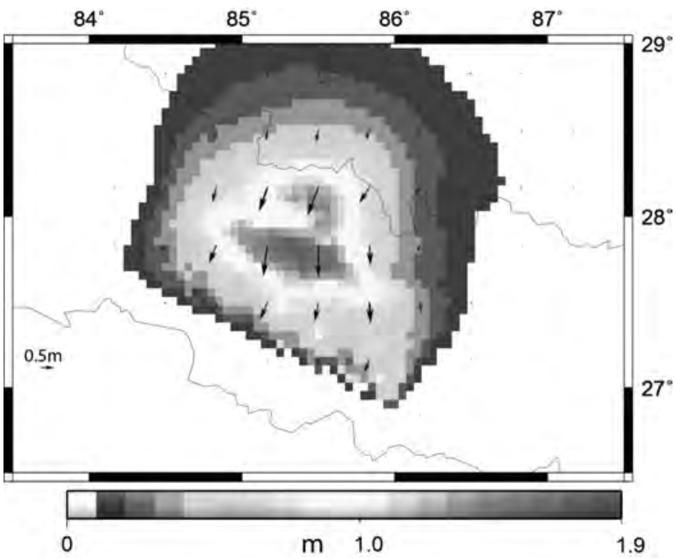

Figure 4 : *Predicted displacement associated with the 25th April 2015 Mw7.8 Gorkha Earthquake*

The deformation model would form a key part of a semi dynamic datum for Nepal which could be based on ITRF2014 with a reference epoch set some time after the end of the current sequence of earthquakes (Pearson et al 2016).

In order to test the effectiveness of a semi-dynamic datum to correct for the deformation from the April 25, 2015 Mw7.8 Gorkha earthquake, we adjusted the GPS data that contained both pre and post-earthquake measurements. These test points define a polygon extending about 40 km in the NW SE direction centered at Kathmandu. Between these points there are nine GPS baselines, three of which were recorded in April 2013, before the earthquake and six of which were observed on 08 May 2015, in the period between the 25th April 2015 Gorkha Earthquake and the 12th May Mw7.3 aftershock. The first adjustment was conducted without using a deformation model while, in the second adjustment, the deformation model was used to correct all the measurements to pre-earthquake values. The Standard error of unit weight for the adjustment which does not apply the NDM is more than 3 times greater than the SEUW for the model which does apply the NDM. This difference demonstrates that the deformation model is effective in correcting for crustal deformation between the two surveys. The improvement in the SEUW due to applying the NDM is significant at the 99.99% level of confidence.

III. CONCLUSIONS

Because of the effect of the 25th April, 2015 Gorkha earthquake, significant earth deformation has occurred in a large area of Nepal centered on Kathmandu Valley. As a result, the geodetic control in this region is significantly distorted with published geodetic control coordinates being displaced from their true position on the ground by up to 2m. Correcting these distortions will require a new geodetic datum. In this paper we consider the possibility of Nepal adopting a semi-dynamic datum, which would be based on ITRF2014 and include a national deformation model capable of correcting for the recent earthquakes and normal tectonic motion. We demonstrate that it is possible to develop a deformation model for Nepal incorporating the Gorkha earthquake and the variation of the long term (or secular) crustal velocity across the country using published information. While this model is preliminary our test shows that its use does a good job of correcting survey measurements for the effect of the earthquake.

ACKNOWLEDGEMENTS

We would li9ke to thank Diego Melgar for allowing us access to his models of the April 25 Gorkha earthquake and the 12th May Mw7.3 aftershock in advance of publication. Much of the work described in this paper was conducted during a 5 week visit to the Survey Department of Nepal by Chris Pearson, which was funded by the New Zealand Ministry of Foreign Affairs and Trade. We also thank the University of Otago for waiving normal overheads for this project.

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Seismic Response Estimation Along NW-SE Section of Kathmandu Valley Alluvium

Dipendra GAUTAM
Structural and Earthquake Engineering Research Institute, Kathmandu, Nepal
Corresponding email: *strdyn@yahoo.com*

Deepak CHAMLAGAIN
Department of Geology, Tri-Chandra Multiple Campus, Tribhuvan University, Nepal

ABSTRACT

Kathmandu valley is characterized by frequent seismic events and seismic hazard is concentrated particularly on alluvial deposits. The local geology, geotechnical site conditions, valley undulations and structural deficiencies have inflicted structural damages during past earthquakes and also similar scenario was observed during 2015 Gorkha earthquake. In order to delineate the seismic performance and ground response of Kathmandu valley, a section running NW-SE has been chosen with six borehole logs running upto the bedrock. One dimensional ground response analysis has been performed using the equivalent-linear site response analysis (EERA) algorithm and results have been obtained in terms of maximum amplification, peak ground acceleration and peak spectral acceleration. While performing one dimensional ground response analysis of the NW-SE section covering the soft soil deposits, it is observed that the preliminary valley geometry effect could have been contributed in the severe damage scenario during 1833, 1934 and 2015 earthquakes underpinned by the devastations during each of the event. Higher spectral amplification is obtained in valley periphery (Harisiddi and Balaju) and lower amplifications are particularly concentrated at central portion of valley. Trend of peak spectral acceleration and peak ground acceleration have shown that the higher values are concentrated at central valley and lower accelerations are in particular, concentrated at valley periphery (Harisiddi). Through preliminary interpretation of two dimensional seismic response in a defined section, preliminary valley effects are disseminated in this paper.

Keywords: *Ground response analysis; soil amplification; valley effect; Kathmandu valley.*

I. INTRODUCTION

After the localized damages during many historical earthquakes like; Michoacan Mexico City (1985), Loma Prieta (1989), Northridge (1994), Kobe (1995), Chi-Chi (1999) local geological and geotechnical considerations that inflict the local amplification and de-amplification of seismic waves, are more pronounced worldwide. Like in the case of Mexico City earthquake (M_W 8.1), damage concentration was particularly at a distance 350 km from the epicenter. Also during 1934 Bihar-Nepal earthquake (M_W~8.1), damage was particularly concentrated in Kathmandu valley ~250 km far from the epicenter due to the soil characteristics and associated mechanisms. The unconsolidated lacustrine deposit was the worst hit by the amplification of seismic waves hence damage occurrence surpassed the rate even as that of the epicentral areas (Hashash et al. 2010). Seismic site effect is identified as a vital consideration in geotechnical earthquake engineering and structural engineering ever since the dominance of local amplification was observed during many historical events (e.g. Bard and Bouchan 1985; Duval et al. 1998; Somerville 1998; Bielak 1999; Pitilakis et al. 1999). There is widespread consent with regard to local amplification of seismic waves by subsurface strata and sometimes behavioral anomalies are frequently observed across the world during earthquakes.

The ground parameters in any site of interest are the function of source activation, propagation path of seismic energy and effect of local geology on the wave-field at the recording site as shown in Fig. 1.

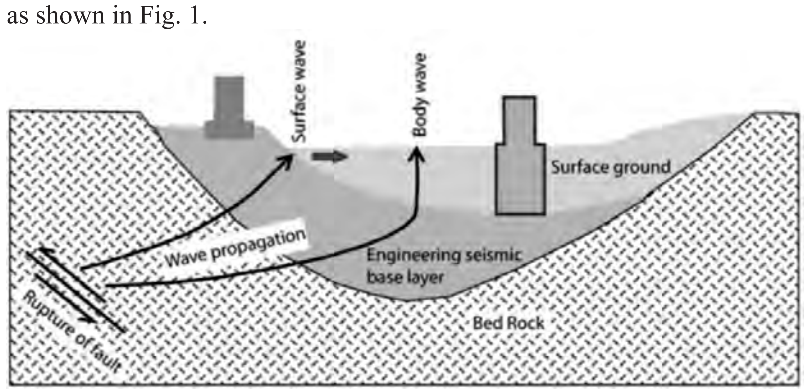

Fig. 1 : *Schematic illustration of the wave propagation from fault to ground surface*

Seismic wave propagation in time domain in terms of physical amplitudes associated with acceleration, velocity and displacement can be represented by expression 1.

$$r(t) = e(t) * p(t) * s(t) \qquad (1)$$

Wherein, e(t) = source signal, p(t) = function characterizing the propagation from the source to the site and s(t) = local site effect.

The frequency domain representation of expression (1) is,

$$R(f) = E(f).P(f).S(f) \qquad (2)$$

Wherein, R(f), E(f), P(f), and S(f) are the respective Fourier transforms of r(t), e(t), p(t), and s(t). The s(t) or S(f) is the mostly governing factor with regard to subsoil contribution leading to discrete and localized damage pattern during earthquakes.

Previous studies have highlighted the ground motion parameters in Kathmandu valley be relatively high in case of one dimensional analysis (e.g. Maskey and Dutta 2004; Paudyal et al. 2013; Chamlagain and Gautam 2015; Gautam and Chamlagain 2015; Chaulagain et al. 2015; Gautam and Chamlagain 2016; Gautam et al. 2016a). Moreover, the devastating earthquakes of 1934 and 2015 have reflected localized damages in some locations of Kathmandu valley (Rana 1935; Pandey and Molnar 1988; Gautam et al. 2016a; Gautam et al. 2016b; Gautam and Chaulagain 2016); this intensifies the necessitates dissemination of local site effects during earthquakes in Kathmandu valley. In this regard, it is imperative to study the exacting behavior of Kathmandu valley soft soil in terms of ground motion parameters and spectral amplification. Being a deep alluvial valley with several geotechnical discrepancies within very small spatial variation, Kathmandu valley is possessing anomalous behavior during earthquakes, which was evinced during the 1833,1934 and 2015 events clearly. Geologic structures like; basins and sediment filled valleys and topographical features have pivotal importance on variation of the earthquake ground motions. From several observations it has been concluded that the ground motion in the frequency range of 1 to 12 Hz is of most engineering interest, apart from this, there's widespread recognition with regard to the effects of local topography and complex surface geology (Su and Aki 1995). As per the damage distribution like in the case of 1934 Bihar-Nepal earthquake, clear demarcation in terms of deep basin as well as basin edge effects is seamlessly visualized in Harisiddi, Lubhu, Sankhu among others at the edge of Kathmandu basin. However, there are seldom no works [never found!?] performed regarding two dimensional ground response analysis of Kathmandu valley accounting basin geometry, valley effects, topographic effects, ridge effects and basin edge effects. In order to carry out the preliminary deep basin and basin edge effects in a specified NW-SE valley section, this study outlines the ground motion and spectral amplification parameters so as to depict the possible two dimensional response in Kathmandu valley. In addition, this study attempts to justify the possible explanation behind uneven damage pattern of 1934 and 2015 earthquakes in specific areas within Kathmandu valley.

II. KATHMANDU VALLEY

Kathmandu valley is an intermountain basin in the 'Lower Himalaya' located in the vicinity of the central seismic gap, which could generate several strong earthquakes [M_W>7/8?]. The urbanization trend shows further exacerbation of risk clustered to this area along with the underlying seismic hazard. Nepal has annual average population growth rate of 1.35% (CBS 2012) and there is huge disparity between the urban and rural population growth rate. After declaration of additional 133 municipalities in 2014, around 37% of people reside in the urban areas of Nepal. The Kathmandu valley is the most densely populated area with concentration of major administrative centers. Kathmandu valley has 613,606 houses accommodating 2.5 million people within three districts Kathmandu, Lalitpur and Bhaktapur having total area of 899 km^2. The population densities in these three districts are respectively 4416, 1216 and 2560 per square kilometers (CBS 2012). Most of the houses in Kathmandu valley are non-engineered constructions-that existed before the enforcement of building codes and seldom strengthened even after Sikkim-Nepal and Gorkha earthquakes of 2011 and 2015 respectively. The rapid land use change, conversion of agricultural land into settlement areas, is mushrooming in Kathmandu valley, however such planning is non-engineered and previous rice fields are converted into newer settlements. This paradigm could be instrumental in terms of future urban expansion and rapid urbanization undergirded by the wave amplification during earthquakes. The situation of present urbanization and non-engineered or poorly engineered and unmonitored construction practices would inevitably cause trauma in Kathmandu. As per 2011 census, the population change in Kathmandu over this decade is around 61.23% (CBS 2012). This inflicts additional pressure on expansion of settlements and haphazard land planning. Young geological formation in the other hand has been triggering the unrestrained damage statistics, the aggregated risk scenario mirrors the urgent need of noble frameworks of risk reduction for Kathmandu valley.

III. METHODOLOGY

Ground response analysis along the NW-SE section of Kathmandu valley is performed on the basis of Equivalent Linear Earthquake Site Response Analyses (EERA) algorithm (Bardet et al. 2000). In the NW-SE section six boreholes reaching upto the bedrock is chosen (Fig. 2) and analyzed in iterative EERA platform. Equivalent linearization of soil properties is accounted in this analysis with the material damping curves developed from various parts of the world separately for sand, silt and clay. The Vucetic-Dobry material properties (Vucetic and Dobry 1991) based on the plasticity index of soil according to the soil classification is used for scenario modeling of each layer. In addition to this, geotechnical parameters like soil density and shear wave velocity are input for each layer based either on field investigation or sometimes empirically adopted. Kathmandu valley consists of inadequate database regarding the shear wave velocity profile and most of soil profiling are available for engineering bedrock level only. For this study, the Mississippi embayment shear profiles (Cramer 2006) are used for deep shear profiling. The impedance contrast is the governing factor for modification of motion parameters at any soil layer can be given as in expression (3)

$$C = \frac{\rho_2 v_2}{\rho_1 v_1} \qquad (3)$$

Herein, C is the impedance contrast; ρ_2 and v_2 are density and shear wave velocity for the stiffer layer and ρ_1 and v_1 represent the same for the overlying layer.

Ideally the material dynamic soil properties are chosen and each layer of soil is modified in terms of the one-dimensional Kelvin-Voigt Model represented by expression 4.

$$\tau = G\gamma + \eta \frac{\partial \gamma}{\partial t} \qquad (4)$$

Wherein, τ is shear stress; γ is shear strain; η is the viscosity and $\frac{\partial \gamma}{\partial t}$ is the rate of shear strain.

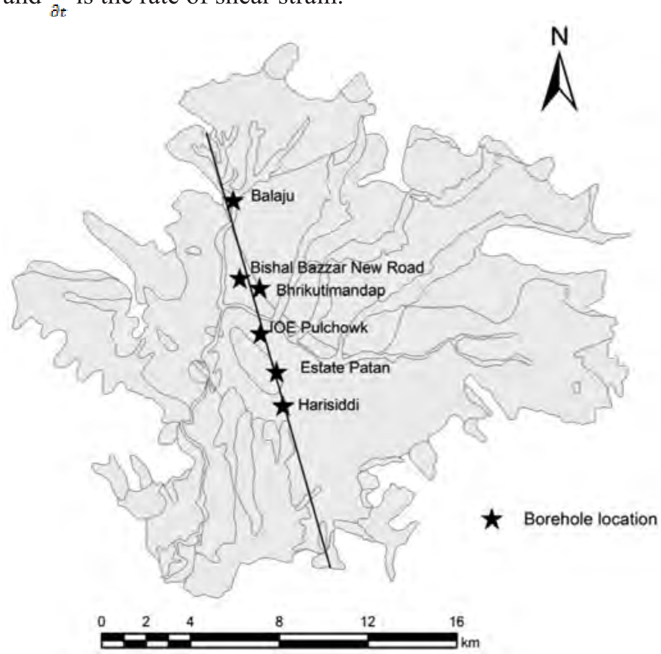

***Fig. 2 :** Borehole logs considered for study*

For each layer, modification in shear stress is made possible through implication of effective shear strain calculated on the basis of the underlying soil parameters. Iterative approach is used for each slice of soil and finally the last iteration is obtained for the first sublayer at the top of the soil column. Input motion is subjected to the bedrock level and parameters are calculated for the surface of each log. For this analysis, Uttarkashi earthquake (M_W 6.9) of 20 October, 1991 measured at Uttarkashi station, 34 km from the epicenter is used. The maximum acceleration of the input motion is 0.3102 g at 5.86 sec (Fig. 3) and the peak spectral acceleration is accounted as 1.2826 g at 0.3 sec (Fig. 4) [For details: see Chamlagain and Gautam 2015].

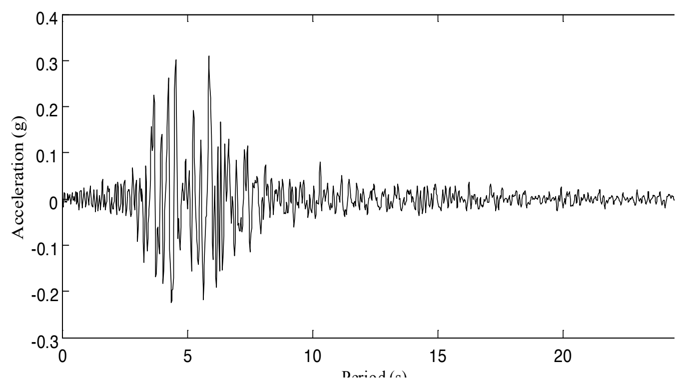

***Fig. 3 :** Acceleration time history of Uttarkashi earthquake (Chamlagain and Gautam 2015)*

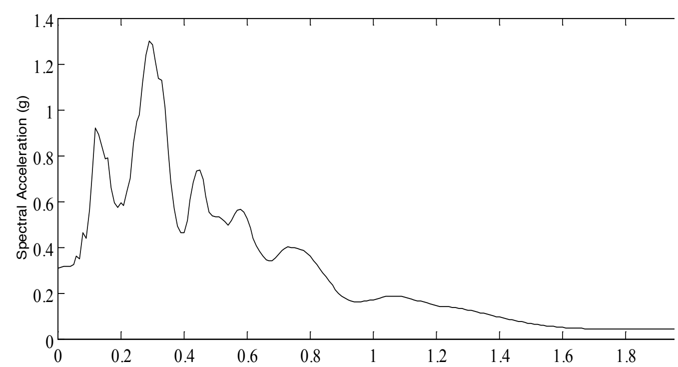

***Fig. 4 :** Response spectra for Uttarkashi earthquake*

As the scenario of this earthquake acquaints in many extent of Kathmandu valley seismic hazards and practices like the potential earthquake sources within the radius of 34 km, trend of earthquake occurrence in Himalaya and also in some respect the design based earthquake for structural and earthquake engineering purposes, the results would be more realistic and representative ones. There is no database access for recorded earthquakes in Nepal, so scenario based analysis and estimated parameters are scarce in Nepal.

IV. RESULTS AND DISCUSSION

Results

In the equivalent-linear platform, the ground motion parameters along with the spectral amplifications are obtained for the surface layer after application of the input motion on bedrock. Six sites from Kathmandu valley are analyzed and results are presented in terms of the spectral amplification, spectral acceleration, corresponding peak ground acceleration (PGA) and average shear wave velocity for 5% damping. Starting from the NW edge of the valley, horizontal distance and parameters estimated for each site are presented in table 1.

Borehole location	Maximum amplification	Peak spectral acceleration (g)	PGA (g)	Average shear wave velocity	Distance from NW edge of valley (km)
Balaju	11.1	1.36	0.44	478	3.30
Bishalbazzar	8.7	1.39	0.49	526	6.73
Bhrikutimandap	3.8	1.42	0.45	653	8.68
Pulchowk	4.8	1.35	0.46	495	9.52
Estate Patan	9.7	1.27	0.41	494	14.00
Harisiddi	10.3	0.65	0.33	374	16.38

The maximum amplification in central valley (Bhrikutimandap) is relatively low, however going outward from the center, the values of maximum amplification are found to be increased gradually. The maximum spectral amplification is found to be occurring in the frequency range of 0.2 to 0.6 Hz conjecturing the seismic demands of medium to high rise structures having similar frequency. However, majority of the analysis sites have the maximum amplification factor at the frequency of 0.4 Hz. Balaju (the farthest towards NW) and Harisiddi (the farthest towards SE) have the largest values of maximum spectral amplification. The general trend of spectral amplification in the center and edge of valley is presented in figures 3 and 4 respectively.

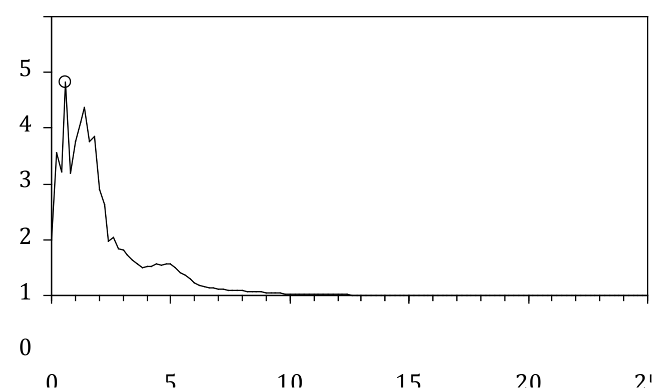

Fig. 3 : Spectral amplification pattern in central valley (Bhrikutimandap)

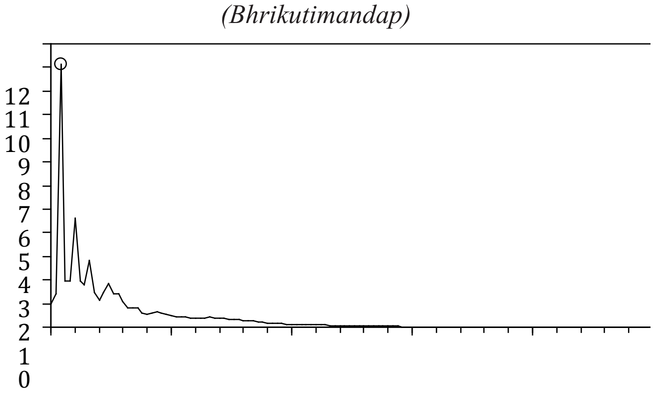

Fig. 4 : Spectral amplification pattern in valley edge (Balaju)

On the contrary, the spectral acceleration trend is depicted to be reversed as that of the spectral amplification while heading towards the edges from valley center. Figure 5 and 6 show the spectral acceleration pattern in central valley and edge respectively.

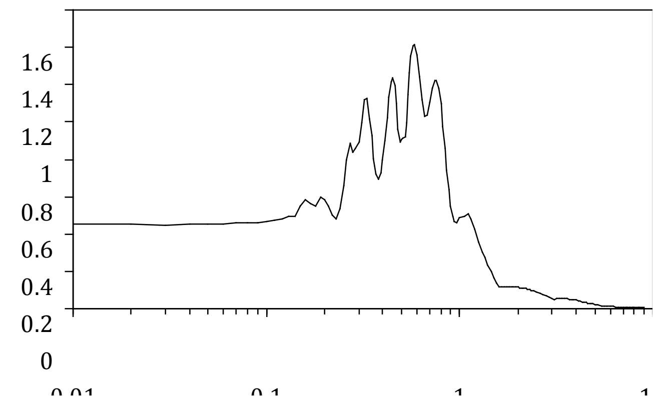

Fig. 5 : Acceleration response spectra in central valley (Bhrikutimandap)

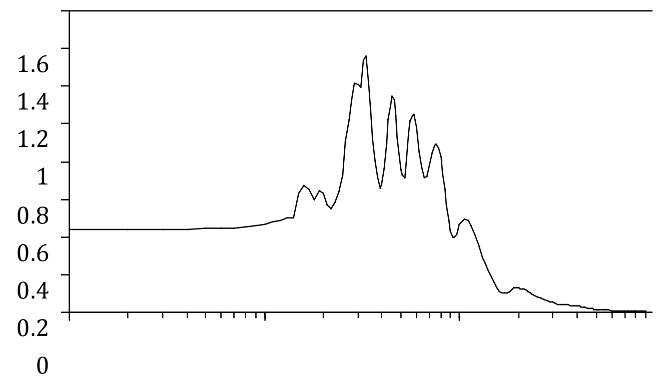

Fig. 6 : Acceleration response spectra in valley edge (Balaju)

Discussion

While considering the damage pattern of 1934 earthquake, likewise in the areas with higher population concentration, the valley periphery was severely devastated (e.g. Rana 1935; Dunn et al. 1939; Ritcher 1957; Pandey and Molnar 1988) and almost similar trend was observed during 2015 Gorkha earthquake (Gautam and Chaulagain 2016). The damage pattern was surprisingly irregular within the soft sediment deposits of Kathmandu valley. Evidently, basin geometry effect and basin edge effect, a combined phenomenon of refraction of seismic waves and concentration could be seen in those areas with higher reliefs are estimated in this study. The valley periphery has relatively sloping terrain of shallow soft sediment deposit hence basin edge effect should be the most probable cause of intense damage. These results even contend the occurrence of basin geometry effect in NW-SE section of valley from one dimensional analysis. The trend clearly connotes that central valley is likely to be suffered from the higher ground shaking and valley periphery would be hit by higher values of spectral amplification leading to severe damage in case of future events. This was also evident during 2015 Gorkha earthquake. It should be critically noted that, the narrow variation of PGA in entire valley seems less significant cause to underpin the damage statistics, rather spectral amplification elucidates well historically. Beside this geotechnical asperities and recent urbanization pattern have been well encompassed into the valley periphery as all of the peripheral areas are nowadays converted into municipalities. Both center and edges of valley show relatively long predominant period in response spectra, this suggests the possibility of vibration resonance during earthquakes like the 2015 Gorkha seismic event in case of particular types of structural forms. Long predominant period also closely justifies the damages in medium to high rise structures and monuments. The collapse of Dharahara in New Road (close to Bishalbazzar) and severe destruction in some medium rise structures in Balaju correlate with such long predominant period.

The UNDP Bureau of Crises Prevention and Recovery has ranked Nepal in 11th place as per the underlying seismic risk, however the geotechnical, geological and geomorphologic setups are not solely contributing in proneness rather construction practices to seismic codes inadequacy are remarkable to aggravate the seismic risk.

Most of modern building codes (e.g. IBC, UBC, and EURO code) from across the world are incorporating local site effects into buildings codes almost after every earthquake event; however, Nepal Building Code seems to be dormant in this practice. Without assuring seismic site effects into building codes, even engineered constructions shouldn't be understood as earthquake resistant constructions. In a small spatial variation, deep valleys like Kathmandu could show very different response, so soil dynamic characteristics are needed to be defined for every settlement in local scale. In addition to this, microzonation study is urgently needed for Kathmandu valley and other growing urban centers of Nepal. For these, detailed studies regarding the response of soil, underlying seismic hazard, knowledge and database management and implementation of findings into practices are key variants to bring substantial changes. The urban population is not also safe in Nepal with regard to seismic safety, as only 28.42% of buildings in urban areas of Nepal have their foundations as reinforced concrete pillar (CBS 2012). Moreover, Kathmandu valley is a hub for construction deficiencies (Gautam et al. 2016); as majority of existing buildings can't assure earthquake resistance and even new constructions do so.

V. CONCLUSION

Being characterized by inherent seismic hazard, Kathmandu valley will be suffered more than as observed during 2015 Gorkha earthquake. Beside the geotechnical and geological asperities, there are additional variants that lead to designate Kathmandu amongst the most vulnerable cities in the world. The topographical features, soft soil deposits and frequent seismic events due to interaction of Indo-Tibetan land masses are intriguing factors behind the inherent vulnerability. However, the imposed vulnerability is not addressed properly in terms of local soil behavior, construction practices, microzonation and development of site-specific design spectra for local level construction. In order to assure proper earthquake and structural engineering solutions to structural resilience, site effects are of pivotal importance so as to model the soil behavior and depict soil-structure interaction. For Kathmandu valley, two and three dimensional site effect estimations are urgently needed to identify the realistic soil behavior during earthquakes, however there are several limitations of one dimensional analysis. In this study, a section running NW-SE is chosen and analyzed in terms of basin geometry effects for two dimensional site effects analysis. As the findings suggest, the central valley is more susceptible to be observing higher ground motion parameters in terms of PGA. While going outward from the central portion, the spectral amplification is more intensified. Notably, significant variation in spectral amplification (3.8-11.1) is obtained rather than the PGA (0.33-0.49 g). As evidenced by the damage pattern of 1934 Bihar-Nepal and 2015 Gorkha earthquakes, the areas like Balaju and Harisiddi are thoroughly correlated to higher spectral amplifications. Studied logs from Bishalbazzar and Pulchowk have shown relatively higher values of spectral amplification compared to the central log of Bhrikutimandap.

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National Building Code and Damage Analysis of the 2015 Nepal Earthquake

Shoichi ANDO

Prof., National Graduate Institute for Policy Studies (GRIPS), Tokyo, JAPAN, e-mail: andosho69@gmail.com

ABSTRACT

In 2002, prior to the formal entry into force of the code, Lalitpur Sub-Metropolitan City (LSMC) initiated the implementation of National Building Code (NBC), becoming the first municipality in Nepal to implement NBC. Kathmandu Metropolitan City (KMC) followed in 2004, Dharan in 2006, Illam in 2008, Hetauda in 2010, Birgunj, and Byas in 2011. In total 12 municipalities implemented NBC by 2012 in Nepal. In addition, 5 municipalities in 2013 and 9 municipalities in 2014 newly started NBC and in total 26 municipalities implemented NBC, before the 2015 Nepal Earthquake.

Number of fatalities concentrates on Ward 15 and Ward 16, following Ward 6, and Ward 22 (and Ward 29) in KMC. The heaviest damaged ward in LSMC resulted in no more than 20 fatalities. Human damage concentrated areas are located in north-west areas in KMC, while south-east areas are heavily affected in Bhaktapur municipality. LSMC has less earthquake damage, if compared with other two cities. One of the reasons of the damage character is related to NBC implementation period.

Keywords: *National Building Code ; Human damage ; KMC; LSMC; Bhaktapur municipality*

I. INTRODUCTION

After 1988 earthquake magnitude of 6.7 Rector Scale in eastern Nepal resulting heavily life loss and numerous buildings including hospitals, schools were severely damaged, it was realized that most of the houses are highly venerable to earthquake of event moderate intensities due to lack of knowledge of the earthquake safety measures. Nepal government Ministry of Physical Planning and Works (MPPW; former Ministry of Housing and Physical Planning) and Department of Urban Development and Building Construction (DUDBC; former Department of Buildings) drew attention for the urgent need of Nepal National Building Code (NBC). The draft National building code was prepared in 1993. It was the first official document prepared which deals primarily with matters of earthquake safety of buildings.

The DUDBC of MPPW developed the NBC in 1993 with the assistance of the United Nations Development Program (UNDP) and United Nations Centre for Human Settlement (UN-HABITAT). NBC went into force when the Building Construction System Improvement Committee (established by the Building Act 1998) authorized MPPW to implement the code. The Ministry published a notice in the Gazette in 2006 and the implementation of NBC became mandatory in all Municipalities and some Village Development Committees (VDCs) in Nepal.

In 2002, prior to the formal entry into force of the code, Lalitpur Sub-Metropolitan City (LSMC) initiated the implementation of NBC, becoming the first Municipality in Nepal to implement NBC. Kathmandu Metropolitan City (KMC) followed in 2004, Dharan Municipality in 2006, Illam in 2008, Hetauda in 2010, Birgunj in 2011, Byas municipality in 2011, Butwal municipality in 2012, Bharatpur in 2012, Dhulikhel in 2012, Banepa in 2012 and Panauti in 2012. Other municipalities are also applying NBC with the passage of time. In total 12 municipalities implemented NBC by 2012 within 191 total municipalities in Nepal. 5 municipalities in 2013 and 9 municipalities in 2014 newly started NBC and in total 26 municipalities are now implementing NBC in Nepal. (Table 1)

Legal arrangement and responsible organizations to implement NBC are given in Table 2 below.

Table 1. *Name of Municipalities Implementing NBC (source: MOHA, 2015.05)*

No.	EQ	Name of Municipalities	Population (2011)	Implementation	(Year of Nepal)
1.	*1*	*Lalitpur*	*220,802*	*2002*	*2059*
2.	*1*	*Kathmandu*	*975,453*	*2004*	*2061*
3.		*Dharan*	*116,181*	*2006*	*2063*
4.		*Ilam*	*18,633*	*2008*	*2065*
5.	*0*	*Hetauda*	*84,671*	*2010*	*2067*
6.		*Birganj*	*135,904*	*2011*	*2068*
7.		*Byas*	*42,899*		
8.		*Butawal*	*118,462*	*2012*	*2069*
9.		*Bharatpur*	*143,836*		
10.	*1*	*Dhulikhel*	*14,283*		
11.	*1*	*Banepa*	*24,764*		
12.	*1*	*Panauti*	*27,358*		
13.		*Pokhara*	*255,465*	*2013*	*2070*
14.		*Dhangadhi*	*101,970*		
15.		*Ghorahi*	*84,822*		
16.		*Biratnagar*	*201,125*		
17.		*Damak*	*75,102*		

18.	*1*	*Kirtipur*	*65,602*	*2014*	*2071*
19.	*1*	*Madhyapur Thimi*	*83,036*		
20.		*Bhimdatta*	*114,647*		
21.		*Tulsipur*	*51,537*		
22.		*Gulariaya*	*55,747*		
23.		*Putali Bazar*	*30,704*		
24.		*Siddharthanagar*	*63,483*		
25.		*Triyuga*	*70,000*		
26.		*Bhadrapur*	*18,164*		

Note 1: *Blue "1" = 2015 EQ affected city, Note 2: light pink city = more than 100,000 population*

Table 2. *Legal Arrangement and Responsible Organizations*

Legal Mechanism	Responsible organizations	Envisaged role
Building Act 1998 (Rev. 2007)	Building Construction System Improvement Committee	Devise Building Code, facilitate enforcement, disseminate code, monitor implementation, revise NBC code
	MPPW	Approve the Building Code
		Publish notice of mandatory implementation of NBC
	DUDBC	Implement Building Code in areas outside of Municipal jurisdiction
		Supervise compliance with NBC
	Municipalities	Ensure compliance with NBC
Local Self Government Act 1999 (Decentralization Act)	Municipalities	Building permit (does not include provision of Building Code)
	House owners in municipal area	Comply with municipal rules and secure formal permit before construction
National Building Code 2003	All concerned	Approved NBC
Notice of MPPW in Nepal Gazette (Feb. 13, 2006)	All municipalities, and some VDC headquarters	Implementation of Building Act

Source: *Building Act-1998, Local Self-governance Act 1999 and NBC-1994*

Nepal National Building Code has 23 parts. The first NBC is '000', which means "Requirements for State of the Art Design" and lays out general provisions of the individual building codes. For clarity, Code number and Code titles of NBC are given in Table 3.

Table 3. *Code number and Code titles of NBC*

Code Number	**Code Title** (GL: Guideline, MRT: Mandatory Rules of Thumb)
NBC 000: 1994	Requirements for State-of the- Art Design
NBC 101: 1994	Materials Specifications
NBC 102: 1994	Unit Weight of Materials
NBC 103: 1994	Occupancy Load
NBC 104: 1994	Wind Load
NBC 105: 1994	Seismic Design of Buildings in Nepal
NBC 106: 1994	Snow Load
NBC 107: 1994	Provisional Recommendation on Fire Safety
NBC 108: 1994	Site Consideration for Seismic Hazards
NBC 109: 1994	Masonry: Unreinforced
NBC 110: 1994	Plain and Reinforced Concrete
NBC 111: 1994	Steel
NBC 112: 1994	Timber
NBC 113: 1994	Aluminum
NBC 114: 1994	Construction Safety
NBC 201: 1994	MRT: Reinforced Concrete Buildings with Masonry infill
NBC 202: 1994	Mandatory Rules of Thumb (MRT): Load Bearing Masonry
NBC 203: 1994	GL for Earthquake Resistant Building Construction: Low Strength Masonry
NBC 204: 1994	GL for Earthquake Resistant Building Construction: Earthen Building (EB)
NBC 205: 1994	MRT: Reinforce Concrete Buildings Without Masonry Infill
NBC 206: 2003	Architectural Design Requirements
NBC 207: 2003	Electrical Design Requirements for (Public Buildings)
NBC 208: 2003	Sanitary and Plumbing Design Requirements.

Note: NBC 201 to NBC 205 do not need structural calculation. "(pre-) non-engineered"

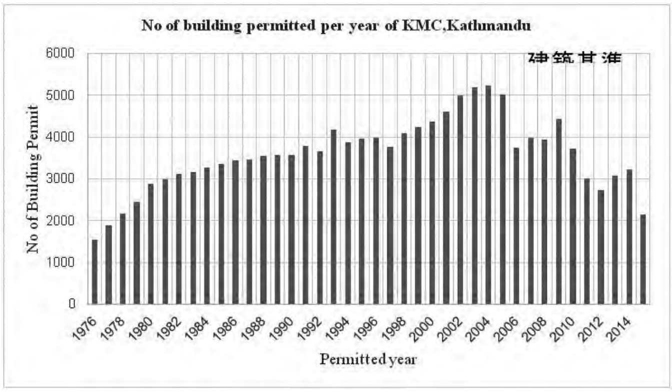
Nepal Building Code is, further, classified as per application in the construction industry of Nepal, which is given in Table 4 below.

Table 4. *Classification of NBC as per Application*

S.N	Type of Building Code			Purpose
1.	International State-of-Art Applicable codes: NBC 000			Applicable to large building structures. The structures must comply with existing international state-of the art building codes
2.	Professionally Engineered Buildings Applicable codes:			Building designed and constructed under supervision of engineers, buildings with plinth area more than 1,000 sq. ft, buildings having more than 3 stories, buildings with span more than 4.5 m and buildings with irregular shapes
	NBC 101	NBC 107	NBC 113	
	NBC 102	NBC 108	NBC 114	
	NBC 103	NBC 109	NBC 206	
	NBC 104	NBC 110	NBC 207	
	NBC 105	NBC 111	NBC 208	
	NBC 106	NBC 112		
3.	Mandatory Rules of Thumb Applicable codes: NBC 201, NBC 202, NBC 205			Buildings of plinth area less than 1,000 sq. ft, less than 3 stories, buildings having span less than 4.5 m and regular buildings designed and constructed by technicians in the areas where professional engineer's service is not available
4.	Guidelines of Remote Rural Building (Low Strength Masonry/Earthen Building)			Building constructed by local masons in remote areas and not more than 2 stories

Source: *NNBC 000 – 1994*

(Reference) ***Fig. 1*** *Number of buildings permitted per year of Kathmandu Metropolitan City*



(Nagendra R. Yadav)

II. Analysis of Human Damages

Fig. 2 shows the location of three target cities in the Kathmandu Valley. i.e. Kathmandu Metropolitan City (KMC), Lalitpur Sub-Metropolitan City (LSMC), and Bhaktapur municipality.

In Kathmandu Valley, there were five municipalities before recent VDC consolidation. Other two cities were Kiritpur in Kathmandu District and Madhyapur Thimi in Bhaktapur District. Within five municipalities, four municipalities have implemented NBC before 2014. Especially, LSMC and KMC are the first cities that started NBC implementation in Nepal. Almost 10 years have passed since the initiation of building structural safety control by municipality. However, Bhaktapur municipality has not started NBC implementation by 2014.

This study tries to compare the damage of three cities where similar seismic ground motion was observed at the recent Nepal Earthquake in April and May 2015.

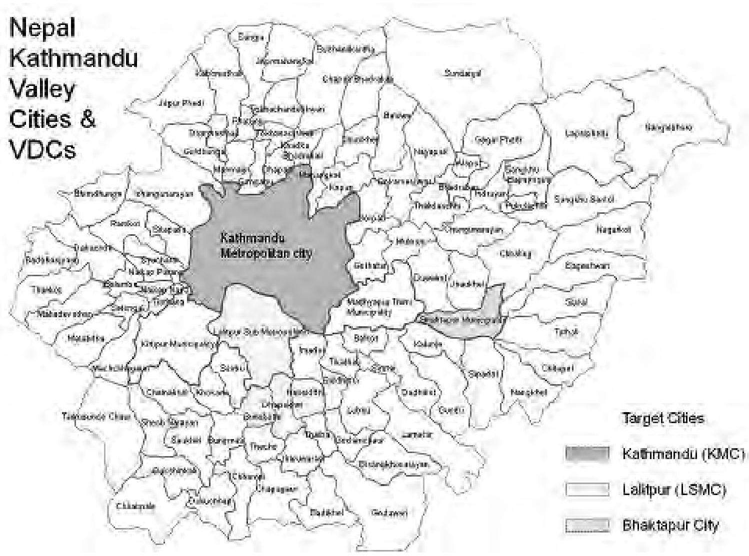


Fig. 2 *: Three Target Cities in Kathmandu Valley (map: before VDC consolidations)*

The Fig. 3 to Fig. 5 shows the comparison of three cities in Kathmandu Valley, i.e. Kathmandu Metropolitan City (KMC), Lalitpur Sub-Metropolitan City (LSMC), and Bhaktapur

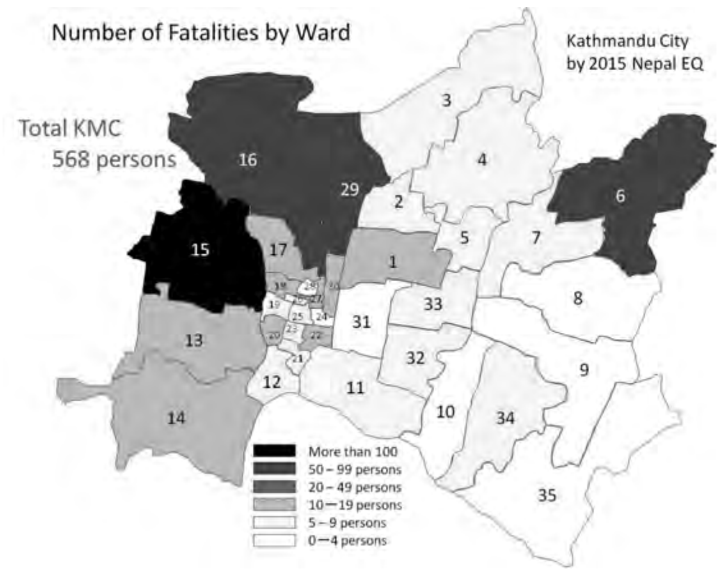


Fig. 3 : *Number of Fatalities of KMC by Ward (2015 EQ) and Ward 16 (53 fatalities) and other Wards in Kathmandu KMC, and Lalitpur LSMC have no more than 20 fatalities.*

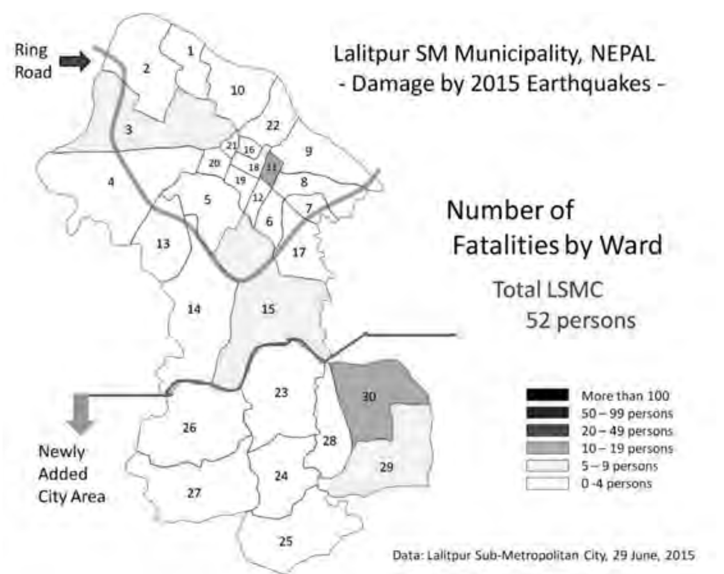


Fig. 4 : *Number of Fatalities of LSMC by Ward (2015 EQ)*

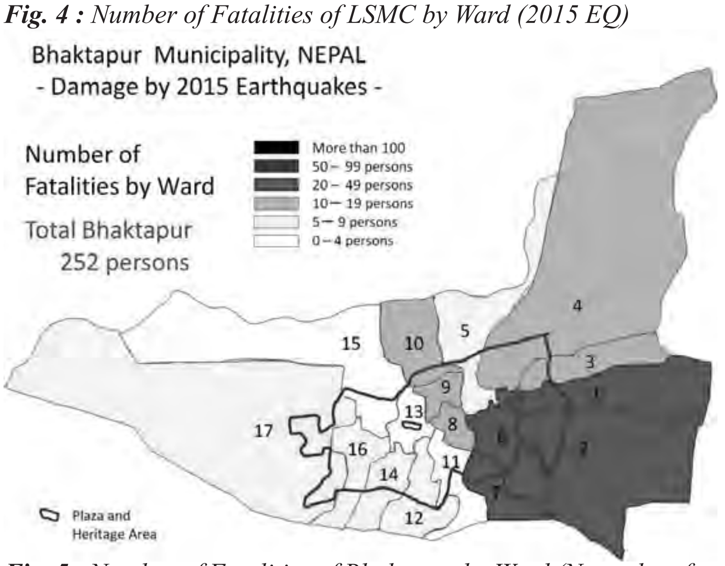


Fig. 5 *: Number of Fatalities of Bhaktapur by Ward (Note: data from 3 municipalities May 2015)*

municipality.

All figures are coloured under the same standard level of classification and easily compared.

(Source: See Section 4 "Basic Data", 2 June, 2016 KMC; 26 June, 2016 LSMC; 31 May, 2016 Bhaktapur)

1) Number of fatalities concentrates on Ward 15 (137 fatalities), following Ward 29 (91 fatalities), Ward 6 (66 fatalities)
2) These cities have old area where old buildings are concentrated, however those old areas resulted in comparatively less fatalities.
3) The human damage concentrated areas are located in north-west and north-east areas in Kathmandu city, while south-east areas are heavily affected in Bhaktapur (and also in Lalitpur LSMC) municipality.
4) Lalitpur City (LSMC) has less human damages compared with other two cities.

Note: The reasons why above-mentioned phenomena can be observed are now investigated based on detailed data by the National Graduate Institute for Policy Studies (GRIPS).

The following Fig. 6 to Fig. 8 show the ratio of fatalities per total population (National Census 2011). The figures identify almost similar characteristics seen in Fig. 3 to Fig. 5.

Every Ward show the ratio of fatalities less than 1%, and if compared with the ratio of the 1995 Great Hanshin-Awaji Earthquake (Kobe city 0.31%), and the tsunami affected areas of the 2011 Great East Japan Earthquake. (Most of Wards in Kathmandu and Lalitpur cities have less than 0.1 % ratio of fatalities.)

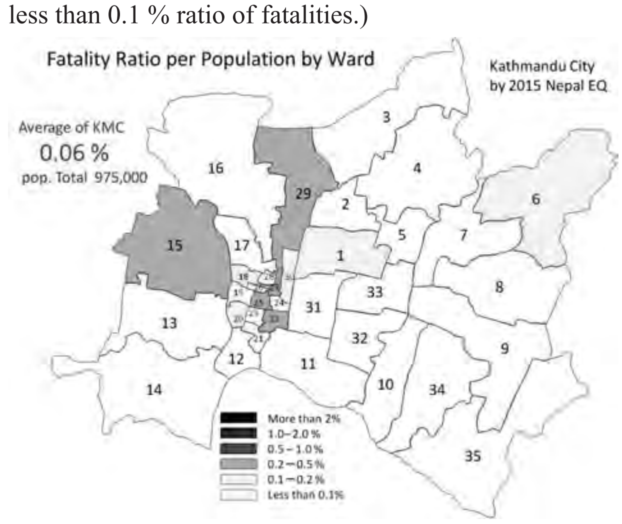


Fig. 6 *Fatality Ratio of KMC by Ward (2015 EQ)*

The following Fig. 9 to Fig. 11 show the number of heavily damaged houses in each city.

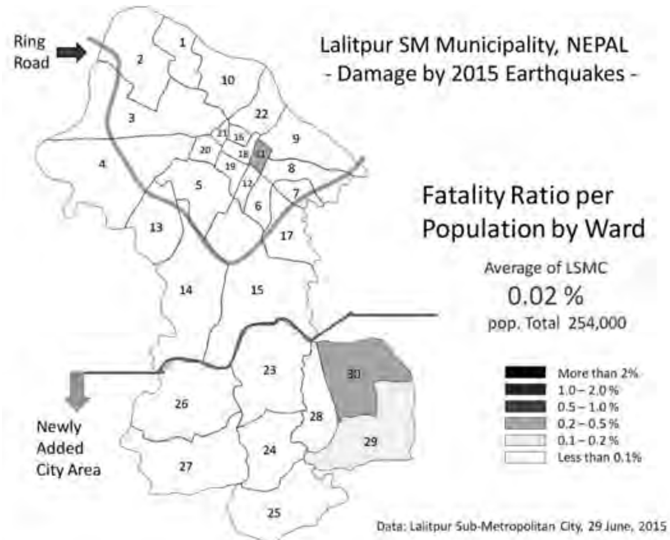


***Fig. 7 :** Fatality Ratio of LSMC by Ward (2015 EQ)*

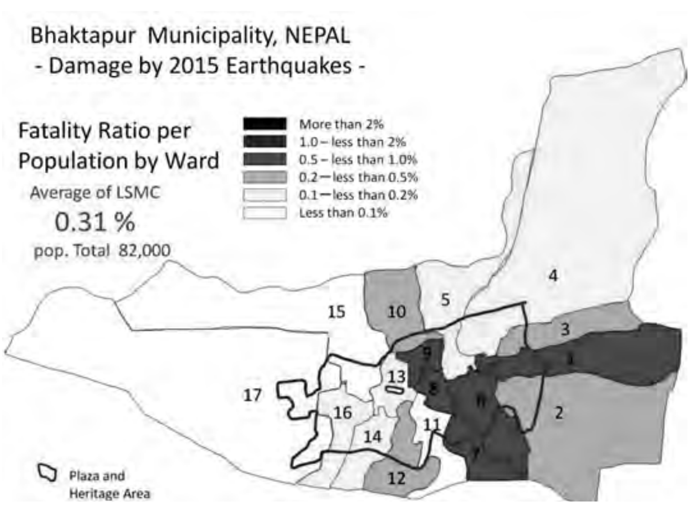


***Fig. 8** Fatality Ratio of Bhaktapur by Ward (2015 EQ)*

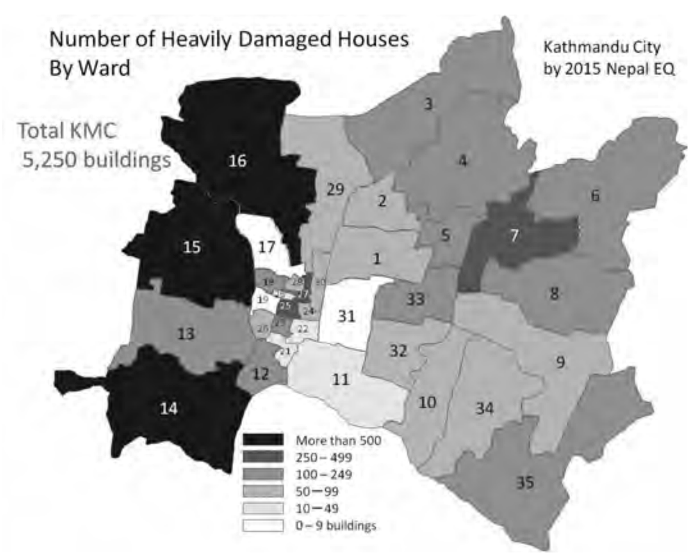
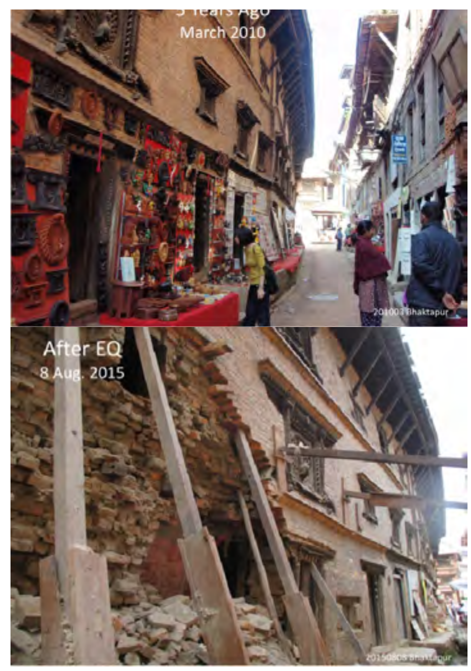


***Fig. 9 :** Heavily Damaged Houses of KMC by Ward*

*(**Note:** all data from 3 municipalities as of May 2015) Ref. Photo 2 (Bhaktapur) 2010/2015*

Ref. Photo 1 (KMC) 2006/2015



1) In KMC, building damage in Ward 7 and Ward 14 where the less human damages observed are severer compared with Ward 6 and Ward 29 where more human damages were recorded.

2) In Lalitpur LSMC, Ward 8, Wards 25-27 and in Bhaktapur Ward 14 have less fatalities, however they have comparatively heavy housing and building damages.

Fig. 12 to Fig. 14 show the “Number of Fatalities per 100 Heavily Damaged Houses” by Ward in Kathmandu

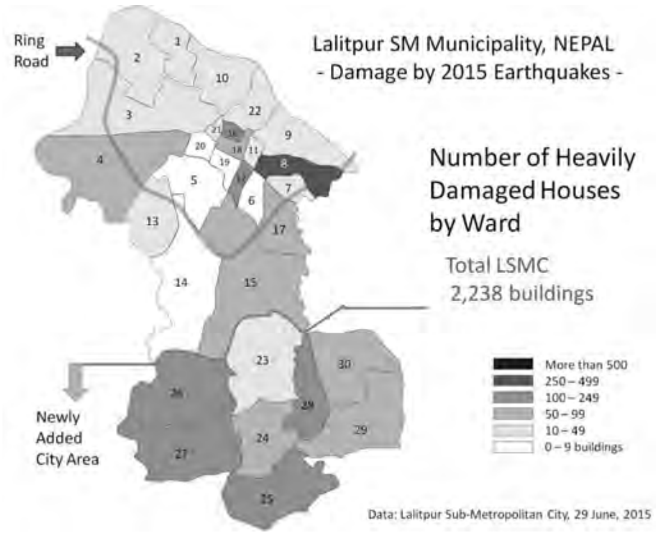


Fig. 10 : Heavily Damaged Houses of LSMC by Ward

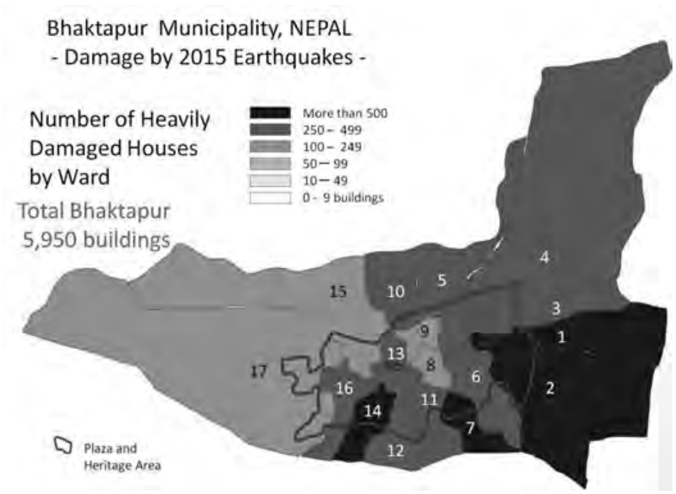


***Fig. 11 :** Heavily Damaged Houses of Bhaktapur by Ward Ref. Photo 4 (Bhaktapur) 2010/2015*

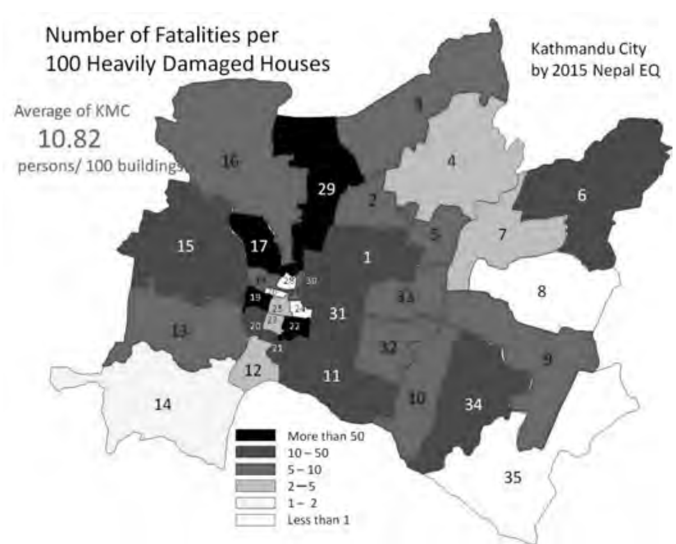
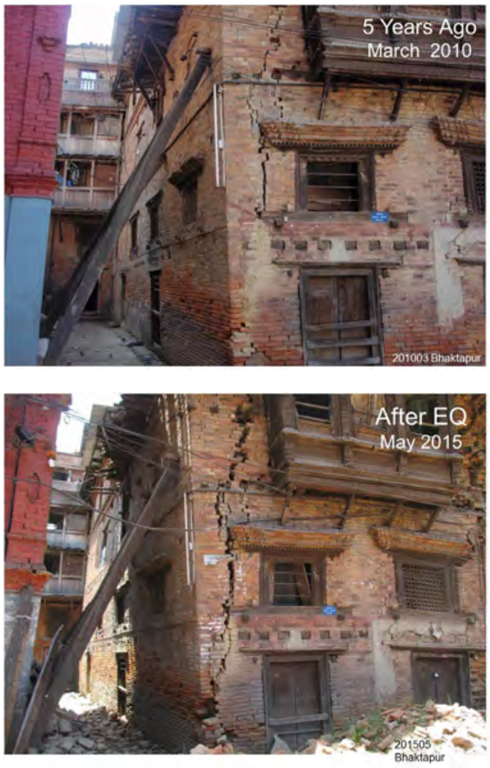


***Fig. 12**: Number of Fatalities per 100 Heavily Damaged Houses of KMC by Ward*

Ref. Photo 3 (KMC) 2006/2015



Ref. Photo 4 (Bhaktapur) 2010/2015

(KMC), Lalitpur (LSMC) and Bhaktapur municipality.

(Source: See Section 4 "Basic Data", 2 June, 2016 KMC; 26 June, 2016 LSMC; 31 May, 2016 Bhaktapur)

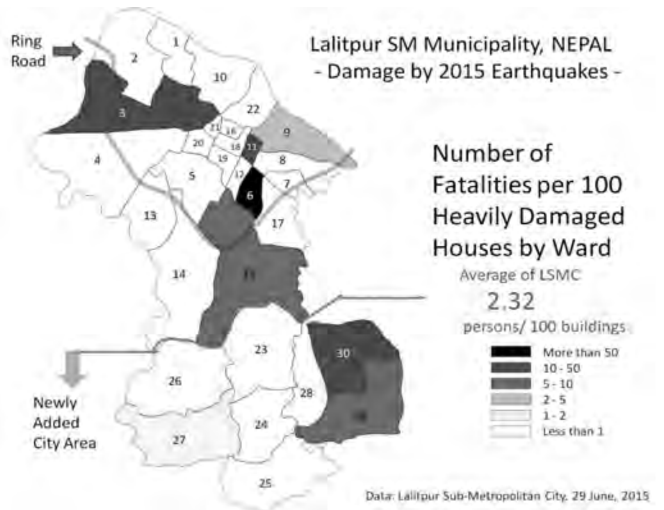


Fig. 13 : *Number of Fatalities per 100 Heavily Damaged Houses of LSMC by Ward*

The detailed analyses from these three figures are described in the latter parts. The original data collected from three municipalities are also shown at the last part (Sect. 4) as reference.

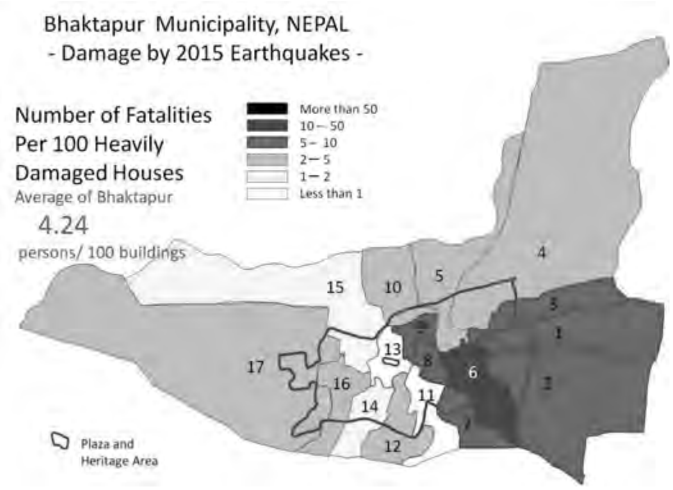


Fig. 14 : *Number of Fatalities per 100 Heavily Damaged Houses by Ward in Bhaktapur city*

III. CONCLUSIONS

The following 7 points shows tentative analysis results from the data collected from KMC, LSMC and Bhaktapur municipality after the 2015 Nepal Earthquake in May and June 2015.

Firstly, from the comparison of all figures (Fig. 3 to Fig. 14), the following facts are observed;

1) In KMC, within the highest 4 wards (No. 29, 17, 22, 19) of Fig. 12 that have more than 50 persons, only Ward 29 is included in the highest 4 wards (No. 15, 29, 6, 16) of fatality in Fig. 3.

2) In LSMC, within the highest 4 wards (No. 6, 11, 3, 30) of Fig. 13 that have more than 10 persons, Ward 11 and 30 are included in the highest 4 wards (No. 11, 30, 15, 29) of fatality in Fig. 4.

3) In Bhaktapur municipality, within highest 4 wards (No. 6, 9, 1, 8) of Fig. 14 that have more than 7 persons, Ward 1 and 6 are included in the highest 4 wards (No. 1, 7, 2, 6) of fatality in Fig. 5.

4) From the highest 4 wards of fatality ratio in KMC, only Ward 22 is included (in the Fig. 6 highest 4 wards) and in LSMC, Ward 11 and 30 are included, while in Bhaktapur municipality Ward 1, 6 and 9 are included (in Fig. 7 and 8 highest 4 wards). That means the correlation between human damage and the indicator of Fig. 9 to 11 become lower in order of KMC, LSMC and Bhaktapur municipality.

   In addition, correlation between the above-mentioned indicator (fatalities per 100 heavily damaged houses; this indicator shows relationship of building damage and human damage. When a large building collapsed and many fatalities occurred, the ratio becomes high) and number of heavily damaged houses can be analysed similarly as follows;

5) In KMC, within highest 4 wards (No. 14, 15, 16, 25) of heavily damaged houses, only Ward 15 shows more than 10 persons in Fig. 12, while Ward 17 and Ward 19 are the lowest. This means that in KMC, the wards that have less housing damage are larger number of fatalities by one heavily damaged house (building).

6) In LSMC, within highest 4 wards (No. 8, 16, 12, 27) of heavily damaged houses, only Ward 27 shows more than 2 persons in Fig. 13, while other three wards have no fatality.

7) In Bhaktapur municipality, within highest 4 wards (No. 14, 7, 2, 1) of heavily damaged houses, there is no ward that shows more than 10 persons, however Ward 7, Ward 2 and Ward 1 have more than five persons.

Above-mentioned seven facts are observed from Fig. 3 to Fig. 14. The author has a plan to investigate and to analyse what happened and occurred in the characteristic wards. (The basic data used in this paper as shown in next page, municipality provided in May and June 2015. Ministry of Home Affaires of Nepal also put detailed data of all fatality on web-site in middle of 2015. The other data is revised damage by Ward in KMC as of Oct. 2015. Those data are not analysed in this paper.)

From the comparison of three cities, most of indicators except number of fatality and the fatality per 100 heavily damaged houses, the damage order shows “Bhaktapur > KMC > LSMC” in all Fig.. Even though the total number of population and houses are in order of “KMC > LSMC > Bhaktapur”, Bhaktapur municipality has more damages than KMC and LSMC. (The red number in each Figure indicates total or average number of the city.)

Lastly from the Table 1, LSMC and KMC implemented NBC from the first period of Nepali building control. One of the reasons of the damage character is related to NBC implementation period. However, further investigation is needed to prove such effect clearly. The Fig. 1 shows that most of the buildings in KMC were constructed before implementation of NBC in 2004. Even if the buildings constructed after 1994 (establishment of code, before enforcement) are almost half of the current stock.

IV. BASIC DATA

Building Damage Data of Kathmandu Municipality 2015 Nepal EQ

			Building Damage		Human Damage	
Ward	Population person	Family No. families	Heavily buidings	Partially buildings	Dead person	Injured person
1	8008	1917	73	196	13	105
2	13448	3599	86	198	5	12
3	34866	9145	113	320	8	7
4	47362	12030	150	783	5	11
5	18320	4774	112	306	6	16
6	60344	15434	241	627	66	38
7	51581	13559	283	946	8	6
8	10738	2773	130	351	1	5
9	40371	10417	56	405	3	2
10	39820	10571	50	73	4	3
11	17765	4416	25	189	6	1
12	13262	3173	130	400	5	11
13	40456	10207	188	1208	15	14
14	58495	15472	743	2060	12	0
15	54476	14093	555	1010	137	0
16	84441	22715	694	1251	53	17
17	25926	6394	9	1284	13	2
18	10746	2746	128	376	11	12
19	10711	2632	8	102	6	
20	10968	2844	57	1144	15	22
21	13727	3389	39	1120	7	
22	5699	1250	19	195	16	8
23	8357	1991	169	380	5	35
24	3488	742	59	113	0	0
25	3406	788	311	200	7	15
26	4133	947	34	491	0	0
27	7592	1888	268	124	16	0
28	5611	1370	67	85	0	5
29	45052	12252	58	786	91	
30	8563	1914	58	679	15	8
31	16211	4112	9	77	1	4
32	33316	9298	54	89	5	
33	25694	6876	107	147	6	10
34	66121	17772	63	164	7	8
35	76299	20792	104	912	0	3
Total	975453	254292	5250	18791	568	380

Analysed by GRIPS Disaster Management Policy Program (Tokyo)
Data from Disaster Management Director of Kathmandu (on 2 June, 2015)
Data on population and family numbers are based on Nepal National Census 2011

Note 1: *It will be needed to investigate to make clear whether the number of fatalities is based on the location of death or location of family registered municipality (ward).*

Note 2: *KMC issued revised human and building damage data by Ward of KMC in Oct. 2015. This study paper uses the data in May 2015 (also issued by KMC), in order to compare with other two cities' data in May.*

Building Damage Data of Bakhtapur Municipality 2015 Nepal EQ

			Building Damage		Human Damage	
Ward	Population person	Family No. families	Heavily buildings	Partially buildings	Dead person	Injured person
1	4805	972	513	155	43	56
2	6694	1456	527	66	33	66
3	3427	697	290	78	15	21
4	11011	2632	307	343	14	18
5	5141	1124	322	66	8	14
6	3126	604	252	66	29	43
7	4437	960	564	156	34	35
8	3138	620	214	162	16	12
9	2071	405	152	42	13	6
10	4509	898	421	69	10	11
11	3287	629	304	126	3	4
12	3782	761	428	26	9	10
13	2225	417	345	111	3	5
14	4466	954	584	60	8	1
15	6044	1374	237	199	3	2
16	3684	793	260	159	6	15
17	9901	2342	230	208	5	60
Total	81748	17639	5950	2092	252	397

Data from Executive Officer of Bakhtapur (on 31 May, 2015)
Analysed by GRIPS Disaster Management Policy Program (Tokyo)
Data on population and family numbers are based on National Census 2011

Building Damage Data of Lalitpur Municipality 2015 Nepal EQ

			Building Damage		Human Damage	
Ward	Population person	Family No. families	Heavily buidings	Partially buildings	Dead person	Injured person
1	8434	2221	38	61	0	0
2	19061	4839	30	306	0	4
3	14082	3528	17	340	5	10
4	15367	3913	53	186	0	3
5	6404	1516	6	122	0	2
6	6700	1563	6	470	4	2
7	7849	1839	15	176	0	0
8	11400	2816	410	232	0	5
9	13908	3484	45	384	1	18
10	6554	1729	14	112	0	0
11	4458	1010	27	427	12	11
12	5891	1342	223	253	0	2
13	14867	3772	19	221	0	0
14	21232	5438	0	0	0	0
15	13858	3480	82	822	3	14
16	4362	858	242	340	0	3
17	10644	2678	90	109	0	0
18	5777	1200	68	788	0	0
19	7385	1774	3	198	0	0
20	7721	1978	7	346	0	1
21	4659	1143	23	257	0	2
22	10109	2460	20	300	0	0
23	7002	1854	28	375	0	4
24	2424	571	51	441	0	12
25	3252	753	117	317	1	0
26	5813	1377	139	336	0	0
27	4279	1020	214	94	4	0
28	2872	756	109	35	0	2
29	4159	1070	74	310	6	21
30	3705	911	68	156	11	15
	(220802)	(54581)				
Total	254308	67893	2238	8509	57	131

Data from Executive Officer of Lalitpur (on 26 June, 2015)
Data on population and family: based on National Census 2011
Numbers of ( ): before consolidation of Ward 23 to 30 to LSMC

Newly added Wards in LSMC as of 2014

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Harish Chandra Lamichhane. *Comparison of Different Quick Inspection Sheets and Proposal of New One for RC Buildings in Nepal*, Master Paper 2014/2015, Disaster Management Policy Program, National Graduate Institute for Policy Studies (GRIPS), Sept. 2015.

Nagendra Ray YADAV. *Effectiveness of Building Code Implementation in Nepal*. Master Paper 2014/2015, Disaster Management Policy Program, National Graduate Institute for Policy Studies (GRIPS), Sept. 2015.

Damage Assessment of RC Buildings of Kathmandu Valley after Gorkha Earthquake 2015

Dr. Rajan SUWAL
Senior Lecturer, Institute of EngineeringCentral Campus,
Tribhuvan University, Pulchowk
rajan_suwal@yahoo.com
Post-doctoral Researcher, Faculty of Engineering,
University of Porto, Porto, Portugal

ABSTRACT

After the Gorkha Earthquake Nepal 2015, several buildings were damaged and collapsed in Kathmandu valley. In this context, the proposed paper presents an overview of the damage that was observed and the general and detail information collected through Rapid Visual Damage Assessment of 64 numbers of buildings around the Kathmandu valley. The main objective of this research work was to make the database of damaged buildings, to find out the general causes of failure of these buildings, to know the lesson learnt from the Gorkha earthquake 2015 and to recommend for design and construction of RC building in future. The existing status of the buildings are obtained by single parameter analysis of collected data in which general information and deficiencies of buildings are ploted out by using statistical tools, then correlation between these parameters is perfomed. The study showed that the selected buildings lie in IX MMI comparing with the seismic hazard map of Kathmandu valley prepared by UNDP. Most of the buildings were damaged due to the not properly designed, low quality of construction materials.

Keywords **:** *Damage assessment, RC Framed Buildings, Gorkha Earthquake 2015*

I. INTRODUCTION

Nepal lies in Himalayan region and Himalaya is highly vulnerable to earthquake. Young geology of Nepal is due to Indian Plate converging towards the Eurasian Plate and it still in process, It has experienced great earthquakes in past. The seismic gap in certain region of Nepal reflects the highly seismically active zone [1].

In 1934 earthquake large destruction of only masonry structure occur as RC structure has not been introduced. So, reinforced concrete structure in Nepal has not experience large earthquake yet. The people don't know the behaviour under seismic action and constructing building as they want. This may lead to heavy loss of life, property in the future earthquake. We should always be prepared for forthcoming earthquake so that the destruction may be reduced.

For the last 15 to 20 years there has been a proliferation of reinforced concrete (RC) framed buildings constructed in the urban and semi-urban areas of Nepal. Most of these buildings have been built on the advice of mid-level technicians and masons without any professional structural design input. These buildings have been found to be significantly vulnerable to a level of earthquake shaking that has a reasonable chance of happening in Nepal. Hence, these buildings, even though built with modern materials, could be a major cause of loss of life in future earthquakes.

II. SEISMICITY OF NEPAL

Earthquake are comon events in Nepal. Fig 2.1.a shows the record of past earthquake around the plate boundary in the south Asia. It is evident that Indian plate is sub - ducting into Eurasian plate every year by 3.8 cm to 4.8 cm [2]. This movement of plates is responsible for accumulated strain energy which is released in time to time. In this figure the largest strain energy release on 1950, Asam-Tibet Earthquake is shown.

2.1 Recent Gorkha Earthquake 2015

Fig 2.1.b shows the location of earthquake epicentre of recent Gorkha earthquake in Nepal map. In the recent, an earthquake of M7.8 occurred in 77 km NW of Kathmandu (in the boarder of Gorkha and Lamjung) at 11:56 on 25 April 2015 with shallow depth of 15 km with maximum Mercalli Intensity of IX, lasting approximately fifty seconds.

The Fig 2.1.b, it shows more than 100 aftershocks that have occurred since the magnitude 7.8 earthquake in Nepal on April 25, 2015 [3]. Nepal faced continued aftershocks throughout the country at the intervals of 15–20 minutes, with one shock reaching a magnitude of 6.7 on 26 April at 12:54:08.

The largest aftershock is a magnitude 7.3 occurred in 18 km south-east of Kodari and epicentre is in boarder of Dolkha and Sindhupalchowk district at 12.51 on 12 May 2015. The 1833 and 1934 stars represent the most recent large historical earthquakes on this portion of the plate boundary.

It was recorded that officially 8,857 and in total 9018 people dead and 21,952 injured. Ancient monuments were collapsed at UNESCO world heritage sites in the Kathmandu Valley including some at the Bhaktapur Durbar Square, the Kathmandu Durbar Square, the Patan Durbar Square, the Changu Narayan Temple and the Swayabhunath Stupa.

The 7.8-magnitude earthquake completely damaged 1,38,182 houses across Nepal and partially damaged 1,22,694 other homes. In total, 10,394 government buildings have collapsed and over 13,000 were partially damaged, according to Home Ministry sources.

Fig. 2.1.a - Earthquake in South Asia
Source:http://mashable.com/2015/05/12/deadly-aftershock-in-nepal

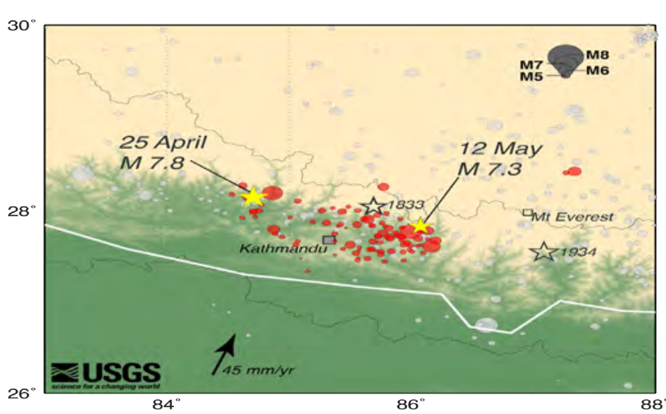

Fig. 2.1.b - Map showing more than 100 aftershocks that have occurred since the magnitude 7.8 earthquake in Nepal on April 25, 2015.
Source:http://www.usgs.gov/blogs/features/usgs_top_story/magnitude-7-8-earthquake-in-nepal/

2.2 Field Survey

The location of the field survey of damaged buildings due to recent earthquake is choosen as Kathmandu Valley since most affected area was identified to be in this region. During the field visit rapid visual screening was performed. The collected information is tabulated. First of all, general information and sorrounding of selected building was collected from global overview. Then information about elevation and plan was collected measuring interior and exterior wall thickness, floor height and bay width. Number of story and of bays in both direction were noted out. Then after detailed information of structural element such as dimension of section size of beam, column, slab along with reinforcement details were collected. Information about material characterstics of concrete, steel and brick work were also gathered. Photographs of damaged buildings was also taken. Some lacking information were noted out by consulting with the owner and builders, when possible. After that the failure of structural and non-structural elements analysed. Engineering judgement was used to quantify the severity of damage. Finally the possible causes of damage were pointed out.

General information and surroundings of building, information of building in plan and in elevation,structural elements, dimensions, reinforcement and infill walls thickness, material characteristics and damages of building were collected from building damage sites.

There are some criteria for assessing the building which is based on the buildings codes and general fundamental principle of structural engineering. For the tilting of the building the code has specified maximum permissibe limit of displacement is 0.04 times total height of building. In case of settlement, 25 mm of settlement is tolerable. For the different criteial of damages study used the FEMA 356[18], ATC 40[19], Eurocode 8: Design of structures for earthquake resistance Part 1: General rules, seismic actions and rules for buildings, European Macroseismic Scale 1998[21].

III. SINGLE PARAMETER ANALYSIS RESULTS

During the rapid visual damage assessment, obseravations of damaged buildings and possible causes of failures are studied. From the building sites, some of available data were collected. Many moment resisting framed buildings have been surveyed in Kathmandu valley, however in the study, only short listed buildings are listed based on availability of data. The analyses of data have been done based on this data and following results are obtained.

3.1 General Information and Surroundings

3.1.1 Coordinate

The Kathmandu valley is very close to epicenters and soil amplification is predominant here due to thick layer of clay deposit on it. Hence, many damages to the structure have been observed in this earthquake.

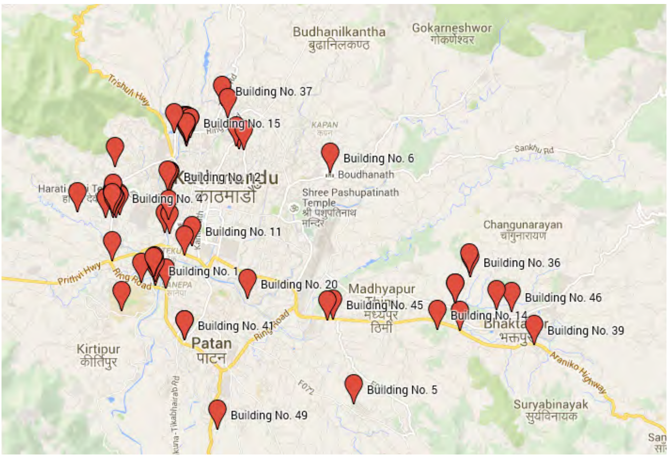

Fig. 3.1.a - Location of assessed building in google map

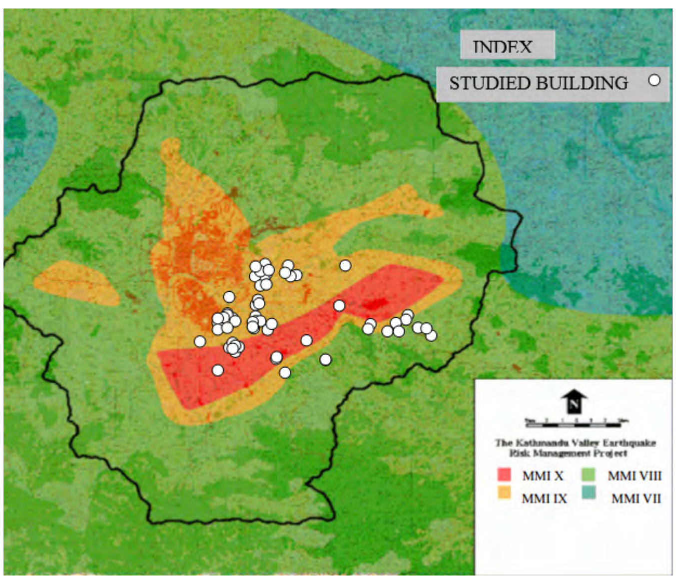

Fig. 3.1.b - Kathamandu valley intensity map with damaged building location. *Source: UNDP/UNCHS Habitat(intensity map)*

For the research work the damaged buildings located in Kathmandu valley were surveyed. Fig. 3.1.a is the plot of those damaged building in google map. The plot shows that the damaged buildings are scattered all over the three cities, Kathmandu, Lalitpur and Bhaktpur.

The probable level of earthquake shaking that the building may face is determined by identifying the location of the building in the seismic hazard map. For this, the available earthquake intensity distribution map of Kathmandu valley developed by UNDP/UNCHS(Habitat)1994 ,"Seismic Hazard Mapping and Risk Assessment for Nepal" based on the intensity distribution of 1934 earthquake of Nepal. Comparing the location of damaged building of Fig. 3.1.a in hazard map of Fig. 3.1.b, one can depicts the intensity of IX MMI (Modified Mercalli Intensity) [4].

3.1.2 Positioning and Ground Condition

Fig.3.1.c shows the categorization of building according to position among the buildings. The results shows that position of building plays vital role relating to vulnerability as 8% buildings which are located in mid are damaged by earthquake out of 64 studied buildings. Isolated buildings were damaged more which reached 66% whereas 26% buildings which are in corner were found damaged. Mid buildings were damaged lesser due to the support in both direction.

In the geographic terrain building lies in not only in plane but also in sloppy area. So, from the result of graph chart in Fig. 3.1.d, it is found that out of studied total 64 numbers buildings, 83% were constructed in plane area whereas 17 % in sloppy ground area. Most of the buildings which were constructed in slope area were severely damaged due to unequal settlement of foundations.

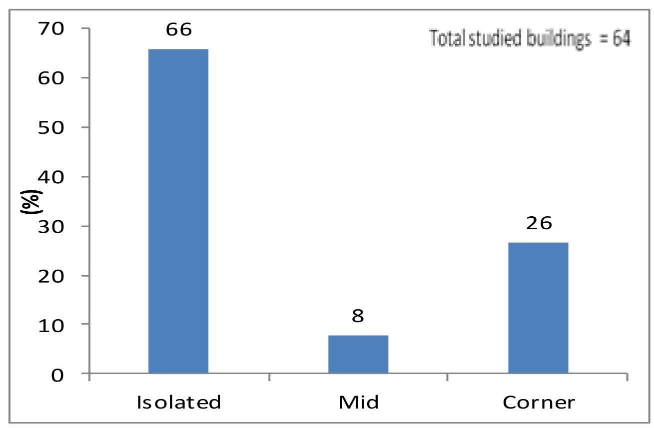

Fig. 3.1.c - Position of buildings

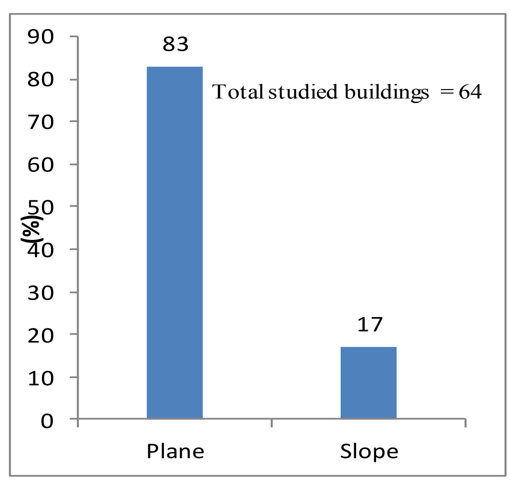

Fig. 3.1.d - Classification of buildings according to surrounding terrain

3.1.3 Number of Stories and Basement

Some information was collected by just looking the building from outside and the results are plotted from available data. Fig. 3.1.e shows the scatterness of the building in Kathmandu valley according to number of stories. The result indicates that most of the buildings surveyed are four (including three and half) and five stories (including four and half) which occurred damage in earthquake. Two and half and three storied buildings are considered as three storied in chart and they are 14% numbers building among the considered 64 buildings. There are 11 % of buildings of six stories which were damaged by earthquake. There are less numbers of higher storied buildings.

In Kathmandu valley out of total studied 64 buildings, 31 % of buildings are with basement and 69% of buildings are without basement. Basements are being used as vehicle parking and storage purpose.

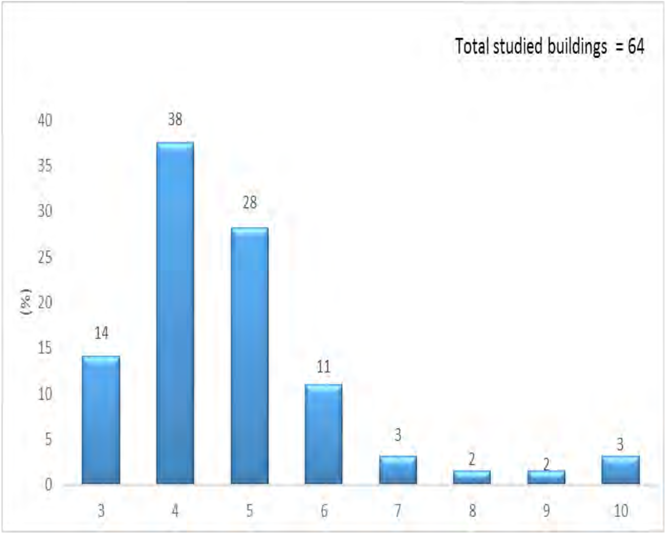

Fig.3.1.e - Buildings damage distribution according to number of stories

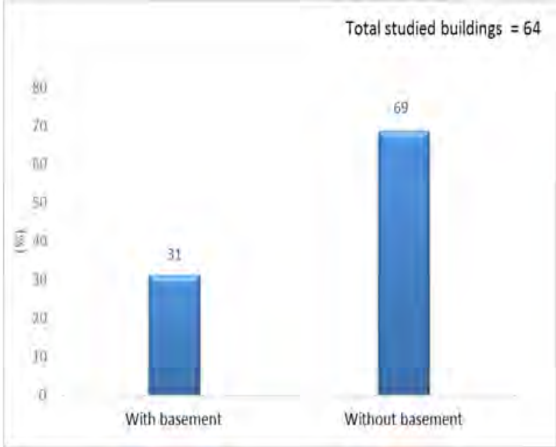

Fig. 3.1.f - Status of basement (building with and without basement)

3.1.4 Height with respect to Adjacent Buildings and was Building Structurally Designed

The floor heights of building with respect to adjoining buildings are presented in Fig.3.1.g. The result shows that about 58% of buildings are isolated, 30% of buildings are in same level and 12% of buildings have level difference (either higher or smaller). It indicates that ponding effect is less likely in the surveyed area.

It is difficult to point out that the building was structurally designed from external outlook. Out of studied 64 buildings, 78% (50 numbers) of buildings are not known whether they were designed or not. They were almost not designed structurally. It is found that only 19% buildings are clearly known that they were structurally designed (see Fig. 3.1.h).

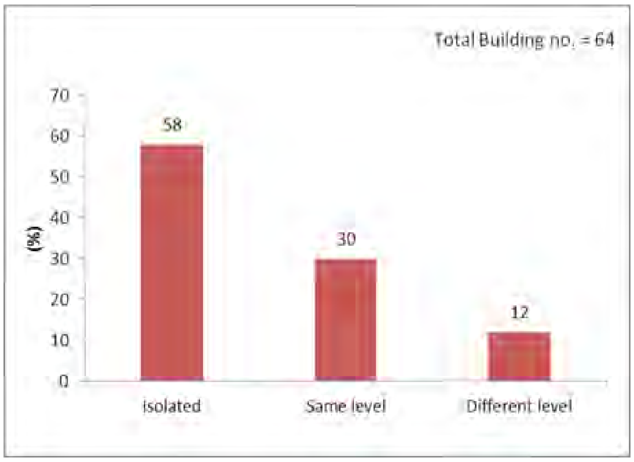

Fig. 3.1.g -Height of building with respect to adjoining buildings

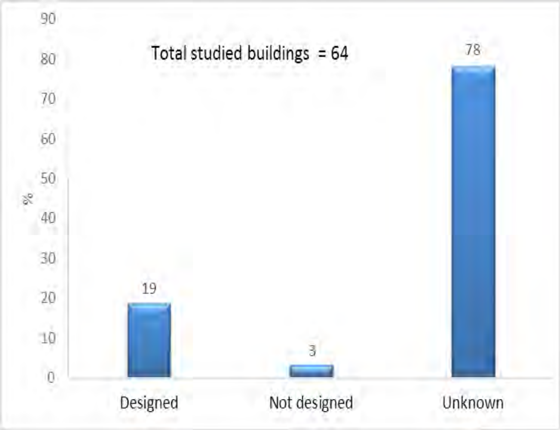

Fig. 3.1.h - Structurally designed or not designed building

3.2 Information in Elevation

3.2.1 Total Height and Regularity in Elevation

In the surveyed building of Kathmandu valley most of the damaged building height lies in between 9 -15 m tall (two and half to four and half or five stories) (see Fig. 3.2.a). This is because of these height buildings are available in city more.

The visual inspection of the buildings shows that in the elevation of building seems to be regular. Out of considered 63 buildings 83% buildings have almost regular floor area and mass. This means most of the buildings are almost same area in different stories. In this case it is considered that staircase cover floor is not considered in the regularity of floor area and mass.

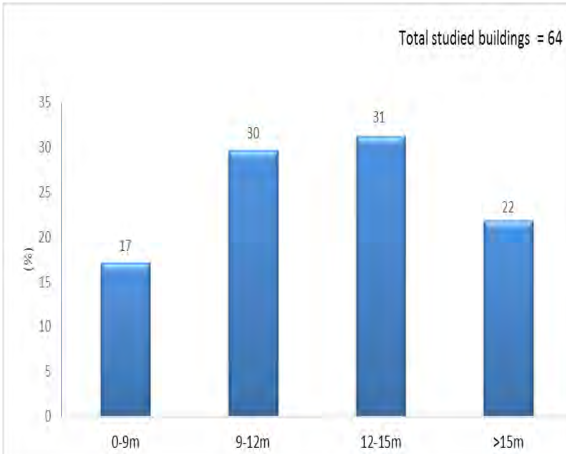

Fig. 3.2.a - Predominant height of building

Fig. 3.2.b - Regularity of building in elevation

3.3 Information in Plan

3.3.1 Regularity of Plan and Number of Bays in both Principle Directions

From the visual inspection, the data were collected to determine the regularity of building plan. In Fig. 3.3.a shows that 53% building have almost regular plan (almost rectangular) and 47% building have plan irregularity (other than rectangular)

Fig. 3.3.b shows number of bays in both major direction in the surveyed buildings. In the survey it is considered that along the road is x-direction and perpendicular to road is considered as y-direction. In the one bay building in x directions are used for shopping and residence purpose where land is expensive. The results show that two and three bay buildings are predominant in both directions. These types of bays are used mainly in common buildings. In case of the bay numbers more than four, bays along x – direction(along the road) is more than y – direction in the comparatively cheap land keeping more space in back side and wider along the road for the purpose of shopping .

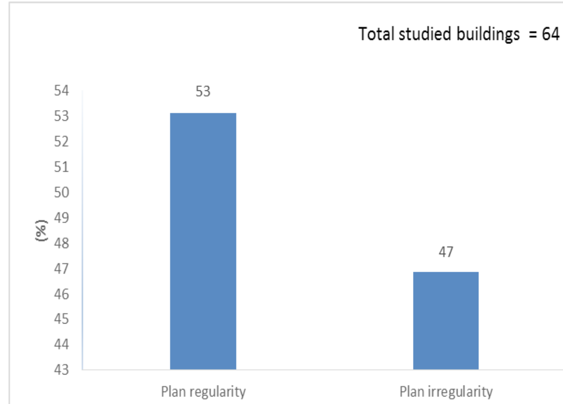

Fig. 3.3.a - Regularity of building in plan

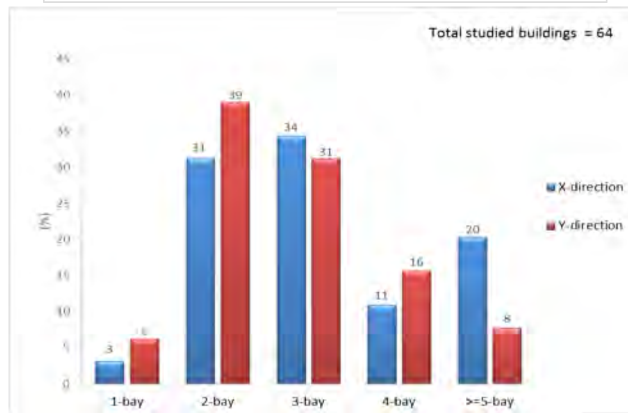

Fig. 3.3.b- Numbers of bays in both direction

3.3.2 Equal Span of Bays in Both Major Direction and Regular Distribution of Infill Walls

Fig. 3.3.c shows large portion of surveyed building of 79% and 71% have almost equal span in x - and y - directions respectively. Spans are considered equal if differences of spans are not more than 1.5 m. It shows that drastically differences in span is not in the most of the building which is the positive part from the point of structural view.

The distribution of infill wall is not regular as shown in Fig. 3.3.d. This unequal distribution of infill wall increases unequal distribution of stiffness which cause torsion in the building during earthquake.

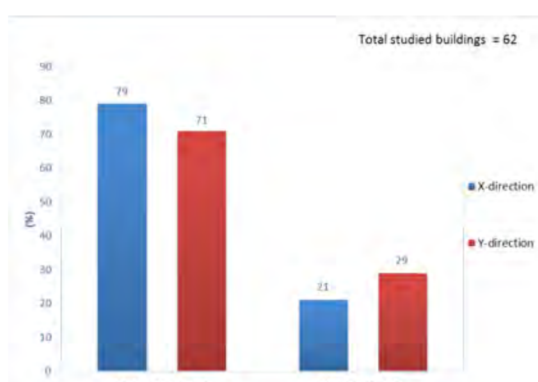

Fig. 3.3.c - Equal span of bays in both direction

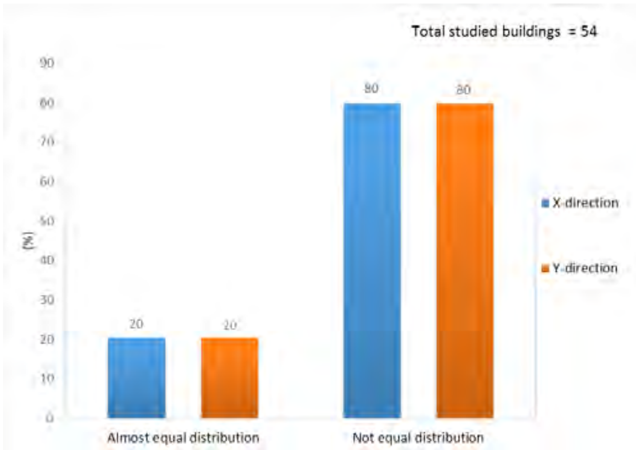

Fig. 3.3.d - Distribution of infill walls

3.4 Structural Elements, Dimensions, Reinforcement and Infill Walls Thickness

3.4.1 Longitudinal Reinforcement in Column

Longitudinal reinforcement is one of the important parameter that signify the vertical load and bi-axial bending moment capacity of column. Fig. 3.4.a shows 15% buildings have less than 1% reinforcement. The highest numbers of buildings of 39% have longitudinal reinforcement of equal and greater than 1% and less than 1.5% followed by 24% of building have reinforcement of equal and greater than 1.5% and less than 2%. Similarly, about 17% of the buildings have longitudinal reinforcement equal and greater than 2% and less than 2.5%. Only 4% of buildings have reinforcement >4%. There are no buildings found which have reinforcement from 2.5-4%. Actual reinforcements in columns are less than required even though in chart it shows higher value because sizes of columns sections are quit smaller than required otherwise it would be lesser than that.

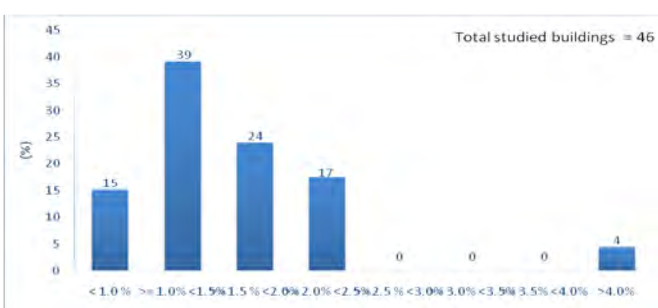

Fig. 3.4.a - Longitudinal reinforcement percentage in columns

3.4.2 Shear Reinforcement Spacing in Column

Transverse reinforcement in column are not visible in most of the cases, however most of the building(58% out of considered 45 buildings) have spacing of 150 mm with diameter of 7 and 8 mm at near the support of column (see Fig. 3.4.b) even though in approximate calculation it needs 100 mm center to center but it is seen that only 7% buildings have this amount of reinforcement. Diameter of transverse reinforcement is mainly used of 7 mm diameter and 8 mm diameter. Fig. 3.4.c shows that out of considered 25 buildings, at the mid height 52% building has 150 mm center to center transverse reinforcements. From this two charts, it can be seen that at mid height of column, shear reinforcements are quite good but near the support of columns, they are not enough.

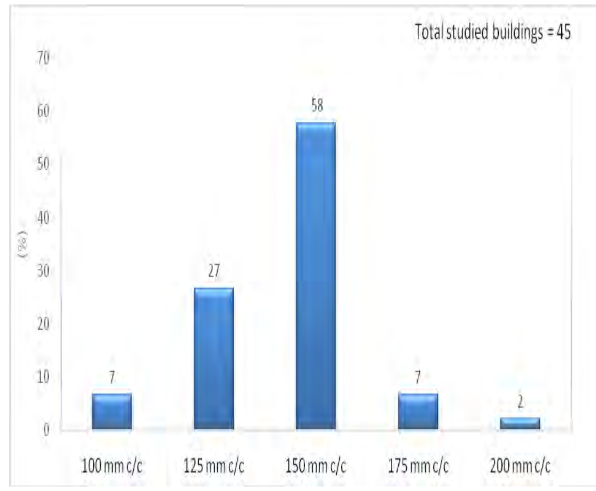

Fig. 3.4.b - Stirrups spacing at support of columns

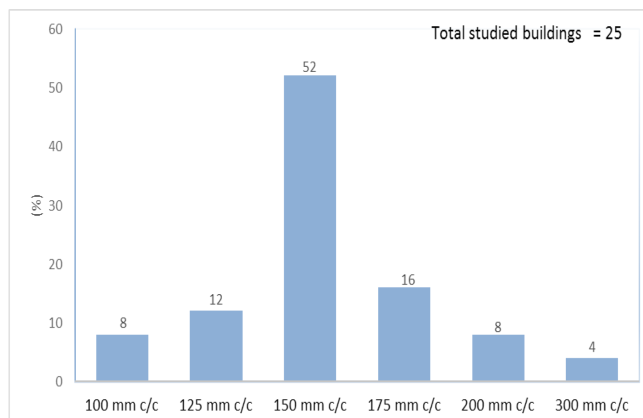

Fig. 3.4.c - Stirrups spacing at mid of columns

3.5 Material Characteristics

Quality of material also influence the strength of the structural members and consequently to structural system itself. Fig 3.5.a shows that out of studied 64 numbers of buildings, 61% of buildings have poor quality of concrete. Similarly, Fig. 3.5.b shows that the quality of concrete in vibration shows 52% buildings have poor quality. This is the cause for the crushing of concrete in the column support and joint. The results show that TMT steel were used mostly in the buildings which are more vulnerable in compared to Tor steel used buildings (see Fig. 3.5.c). This might be because TMT have less ductility than Tor steel.

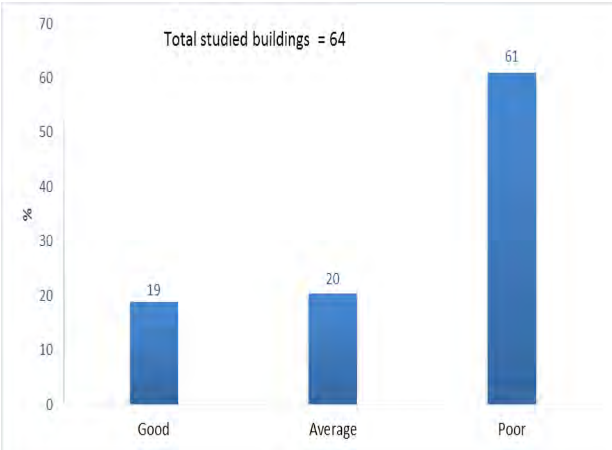

Fig. 3.5.a - Quality of concrete

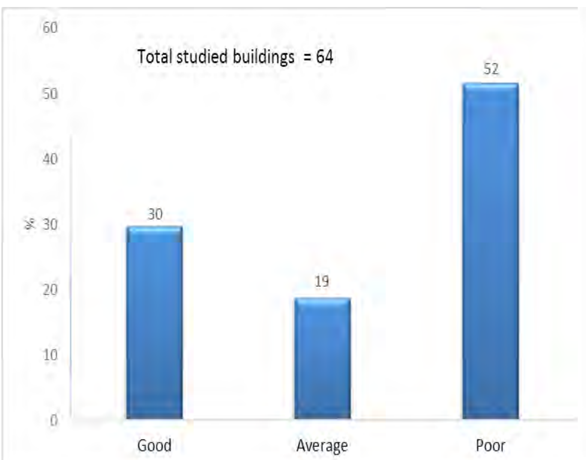

Fig. 3.5.b - Quality of concrete in vibration

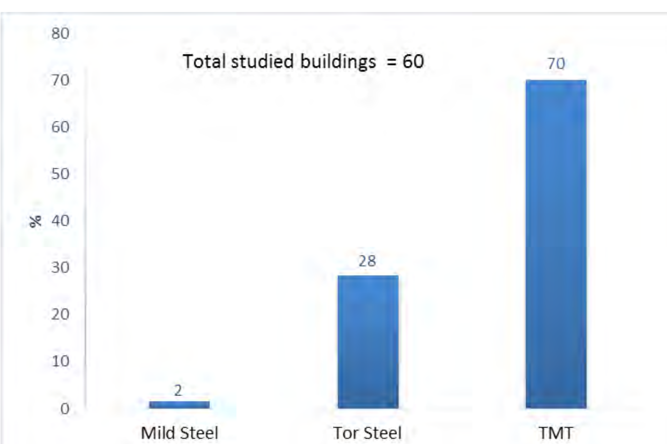

Fig. 3.5.c - Buildings with different types of reinforcements

3.6 Damages

3.6.1 Damage Grades

Damage of the building can also be classified according to European Micro-seismic Scale (EMS 98). Fig. 3.6.a shows that the heightest percentage of buildings of 38 % lies in damage grade G5 followed by G4, G2, G3 and G1 respectively.

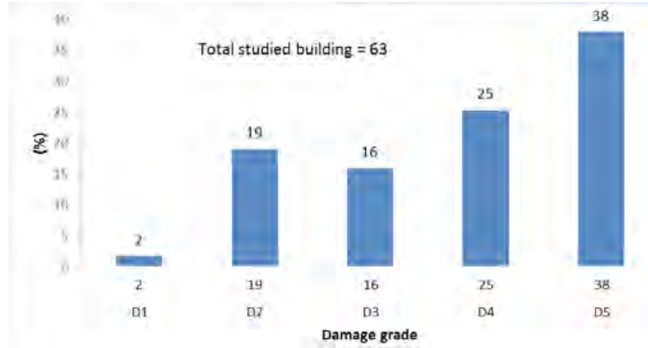

Fig. 3.6.a - Classification of building based on damage grade

3.6.2 Soft Story Evidence and Settlement of Building

The analysis of data collected from field survey shows that soft story failure evidence is observed in 41 percent of building. Fully collapse of ground floor, most of the damages of ground floor columns with respect to above stories is considered as a soft story evidence.

Due to very weak soil deposit area, soil amplification, settlement and liquefaction are most probable in this zone. The graph in Fig. 3.6.b shows out of studied 64 buildings, 14% building are settled. It happens due to foundation in weak soil, not enough size of foundations, heavy load in small foundations, unequal level of foundation footings, different soil strata in different footing, high water table and other causes.

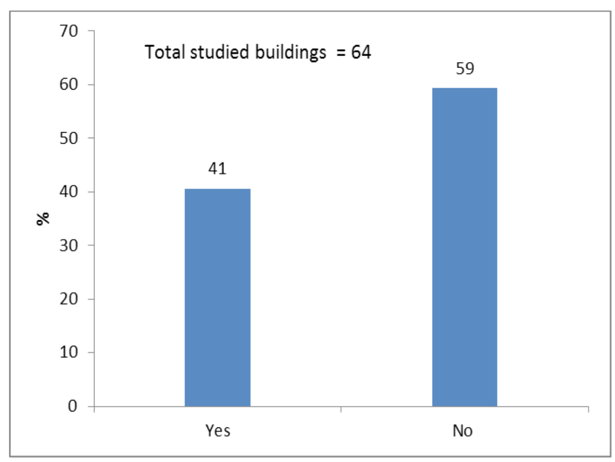

Fig. 3.6.b - Classification of building according to soft story

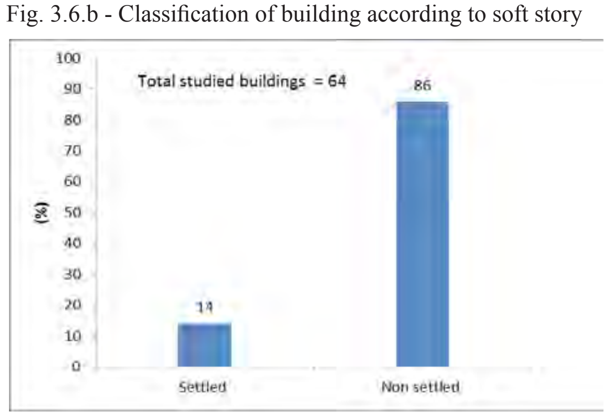

Fig. 3.6.c - Classification of buildings according to settlement

3.6.3 Tilting of Building and Damage of Structural Elements

It is believed that Kathmandu valley have high soil deposited of up to about 600 m (source: slide of B. N. Upreti). Tilting of building is observed in some of the building. It is observed that 19% of buildings from the 64 studied buildings seem tilted (see Fig. 3.6.d). It is due to weak soil, very slender building, unequal mass in plan is and in elevation.

Damage of the structural elements such as column, beam, joint, slab-stair and infill wall are plotted in Fig. 3.6.e. The results show that the most probable damage are observed in column and infill wall followed by slab-stair and joint. The least damage was observed in beam, this might be because of strong beam weak column mechanism.

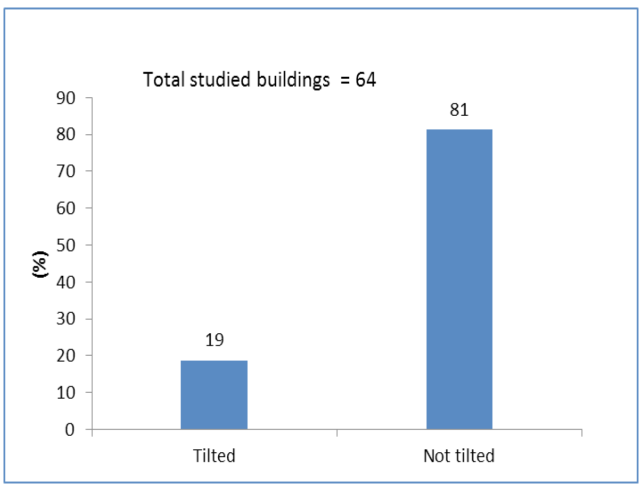

Fig. 3.6.d - Tilting of buildings

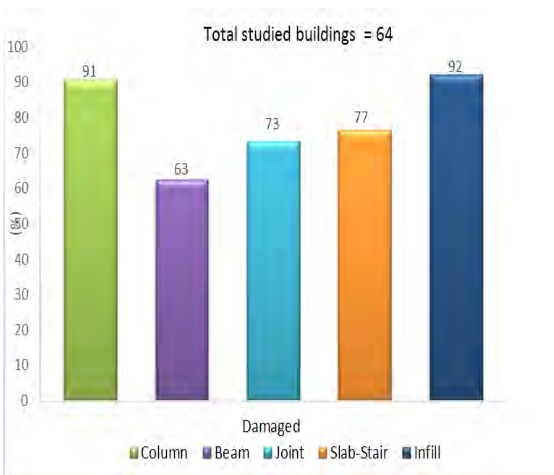

Fig.3.6.e - Damage of structural elements

IV. CORRELATION BETWEEN PARAMETERS UNDER ANALYSIS

In the above section 3, it is analyzed distribution of buildings according to single parameters. In this section, the correlation between two or more parameters will be dealt. The damages of structural elements such as column, beam, beam-column joint and slab/stair and non-structural element such as infill wall with respect to other parameters are discussed briefly. The damage grade of the buildings are also studied with respect to other parameters.

4.1 Damages of Building Components According to Damage Grade

Out of studied 64 buildings, 12 buildings are designed, 52 buildisngs were not confirmed, so they are catagorised as unknown. Fig.4.1.a shows that structural elements of undesigned buildings were more damaged than designed one. The non-structural element like infill walls were equally damaged for both.

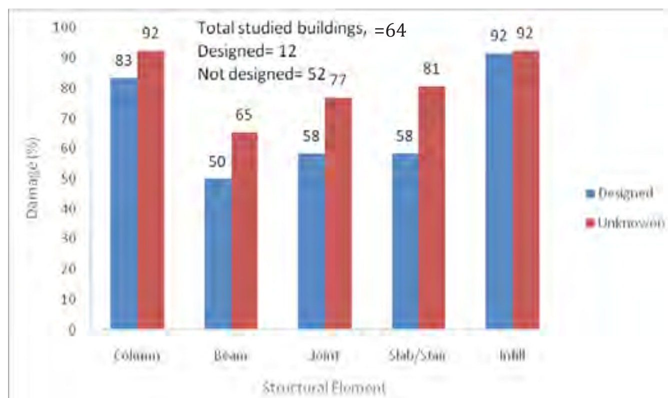

Fig. 4.1.a. - Damage of structural elements with respect to designed and unknown to designed buildings

4.2 Damages of Building Componests and Damage Grade with Respect to Material Characteristics

Quality of concrete highly influences the damage of structural and non-structural component. Fig. 4.2.a shows that significant damages are observed in column, beam, joint, slab/stair and infill wall of poorly constructed buildings.

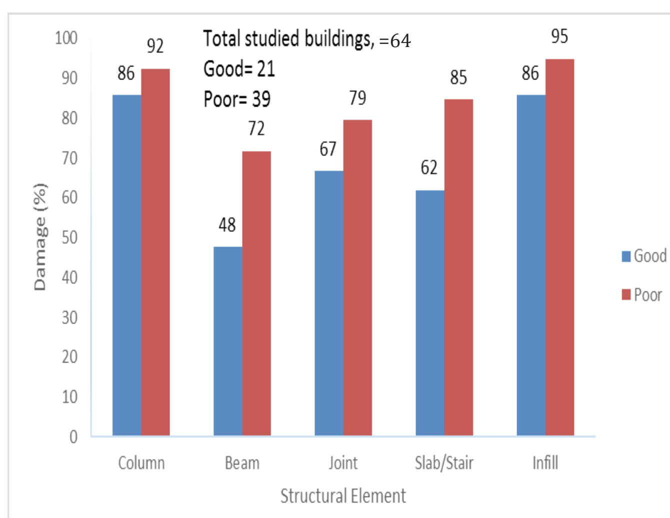

Fig. 4.2.a - Damage of buildings with respect to quality of concrete

4.3 Damage Grade of Buildings with respect of its Components

Severity of damage of the buildings increases with increase of damage grade. Fig. 4.3a shows that degree of damage of components of buildings increases with increase of damage grade.Beam failure is more than column failure in case of damage grade 4 and 5 which is oppose to damage grade 2 and 3.

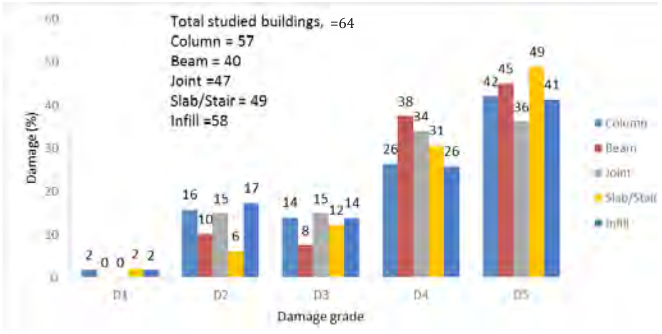

Fig. 4.3.a - Damage grade of buildings with respct to damage of its component

V. CONCLUSION AND RECOMMEDATION

- The surveyed buildings for damage assessment of the buildings lie in IX MMI comparing with the seismic hazard map of Kathmandu valley prepared by UNDP. Most of the damaged buildings are found to be isolated of 4 story(including three and half) and 5 storey(including four and half). About 30 percent of the buildings have basement and about only 19 percent of the buildings are properly de-signed.
- Regarding information in elevation, the predominant height of the buildings are found in the range of 12 – 15 m and 83 percent of the buildings are regular with respect to floor area and mass.
- About 53 percent of the building are almost regular in plan. Most of the buildings have 2-3 bays in with equal span along both major direction. On the contrary, 80 percent of the buildings do not have equal distribution of infill wall.
- Regarding to longitudinal reinforcement in column, the highest 39 percent of the buildings have rein-forcement ranges in 1%-1.5%. There is no buildings found which have reinforcement form 2.5-4%. Most of the building have stirrups spacing of 150 mm with diameter of 7 and 8 mm diameter at mid and support of column. Shear reinforcement in columns are lacking as per norm.
- In the surveyed buildings, the quality of concrete in 61 percent is poor and it is seen that 52 percent buildings have poor in concrete vibration. In case of the steel used in the buildings, 70% have TMT steel(comparatively less ductile) and only 2% have Mild steel.
- The highest percentage of 38% of the buildings are assessed as Grade 5, 25 % Grade 5, 16% Grade 4 and 19 % Grade 3. Only 2% are in Grade 1. 41% of the buildings have soft storey, 14% of the buil-dings are damaged due to settlement and 19 % buildings are found to be tilted. About 90% of the co-lumns and infill walls, about 75% of joints and slab-stair and about 60% of the beams are observed to be damaged.

Due to high seismicity of Kathmandu valley it is recommended to introduce general principle of earthquake resistant elements such as (bands, stitches) in the building. The geometry of the building should be symmetri-cal from both major direction and there must be equal distribution of infill wall in both direction. The flexural and shear reinforcement provided in column is not sufficient, hence capacity design with strong column weak beam principle is recommended. The workmanship of the concrete work is not satisfactory, so it is better to use concrete mixer and vibrator instead of manual mixing and compacting. Regarding the reinforcement it is better to use Tor steel rather than TMT or TMT with sufficient ductility . It is recommended to avoid soft sto-ry, short column in the buildings and soil test is recommended for any construction to avoid settlement and til-ting. To prevent brittle failure, ductile detailing should be followed as per the norms. In addition, enforcement of codes should be strict and built the structure only after proper analysis and design.

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Lessons Learnt From Gorkha- Earthquakes: Observation and Structural Detailing of RC Building

Er. Krishna Singh BASNET
Senior Structural Engineer
krishnasinghbasnet@gmail.com

ABSTRACT

The Gorkha earthquake and its aftershocks caused extensive damage to building structures in Nepal. A RAPID visual assessment survey was carried out to assess the damaged buildings. It was found that many old as well as modern reinforced concrete residential, hospital, school and high-rise apartment buildings sustained minor to major damage and collapse. Most of the well-designed reinforced concrete buildings sustained minor or non-structural damage. As per the field observation, most of the building structures are found to have been damaged due to improper use of building codes, poor design and construction practices. Accordingly, soft story reinforced concrete buildings were seen collapsed seen and non-structural damage was seen in masonry infill walls.

This paper havs summarized the damage to RC buildings with specific focus on the causes and types of damage due to the earthquakes and educate the owners, contractors, engineers, workers about the earthquake resistant constructions consideration for improving the seismic performance of RC buildings in Nepal. The first and foremost objective of this paper is to sensitized the stakeholders concerned and strictly implement the building codes for the seismic design of any infrastructure in Nepal.

Key Words: *Gorkha earthquake, RC buildings, structural concept, proportioning and detailing*

I. INTRODUCTION

The Gorkha earthquake Mw7.8 occurred at 11:56 NST on 25 April 2015 with an epicentre 77 km ,48 miles northwest of Kathmandu, the capital city and at a focal depth of approximately 10-15 km. This earthquake was the one of the most powerful earthquakes to strike Nepal since the 1934. The earthquake mainly resulted in about loss of more than 9000 people and collapsed and heavy damaged historic, public and residential buildings structures. From the Gorkha earthquake the performance of building structures observations has educating proper and improper construction of earthquake load resisting systems. As a structural engineer, observations will also draw upon a broad data base of engineering that have to report systematically.

1.1 Construction Practice in Nepal

Reinforced concrete framed structures are a very popular construction practice in Nepal (JICA, 2002). Low-rise reinforced concrete buildings are designed using the seismic design code NBC-105 (DUDBC, 1994) whereas high-rise buildings are designed following the guidelines provided by the Indian seismic code IS1893 (BIS, 2002). Detailing for ductility is based on Indian standard IS 13920 (BIS, 1993). Although various guidelines are prevailing in the area of seismic design of buildings in Nepal, most of the low-rise structures do not follow the design codes. Owners themselves compromise with the quality of construction materials, design and construction process for economic reasons. The construction practice in most of the cases is based on thumb rules and the masons are not well trained. Due to the increasing urbanization in the Kathmandu valley, the owners opted for addition of stories one after another in old constructions, which were far from well designed. These are some of the weaknesses of the current building design and construction practices in Nepal. Chaulagain et al. (2013) studied the seismic vulnerability of common RC buildings in Nepal and found that the buildings constructed using current construction practice and designed in accordance with Nepal Building Codes (NBC) are highly vulnerable to earthquakes.

1.2 Observation Damages RC Building

Reinforced concrete frame buildings performed poorly in the Gorkha earthquakes. Many low-rise reinforced concrete buildings were either collapsed or sustained extensive damage. Most of the poorly designed buildings not confirming to design standards suffered severe damage. But well designed reinforced concrete buildings suffered only minor non-structural damage. The damaged RC buildings are mostly residential buildings as well as school and high-rise apartment buildings. Most of the high-rise apartment buildings were subjected only to damage in the non-structural elements and are livable after repairing.

All the reinforced concrete framed constructions in Nepal have heavy brick masonry infill walls increasing the seismic weight of the buildings. Many poorly designed residential buildings collapsed or suffered heavy damage due to the earthquakes. The photos showings the damaged RC buildings are as follows;

Photo 1: Collapsed RC frame building in Sitapaila

Photo 2: Soft story failure (lost their lower 2 and 4 stories) of buildings in Gongabu

Photo 3: Pounding between two building, Bhurungkhel

Photo 4: Collapse of a 3 storied RC (poor ductile detailing) Building

Photo 5: Damage to the first story of the 3 storied building

More than 8,000 ***school buildings*** were destroyed due to the earthquake and its aftershocks. Fortunately, the schools were closed on the day when main shock and its major aftershocks occurred. Temporary shelters have been constructed to conduct classes in the Kathmandu valley

Photo 6: (a) Collapsed building (b) Shear failure of columns.

High-rise apartment buildings in Kathmandu valley suffered mostly non-structural damage. Only minor structural damage was observed in buildings.

Photo 8: Diagonal cracks in the masonry infill walls

Photo 9: Diagonal shear failure of a beam

II THE STRUCTURAL CONCEPT

It has been observed that proper selection of load carrying system for structure is not properly compiled with on the damaged buildings. Proper selection of the load carrying system for structure is essential for good performance under any loading. A properly selected structural system tends to be relatively forgiving of oversights in analysis, proportion, detail, and construction. On the other hand, extra attention to analysis and detail is not likely to improve significantly the performance of a poorly-conceived structural system.

This observation is particularly appropriate in earthquake-resistant design where the intensity and orientation of loading are highly uncertain. Buildings having simple, regular, and compact layouts incorporating a continuous and redundant lateral force resisting system tend to perform well and thus are desirable. Complex structural systems that introduce uncertainties in the analysis and detailing or that rely effectively on-non-redundant load paths can lead to unanticipated and potentially undesirable structural behaviour. An essential characteristic of any lateral load resisting system is that it must provide a continuous load path to the foundation. Inertial loads that develop due to accelerations of individual elements must be transferred from the individual reactive elements to floor diaphragms, to vertical elements in the lateral load system, to the foundation, and eventually to the ground failure to provide adequate strength and toughness of individual elements in the system or failure to tie individual elements together can result in distress or complete collapse of the system (Bertero, V. V., et. Al).

A lesson learnt from the Gorkha earthquakes was realization that structural and non- structural elements must be adequately tied to the structural system. Numerous examples can be found of detachment of exterior cladding of parapets and of various non- structural elements within buildings. This observation specifically requires that individual elements of a building be adequately tied to the structure. Inertial loads that develop in individual elements must be carried to the vertical elements of the lateral load resisting system by horizontal floor diaphragms. Concrete diaphragms, with appropriate struts, ties, and boundary elements should be provided with adequate reinforcement to transmit these forces.

2.1 Regularity

Sudden changes in stiffness and strength between adjacent stories are commonly observed. A tall building adjacent to a shorter building may experience irregular response due to effects of impact between the two structures. The effect can be exacerbated by local column damage due to pounding of the roof of the small building against the columns of the taller one.

Partial-height frame infills are also common. In this form of construction, an infill extends between columns from the floor level to the bottom of the window line, leaving a relatively short portion of the column exposed in the upper portion of the story. The shear required to develop flexural yield in the effectively shortened column can be substantially higher than that which would develop for flexural yield of the full-length column. If the design has not considered this effect of the infill, shear failure of this so-called 'captive column' can result before flexural yield. This form of distress is a common cause of building damage and collapse during earthquakes. Mass, stiffness, and strength plan irregularities can result in significant torsional response. Inelastic torsional response cannot at present be rectified with results of elastic analysis, and techniques for inelastic analysis of complete building systems considering torsion are largely unavailable and unverified. Control of lateral drift should be a central element of any seismic design. Excess drift can lead to excessive distortion, and thence, damage, to structural and non-structural components. Because repair cost is the primary measure of the success of a building that has survived an earthquake, damage control is essential. Control of damage in non-structural components is important because they typically comprise a significant majority of the total value of a building and because falling non-structural elements can cause injury and death to building inhabitants.

Control of drift is also important because non-ductile or moderately-ductile elements of the structural system can be damaged by excessive distortion. Control of drift by structural walls is a proven means of reducing damage to weak, low-ductile framing. Drift control is important also in preserving the vertical stability of a structural system. If a structural system is excessively flexible, and particularly if it is also massive, collapse can occur due to P-delta effects. Observations Gorkha earthquake demonstrated that this was a particularly prominent problem with flat-slab structures because of their relatively low lateral load stiffness.

2.2 Proximity to Adjacent Buildings

Buildings are constructed in close proximity to one another; damage due to pounding between the buildings is possible. Several examples of building failures due to pounding have been observed in the Gorkha earthquake. Pounding may result in irregular response of buildings of different heights, local damage to columns as the floor of one building collides with columns of another, collapse of damaged floors, and in many cases collapse of entire structures. Damage due to pounding can be minimized by drift control, building separation, or as a last resort, aligning floors in adjacent buildings so that columns do not bear the blows of oncoming floor slabs.

Several examples can be cited where buildings in close proximity apparently supported one- another rather than resulting in damage. In most cases, the buildings are of similar story and total height, of similar stiffness, and located sufficiently close that pounding impacts are of relatively low energy.

2.3 Mass

Excess mass can lead to unnecessary increases in lateral inertial forces, reduced ductility of vertical load resisting elements, and increased propensity toward collapse due to P-delta effects. For these reasons, efforts should be made to achieve a system that is as lightweight as possible. However, concrete as an architectural fill and soil for landscaping on top of structural slabs provide unnecessary mass without structural benefit. Numerous buildings that collapsed due to the presence of excessive vertical loads, more often excessive live loads resulted from change in occupancy of a building. Irregularity of mass distribution in vertical and horizontal planes can result in irregular responses and complex dynamics. These should be avoided where practicable.

2.4 Redundancy

Structural systems that combine several lateral load resisting elements or subsystems generally have been observed to perform well during earthquakes. Redundancy in the structural system permits redistribution of internal forces in the event of failure of key elements. Without capacity for redistribution, global structural collapse can result from failure of individual members or connections. Redundancy can be provided by several means; a dual system, a system of interconnected frames that enable redistribution between frames after yield has initiated in individual frames, and multiple shear walls. Redundancy, combined with adequate strength, stiffness, and continuity, can alleviate the need for excesses in ductile detailing.

III STRUCTURAL PROPORTIONING AND DETAILING

Conventional earthquake resistant design of buildings relies on element ductility to enable redistribution and reduction of internal actions, and dissipation of earthquake energy. Observations have shown repeatedly the necessity of attention to proportioning, to ensure that inelastic action occurs at appropriate locations, and detailing, to ensure adequate ductility in these locations that yield. Some of the more prominent observations and lessons are;

3.1 Locations of Inelastic Deformation

Structures should be proportioned to yield in locations most capable of sustaining inelastic deformations (Leeds, A., ed). In reinforced concrete frame buildings, attempts should be made to minimize yielding in columns because of the difficulty of detailing for ductile response in the presence of high axial loads and because of the possibility that column yielding may result in formation of demanding story sway mechanisms and collapse. Collapse of stories or complete structures is due to column weakness and limited ductility. The problem of yielding in columns rather than beams is particularly pronounced in structures for which gravity load effects control proportions and strengths, resulting in beam flexural strengths exceeding by some margin the column flexural strengths. This situation typically occurs in buildings having long beam spans, and in the upper floors of buildings where design seismic effects are relatively low.

Observations of failures due to yielding in columns have led to formulation of the weak beam- strong column design philosophy in which column strengths are made at least equal to beam strengths. The intended result is columns that form a stiff, unyielding spine over the height of the building with inelastic action limited largely to beams. In buildings where architectural requirements require wide bays with resulting strong girders, the strong column-weak beam design philosophy may be difficult to implement. In such cases, columns should be detailed to sustain inelastic action or, preferably, continuity of deformations over height should be enforced by providing continuous structural walls. Coupled wall systems are generally proportioned so that a considerable portion of the inelastic energy dissipation occurs in the coupling beams. Closely spaced transverse reinforcement or special reinforcement details are recommended.

3.2 Determination of Member Actions

The structure should be proportioned and detailed in a manner that is consistent with the expected inelastic deformation mode. If inelastic flexure is preferred in selected elements, design actions and appropriate proportions should be selected to ensure that the elements can achieve the flexural strengths (Kreger, M. E., and Sozen, M. A). For beams and columns, shear failures have occurred because design shear forces were determined on the basis of design lateral forces rather than the shear required equilibrating the plastic moment capacities of the member. Consequently, most modern codes stipulate that design shears be evaluated on the basis of likely plastic hinge locations with appropriate factors of safety applied to member strengths and transverse loading.

Failures have also apparently occurred because bar cut-offs were inconsistent with the moment distribution that develops when flexural strengths are reached at member ends. Uncertainties in determining these moment distributions have in part motivated most model code writing bodies to recommend continuous nominal reinforcement on both faces of all structural elements. Non-structural components have been observed to substantially alter structural behaviour. For example, slabs on grade can change assumed base fixity conditions, and stairways and partial infill in frames can alter the member actions. These interactions can result in increased member shear demands as well as formation of plastic hinges away from those regions detailed for ductile action on the basis of intended behaviour. Corner columns have statistically greater damage rates than other columns in moment resisting frames. An apparent cause is the combined effect of actions from perimeter frames oriented perpendicular to one another and connecting at the corner column. Extra attention appears warranted in selection of design actions considering orthogonal effects and in detailing and construction

3.3 Transverse Reinforcement

Generous supply and appropriate placement of transverse reinforcement in reinforced concrete beams, columns, beam-column connections, and walls have proven to be desirable. Such reinforcement is useful for concrete confinement, resistance to shear, restraint of longitudinal reinforcement buckling, and improved anchorage. Failure to provide transverse reinforcement at member ends where plastic hinges are anticipated results in reduced flexural strength and ductility, as well as degradation of shear resistance. Closely spaced transverse reinforcement is particularly recommended for the unrestrained length of captive columns where inelastic flexure is combined with high shear force. Boundary elements of walls where significant inelastic action is anticipated should be well confined to provide ductility under axial compression (Mahin, S. A., et al). Columns supporting discontinuous walls should be confined over their entire height.

Distress in beam-column joints, in some cases leading to building collapse, has been attributed to inadequate joint confinement. Several disastrous failures during the Gorkha earthquake could apparently be attributed to joint failure in cases where heavy spiral or rectilinear confinement in columns above and below a joint was discontinued at the joint. In general, confinement in columns should continue through the connection region. Effective concrete confinement can be obtained by using either spiral or rectilinear reinforcement. To be effective, transverse reinforcement must be coupled with well-distributed longitudinal reinforcement.

3.4 Anchorage and Connections

Strength and toughness must be developed not only within members themselves but also in the connections between members. Photo 4 can be found where beam-column connections with inadequate transverse reinforcement failed during the Gorkha earthquakes.

Slab-column connections have suffered distress in the Gorkha earthquakes, and in several cases have contributed to collapse. In, the presence of heavy vertical loads (Photo 4) is believed to have resulted in excessive shear stress on connections, with resultant decrease in connection moment capacity and ductility and increase in P-delta moments. Combined with the relatively large flexibility of slab-column frames, several collapses resulted. In the event of punching failure at a connection, bottom slab reinforcement anchored through the columns has been observed to be an effective means of preventing or delaying collapse; lack of such reinforcement has been observed to result in catastrophic failures. Continuity between members and between members and joints is also essential. In the Gorkha earthquake, several collapsed buildings were found where columns had not been adequately interconnected through the joints by continuous longitudinal reinforcement. Failures have also been observed where reinforcement splices within members were of insufficient length or were inadequately located. These observations point to the general need of providing continuous ties within the structure, and the specific need of considering distributions of internal member actions that can occur under severe seismic loading.

Proper anchorage of reinforcement under the action of cyclic inelastic load reversals requires that the reinforcement to be developed have adequate transverse reinforcement and concrete surrounding each bar. In particular, inadequate bond of bundled column reinforcement is believed to have been the cause of damage to and collapse of several buildings in earthquake.

3.5 Construction

Observations of earthquake performance demonstrate clearly and repeatedly the need for punctilious attention to design, detail, and construction. In particular, structures that rely on ductile response and that do not provide multiple load paths to the foundation require dedicated, professional inspection to ensure that required ductile details are properly implemented. The designer must ensure that construction drawings and documents are clear and unambiguous, and that actual conditions of construction do not interfere with the behaviour intended in design.

Improper anchorage of transverse reinforcement has resulted in failure of confinement in columns, improperly executed construction joints in shear walls have resulted in movement and damage along the joints (Zeris, C., Mahin, S. A., and Bertero, V. V).

IV. CONCLUSIONS

The Gorkha earthquake of 25 April 2015 reflected various types of technical deficiencies in the buildings in local and urban areas. The majority fraction of damage is found to be consisted by URM, rubble stone and adobe buildings at the local level. The common types of failures in RC construction were identified as the soft storey, pounding, shear failure, and other failures associated with construction as well as structural deficiencies like building symmetry, detailing and others.

Furthermore, behavioural aspects of many forms of inelastic response (for example, joint deformations, failures in shear and anchorage, severe discontinuities, and three-dimensional inelastic response including torsion) at present cannot be modelled confidently even when identified. The influence of such behaviours should be minimized through layout of the structural system, and proportion and detail of its components. Simple design procedures can clear responses involving the uncertain modes of failure.

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Impacts of Nepal Earthquake Swarm of April and May 2015 on Constructed Facilities of Kathmandu Valley

Debasis ROY
Professor, Department of Civil Engineering, Indian Institute of Technology, Kharagpur, WB 721302, India
debasis@civil.iitkgp.ernet.in

Alpa SHETH
Managing Director, VMS Consultants Private Limited, Mumbai 400021, India

ABSTRACT

East central Nepal was hit by an earthquake swarm comprised of more than 280 events larger than moment magnitude (M_w) 4 since the M_w 7.8 event of April 25, 2015 located off Lamjung. Five other earthquakes of the swarm were larger than M_w 6. Facilities built along rivers and drainage courses in Kathmandu valley suffered intense damages. Buildings also suffered damages. Damages due to permanent horizontal ground deformation were more intense in the east-west direction, whereas short-period structures were mainly affected by the north-south ground motion. Unconfined traditional masonry structures suffered heavy damages. Structural damages suffered by reinforced cement concrete structures were relatively small. These observations strongly indicated a need for adoption of a displacement-based structural design strategy. This paper summarizes the observations related to deep alluvium sites of the Kathmandu Valley obtained during the visit.

I. INTRODUCTION

East central Nepal was hit by an earthquake swarm comprised of more than 280 events larger than magnitude (M_w) 4 since the M_w 7.8 event of April 25, 2015 located off Lamjung (Figure 1: the event marked "A"). Five other earthquakes of the swarm were larger than M_w 6: the M_w 7.3 Kodari earthquake of May 12; M_w 6.7 event also located near Kodari of April 26; M_w 6.6 event in the Lamjung area of April 26; M_w 6.3 event off Ramchhap of May 12; and M_w 6.1 event off Banepa of May 12. The epicentres of these events are marked in Figure 1 as well using letters "B" through "E," respectively. Location of epicentres of earthquakes larger than magnitude 6 in and around Nepal that occurred since 1915 clearly indicates that the earthquake swarm of 2015 was very much confined within the seismic gap of eastern Nepal. These earthquakes, the M_w 7.8 and M_w 7.3 events of April 25 and May 12, 2015 in particular, led to the destruction of more than half a million homes and in excess of 8800 casualties as per Nepal Disaster Risk Reduction Portal, drrportal.gov.np.

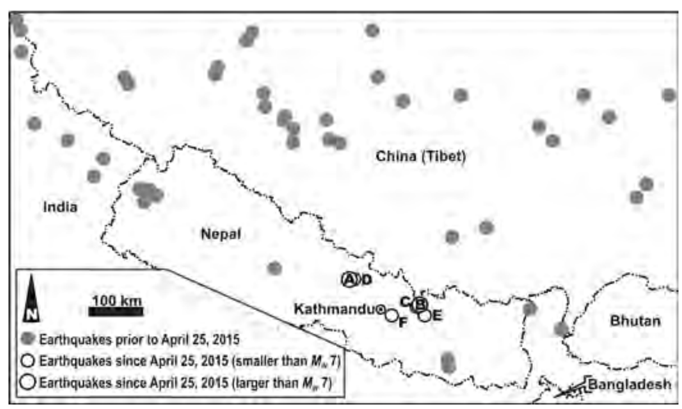

Figure 1. *Earthquakes of magnitude 6 and larger in Nepal and its vicinity since 1915 (data source: http://earthquake.usgs.gov)*

In Kathmandu, located at a distance of about 60 km from the epicentre of the M_w 7.8 event of April 25, 2015, the peak horizontal ground acceleration, PHGA, was 0.25*g* on a rock outcrop at a recorder station at Kirtipur Municipality Office and 0.16*g* at a deep alluvium site at Kantipath (Takai et al. 2016) during the event. There was a remarkable difference in the spectral signatures of Kantipath accelerograms in two horizontal directions; north-south peak spectral response was at 0.5 s, while that for east-west was at 5 s. The PHGA, for the M_w 7.3 event at Kantipath, Kathmandu was 0.09*g* with intense motion observed at periods of up to about 3 seconds. The epicentral distance for Kathmandu for the event was 75 km.

The authors were members of a reconnaissance team invited by Nepal Engineers' Association, Kathmandu that did an extensive damage survey of structures and built facilities in and around Kathmandu and visited the epicentral area of the M_w 7.3 event of May 12, 2015. The main objective of the survey was to assess the health of buildings and structures particularly those hosting telecommunication towers. Impacts of potential future underperformance of these structures on their surroundings were also assessed. This paper summarizes the observations related to deep alluvium sites of the Kathmandu Valley obtained during the visit.

II. EARTHQUAKE-RELATED DAMAGES IN THE KATHMANDU VALLEY

Observations made at various locations in Kathmandu valley are summarized in Figure 2. The valley is largely underlain by firm to stiff clayey silt and clay of the Pleistocene age although at some locations Holocene sand and silty sand, colluviums, residual soils and sand and silt of Pleistocene terrace deposits are found (Figure 3). Observation in Kathmandu valley consistently indicated that damage intensity was less severe compared to that observed at comparable epicentral distances during the M_w 6.9 Sikkim earthquake of 2011. Damping offered by the alluviums of Kathmandu valley appears to have led reinforced concrete structures – and even unreinforced masonry structures at some locations – to perform reasonably. However, damage intensity and frequency generally became more severe near valley margins possibly because of valley edge effects or topographic amplification. The damages near the valley trough – the area around Kathmandu Durbar Square – were often associated with permanent ground deformation arising, for instance, from differential settlement in areas with low relief (Figure 4) and down slope deformation of soils at locations with a steep terrain or earthquake related earth support system failure (Figure 5). While structures supported on isolated shallow foundations suffered intense damages (e.g., Figures 4, 5b and 6a), modern structures on combined footings or mats (e.g., Blue Diamond Society office at Dhunbarahi Marg) appeared to have suffered less even in the presence of permanent ground deformations.

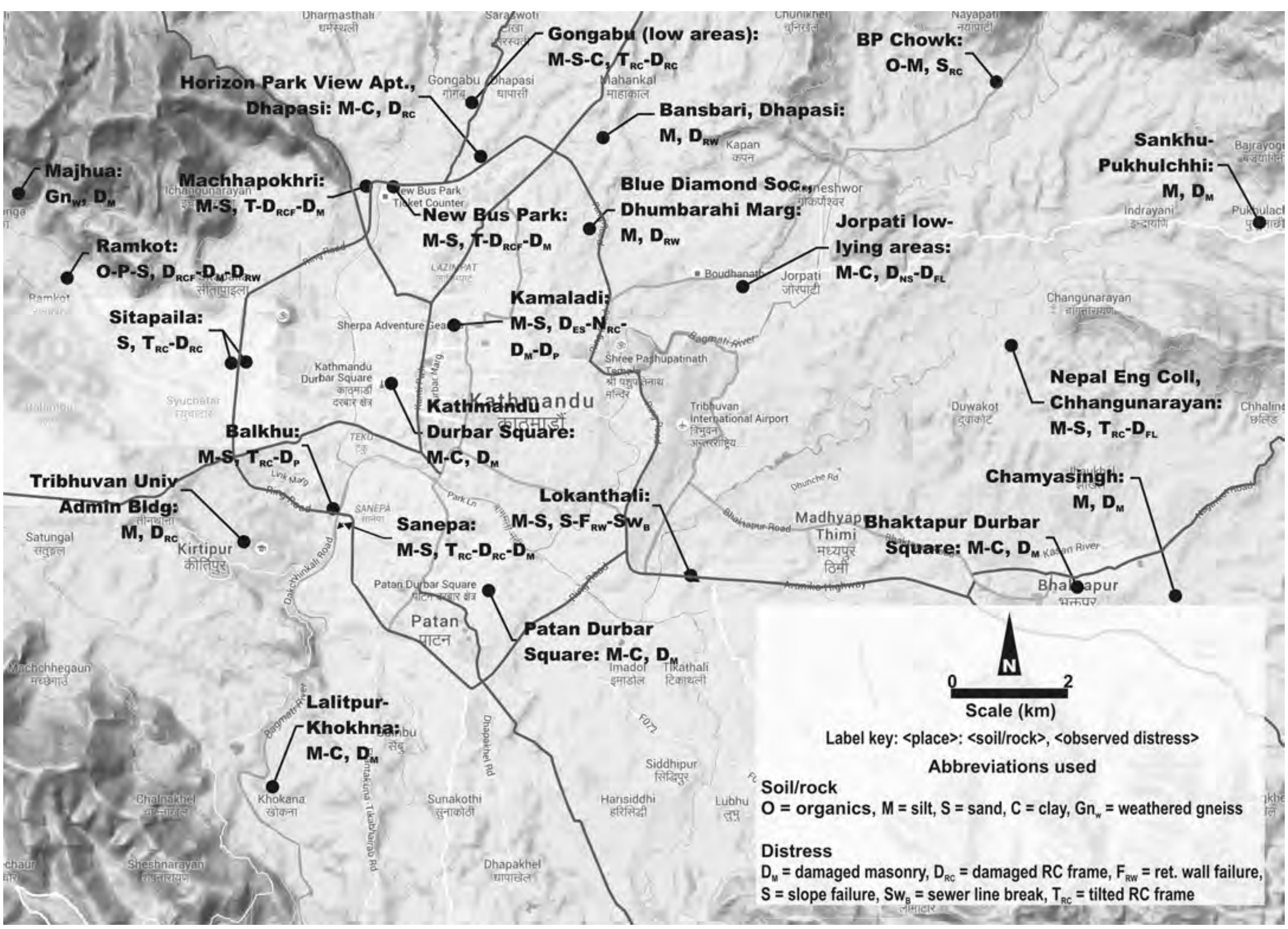

Figure 2. Damages observed in Kathmandu valley

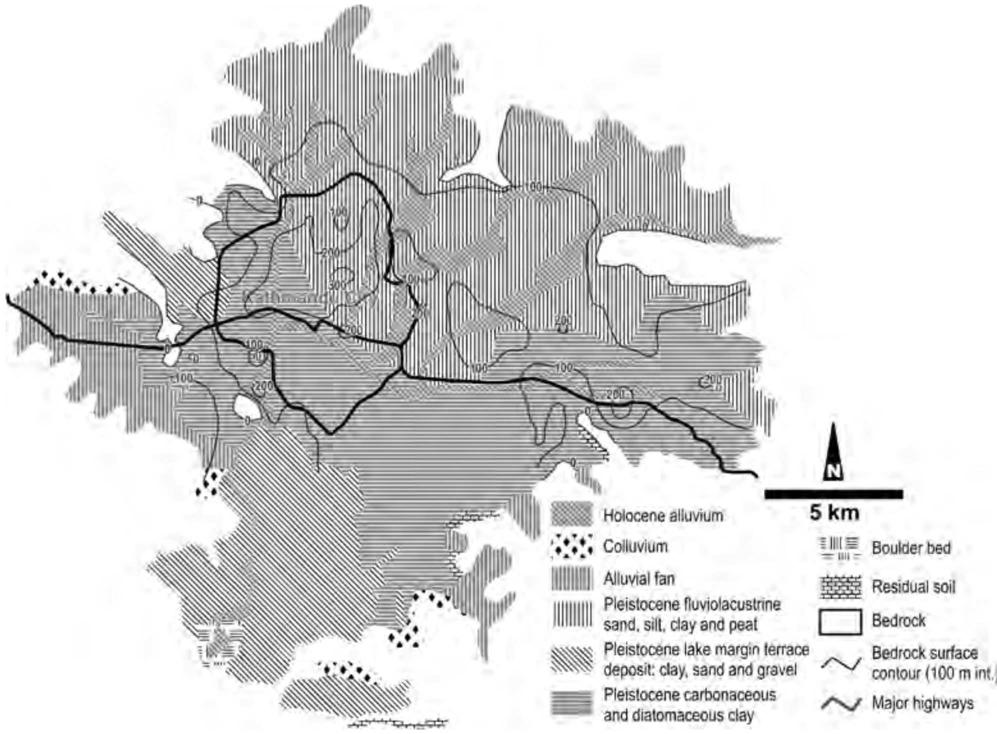

Figure 3. *Kathmandu valley geology (based on Department of Mines and Geology 1998, Piya 2004 and Paudiyal 2012)*

a) Westward view of damaged Basantapur Palace Tower (with Gaddi Baithak partially visible at the left of the photograph), Kathmandu Durbar Square

b) Southward view of a damaged unreinforced masonry structure west of Bhaktapur Durbar Square

Figure 4. *Building damage due to differential settlement*

a) Northward view of the coseismic shoring failure at Kamaladi

b) Westward view of damaged facade of the Sherpa Adventure Gear building due to shoring failure next door

Figure 5. *Building damage due to failure of earth support system*

Damages were particularly intense on river banks. Notable among such areas were Gongabu and Machhapokhri (located on the banks of the Bishnumati), Sanepa and Balkhu (on the Bagmati), Kamaladi (on the Tukachu) and Lokanthali (on the Manohara). Pore water pressure build up within poorly compacted Holocene alluviums comprised mainly of micaceous sands and silty sand would have been the major cause of damages in these areas. Sand and silt ejecta were found around the plinth of the heavily damaged Government Fisheries Department office building in Machhapokhri (Figure 6). Sand and silt ejecta reportedly fouled the fish tanks at this location, although the affected tanks were already cleaned by the time of this field visit. Residential buildings located on Gongabu fish tanks also developed noticeable tilt.

a) Ejecta from underneath the Fisheries Building

b) Eastward view of tilted buildings near fisheries facility

Figure 6. *Building damage possibly due to pore water pressure development at Machhapokri*

Sand boils had also been reported from Imadole, Hatibari, Bungmati and Ramkot area indicating sporadic instances of liquefaction. The embankment and mechanically stabilized earth wall supporting the ring road in Lokanthali suffered heavy distress. A surface sewer line at this location sheared off by about 100 mm also probably due to pore water pressure built up within underlying Manohara river alluvium (Figure 7). Apparent directionality of the ground motion inferred from the damage patterns was another key observation feature of the April-May 2015 earthquakes. Damages due to permanent ground deformation were by and large in the east-west direction, whereas short-period structures were mainly affected by the north-south ground motion.

Historical unreinforced and unconfined masonry structures collapsed or were significantly damaged on account of the shaking intensity coupled with heavy massing. However, masonry structures confined within timber frames performed better (Figure 8). Out of plane collapse and damage of unconfined masonry structures was extensive. Reinforced concrete structures on the whole performed better for the same shaking intensity, than masonry structures but suffered extensive damages due to localised ground motion amplification (site effects) especially along the Kathmandu basin edge and in pockets of Gongabu, Sitapaila and Ramkot. Gongabu area had many buildings on soft stories or stilt, which may be a cause of extensive structural damage observed in this area. Although the damages in reinforced concrete structures in other areas mainly affected the brick infill walls for the most part, shear failure in structural beams was also observed at a few locations. Damage caused by pounding was also widespread. Notable among were those affecting high rise apartment buildings including the newly-built Horizon Tower in Dhapasi, which suffered heavy damages. Confinement within adjoining structures on the other hand kept a number of structurally deficient buildings from collapsing during the earthquake.

a) Westward view towards the Manohara river at Lokanthali ground failure site

b) Northward view of the broken sewer line along the Manohara

Figure 7. *Downslope ground deformation, building damage and sewer line break at Lokanthali*

a) *Undamaged masonry confined within timber straps, Chamyasingh*

b) *Damaged masonry buildings of Chamyasingh*

Figure 8. *Earthquake resistance of confined masonry*

III. LESSONS LEARNT

Nepal earthquake swarm of April and May 2015 caused widespread damages to facilities constructed upon deep alluviums of Kathmandu valley. Insights derived from site response and structural performance during the April-May 2015 Nepal earthquakes could be pertinent in other deep alluvium sites with similar geologic and seismotectonic settings.

Attenuation of ground motion generally led to a reasonable performance at the bottom of Kathmandu valley particularly in those areas underlain by Pleistocene clays and silts, although differential settlement inflicted minor to severe damages to structures in these areas. Damages were more intense on valley margins because of topographic amplification and along stream and river banks because of development of pore water pressure.

Damages were often related to permanent ground deformations triggered by the east-west component of the ground motion, short period amplification for the north-south component of the ground motion and inappropriate choice of foundation systems. Structures supported on isolated shallow footings constructed within backfill placed behind inadequately engineered unanchored earth retaining systems or those overlooking steep slopes were particularly vulnerable to damages due to coseismic permanent ground deformation.

Risks of earthquake-related damages to residential structures at deep alluvium sites near a subduction zone could thus be mitigated somewhat by relying on displacement-based structural design and avoiding isolated, shallow spread footings and constructions behind non-engineered retaining walls. Developments along the banks of streams needs to account for permanent ground deformation due to pore water pressure rise, while those on hilltops or headlands need to account for topographic ground motion amplification and permanent ground deformations due to potential downslope soil movements.

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The Gorkha Earthquake and the Tehri Dam

Roger BILHAM
CIRES and University of Colorado, Boulder CO 80309 USA
bilham@colorado.edu

ABSTRACT

The Tehri dam was designed to withstand a Mw=7.2 earthquake resulting in 0.25 g of shaking for 20 seconds. Geophysical advances in understanding earthquake genesis in the Himalaya have revised upward the maximum credible earthquake for the dam from Mw=7.2 to Mw=8.5, with conjectural peak accelerations possibly exceeding 1 g, and with much prolonged shaking duration. Strong motion data from the 2015 Gorkha earthquake suggest that the Tehri dam could readily survive a replica of the Mw=7.8 earthquake should one occur in the Garhwal Himalaya. The possibility of scaling the Gorkha strong motion record to emulate a possible Mw=8.5 earthquake is introduced in the context of macroseismic intensities observ. ed in the 1934 and 2015 earthquakes. A doubling in the amplitude of the slip pulse between these two earthquakes is probably associated with a doubling in acceleration, although this was not confirmed from intensities observed above points where surface slip doubling occurred in the Gorkha earthquake.

Keywords: *Conjectural peak accelerations, macroseismic intensities, surface slip doubling*

I. INTRODUCTION

The Tehri dam is an earth-fill dam with the approximate shape of a tetrahedron. Its 1.1-km-long lower edge follows one of the two rivers it now impounds, and its 0.57-km-wide crest straddles the gorge through which the Bhagirathi River once flowed. The dam rises 260.5 m above the valley floor and its reservoir can store 4 km^3 of water in a 30-km-long approximately east-west sinuous valley with a surface area of 52 km^2. It was completed in 2006 at a cost of $1000M and provides a power generation capacity of 1 GW. The dam is without doubt a remarkable engineering achievement (Adikhari, 2009) and although it is among the sixth highest in the world, it is less than half the 650 m crest height of the Usoi dam that was created in less than 2 minutes by an earthquake-triggered landslide in 1913 (Ambraseys and Bilham, 2014). Now that the dam exists, a question that must be addressed is whether the dam can safely survive the largest credible earthquake that has been postulated for the Garhwal Himalaya, an earthquake approximately 60 times more powerful than for which it was initially designed.

The location of the Tehri dam was conceived in 1949 and proposed as a viable project in 1961 before the formulation of plate tectonics and before it was realized that the Himalaya define a plate boundary. Because no major earthquake had occurred in the preceding 200 years the notion that historical earthquakes exceeding Mw=8 have could occur near the dam was initially resisted by engineers. In 1972 the possibility of a great earthquake in the Garhwal Himalaya had still not been fully accepted, and as a consequence a design earthquake of Ms=7.2, with a design peak ground acceleration of 0.25g, was considered adequate for engineering considerations. The dimensions of such an earthquake are equivalent to 2 m of slip on an 80 km x 20 km rupture zone.

In the two decades following 1980, discussions among engineers, government officials, and seismologists eventually led to agreement that the maximum credible earthquake (MCE) for the Tehri region should be raised from Ms=7.2 to Mw=8.5 corresponding to an earthquake rupture zone measuring 100 km by 200 km with 10 m of slip (Gaur and Valdiya, 1993; Gaur, 2015). An additional realization was that the rupture zone of the hypothesized maximum credible Mw=8.5 earthquake could occur directly beneath (15 km below) the dam, rather than at some arbitrary distance from it laterally (20-100 km). Despite the 60-fold energy increase associated with this increase in maximum credible earthquake from Mw=7.2 to 8.5, and the 15 km proximity of the rupture zone, the design acceleration for the Tehri dam remained unchanged at 0.25g, as it does to this day.

Since the dam's completion in 2006 additional studies have shown that the maximum credible earthquake in this sector of the Himalaya of Mw=8.5 may be too low. The possibility of magnitudes exceeding Mw=8.8 occurring in the region is based on a 500 year or longer absence of great historical earthquakes, the current rate of contraction observed geodetically (18 mm/yr), and the observation that coseismic displacements in some

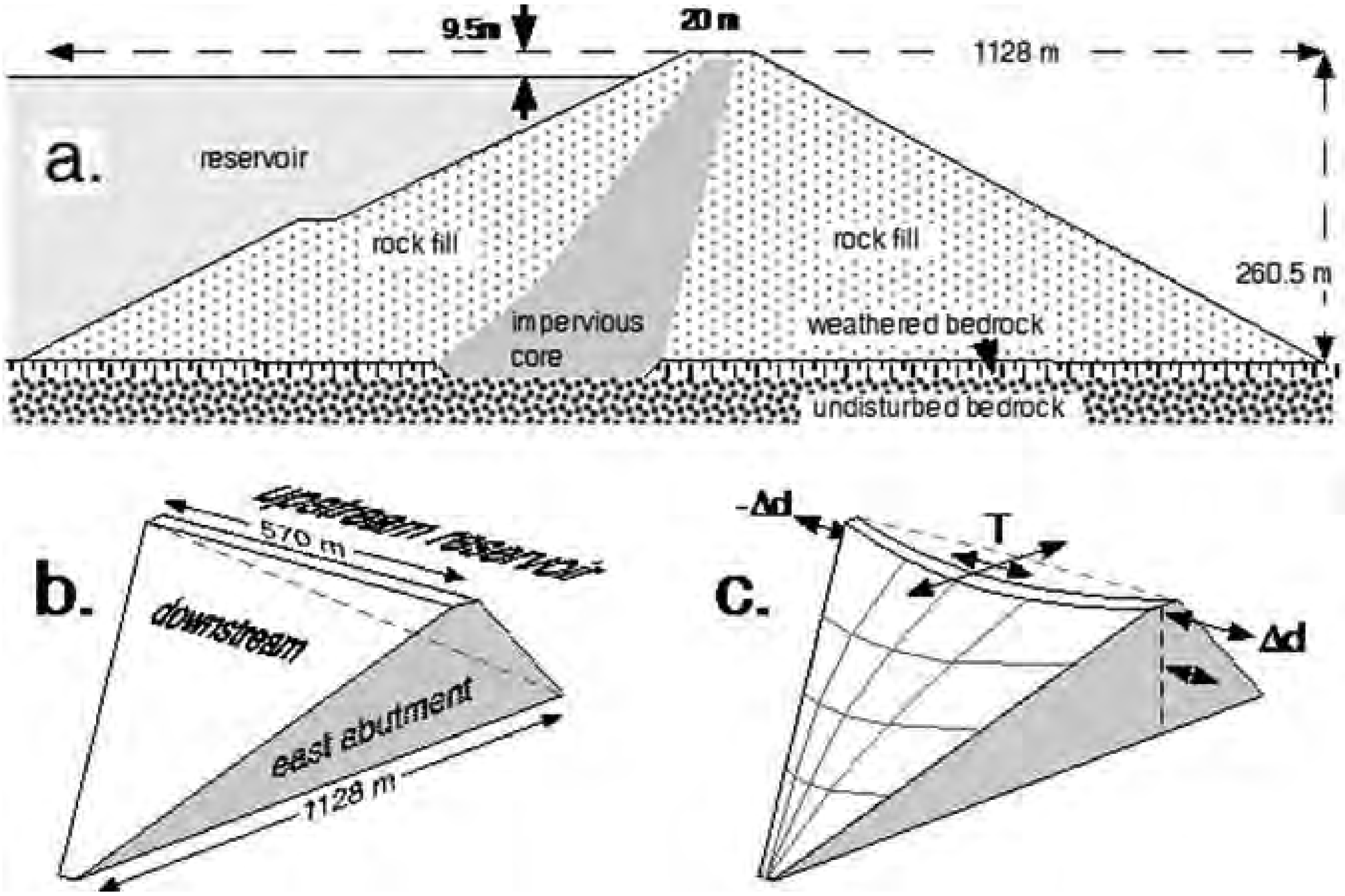

Figure 1a. Schematic section through the central part of the Tehri dam. b. Schematic 3-d view of dam from SE illustrating its tetrahedral form, and c. the fundamental tranverse and longitudinal oscillation modes of its crest T_t and T_L due to its mass and rheology, and strain amplification Δd of the natural buttresses applied to the dam during strong shaking.

Himalayan earthquakes have apparently exceeded 23 m (Kumar et al., 2006; Schiffman et al., 2012; Stevens and Avouac, 2015; Mugnier et al. 2013). Notwithstanding these larger possible ruptures, for the purposes of this discussion we shall retain the adopted MCE of Mw=8.5. We shall also ignore the recent discovery of an active fault near the dam (Gupta et al., 2010), which is unlikely to slip in isolation, but is very likely to be reactivated in a great earthquake.

The adoption of a MCE magnitude of Mw= 8.5 brought with it a further consideration for engineers - an extended duration of strong shaking. A Mw=7.2 earthquake can rupture in 20 seconds (a rupture length of 60 km with a rupture propagation velocity of 3 km/sec), but a Mw=8.5 earthquake, unless it propagates bilaterally would have a duration of 60-80 s (200-240 km along-arc length, and 80-100 km across-arc width). Because continued shaking leads to progressive failure in earth and rock fill dams, critics argued that these early computer test models of shaking at 0.25g for 20 seconds were unreasonably short. The wisdom of retaining 0.25 g as an upper limit to anticipated accelerations was further questioned, some suggesting 0.56 g or even 1.0 g (Brune, 1993). This was countered by the curious proposal by some seismologists that Peak Ground Acceleration (PGA) should be replaced by the notion of EPGA or "effective peak ground motion". EPGA is derived by omitting the largest 10% of all observed peaks in acceleration data, yielding an EPA typically half the value of PGA. EPGA in simple terms, discards the peak accelerations that are actually experienced by a structure, in favor of a truncated accelerogram for the sake of computational simplicity. The ramifications of this physically unrealistic redefinition of the Tehri design acceleration are discussed by Iyengar (1993).

The Tehri dam is now a reality, and the two-decade discussion concerning its design is moot. Seismologists and engineers concerned with downstream population safety have now shifted their attention to calculations posed to identify what combination of acceleration, velocity, displacement and duration would compromise the stability of the dam. These synthetic tests are designed to determine the precise input conditions that can dynamically strain the dam beyond its elastic limit . The dam has an impervious core surrounded by a stabilizing envelope of rock aggregate, and the simulations must determine under what conditions its base, and lower levels spread laterally, thereby lowering its crest, or weaken or rupture its core. Such deformation would permit overtopping,

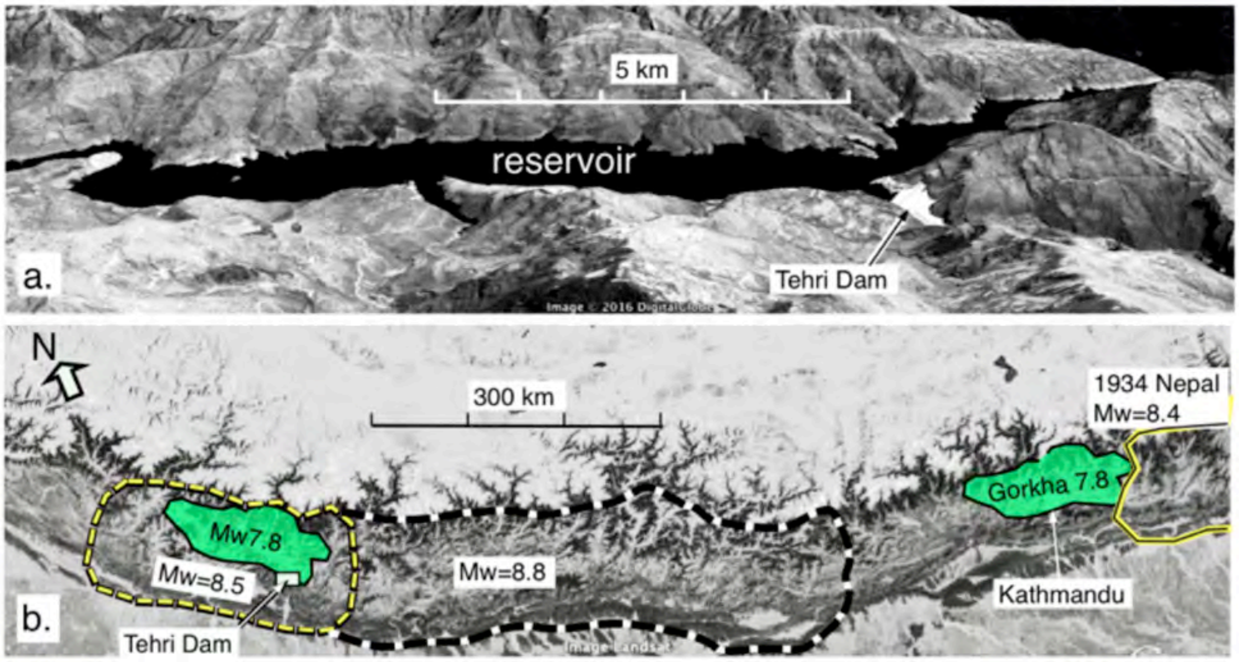

Figure 2. a. Oblique Google Earth view of the Tehri Dam and reservoir from SW in October 2010. b. The location of the dam with the Gorkha rupture zone superimposed on the Garhwal Himal. Strong motion records from rock sites near Kathmandu offer a template for possible future accelerations for a Mw=7.8 earthquake near the Tehri Dam. A possible rupture area for the proposed Mw=8.5 maximum credible earthquake (MCE) is dashed, with the conjectural rupture zone of the enigmatic Mw≈ 8.8 June 1505 earthquake extending eastward into western Nepal.

and ultimately, under sufficient shaking, a breach and collapse of the dam. Testing the dam is undertaken using computer models. The modeling of the dynamic behavior of the dam requires engineering insights concerning its geometry and the strength of its materials, but the realism of synthetic tests depends on the spectral content and duration of seismic shaking used as input to the models. The remainder of this article discusses the intensity and instrumental data from the Nepal 1934 and Gorkha 2015 earthquakes that provide possible constraints on future input to synthetic models of future shaking of the dam.

The search for the perfect seismogram

Preferred dynamic models of the Tehri Dam use as input the seismograms of existing earthquakes. In the mid 1990s, in the absence of a suitable instrumental Mw=8.5 earthquake record, the seismogram from the 1976 Gazli Mw=7.2 earthquake was used in input in computer simulation of dam integrity. The record has the advantage of peak accelerations of 1.3 g vertically and 0.72g horizontally (larger than expected) but has the noted disadvantage that the seismogram has a duration of only 14 s, and although it was associated with 3.3m of slip

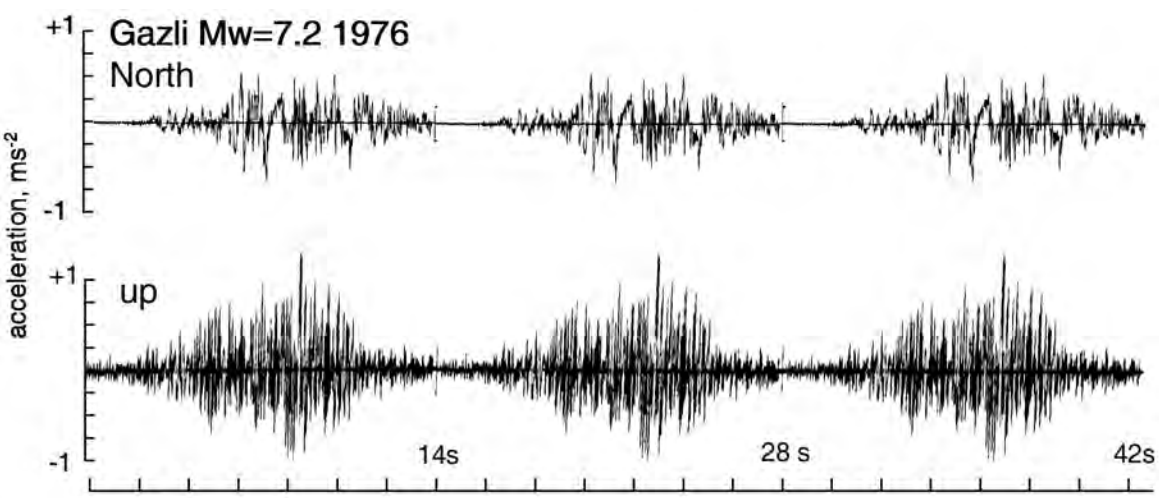

Figure 3 The north and up accelerograms from Karakyr point for the 1976 Gazli earthquake. The 14s record was triplicated to emulate a 42 s duration earthquake (!).

(Hartzell, 1980) it is not a shallow thrust type of earthquake as expected near the Tehri Dam. The dam survived the synthetic Gazli test, but critics noted that the dam was beginning to respond unfavorably to shaking 8s into the test, reaching a maximum response to shaking after 12 s, near the end of the record. To overcome criticism that the brevity of the seismic record thwarted a true test of prolonged shaking the Gazli record was artificially extended by the unusual expedient of running the seismogram through the model three times end-to-end (Ramachandran, 2001). The resulting 42-s-duration synthetic seismogram with its three strong pulses of shaking and three succeeding lulls (Figure 3), apart from still being too short, has no resemblance to seismic shaking during a real Mw=8.5 earthquake. The Gazli results were viewed with scientific dismay (Gaur, 2015), but they satisfied the government-appointed committee (a single engineer), who advised the government that the dam design was adequate to permit completion.

A more thorough synthetic test of the integrity of the dam was undertaken by Sengupta (2010) who used as input a seismogram from the 1985 Mexico City (Michoacan) Mw=7.6 earthquake. This earthquake (Medoza and Hartzell, 1989) more closely emulates what might be anticipated in the Himalaya in that it was recorded in the hanging wall of a subduction zone earthquake on the Mexico Pacific coastline. The surface rupture underwent 6 m of coseismic slip, that was manifest as 1 m of surface slip recorded by the accelerometer. By scaling the observed accelerations from 0.22 g to 0.45 g (a linear multiplication of wave amplitude) he more closely emulated the possible effects of an Mw=8.5 earthquake on the Tehri Dam, however, the record remains short, and in models he used only the first 23s of the 50 s available. Using four well-documented modelling procedures he was able to synthetically induce a range of non-recoverable lateral displacements in the body of the dam, with amplitudes from 0.1 m (negligible) to 7.8 m (perilous), the largest of which would lead to partial settlement of the crest of the dam. The results from some models agreed better than others, but none included the free surface amplification of the valley sides, or the increased transient shear loading from the tsunami in the reservoir north of the dam.

Three shortcomings in the 42s long Michoacan earthquake are that that it does not emulate the ≤80 s of shaking anticipated in a Mw=8.5 Himalayan earthquake, the ≈10 km closer proximity of the Himalayan rupture surface, or the probably much larger (≈10 m) southward offset anticipated in a great Himalayan earthquake. In the past decade two great earthquakes have yielded strong motion data from near their rupture zones: Maule 2010 Mw=8.8 and 2011 Tohuko M=9.0. However, not only are their magnitudes too large to constitute a fair test for a Mw=8.5 Himalayan earthquake, they are recorded by strong-motion instruments near the ocean/continent boundary and hence are not good templates for a Himalayan event. Strong-motion records from these earthquakes are equivalent to recording a Himalayan earthquake from instruments placed in southern Tibet.

The 1999 ChiChi Mw=7.6 and 2015 Gorkha Mw=7.8 earthquakes, however, yielded strong motion records of ground motion above a décollement rupture similar to that which will be experienced by the Tehri Dam. Both records include "fling" (Abrahamson, 2006) and directivity (Somerville et al., 1997). Fling in the Gorkha earthquake in some locations amounted to 3 m, but only 1.5 m was recorded at the location of the available strong motion records (Figure 5a). The Chichi earthquake ruptured to the surface, whereas the Gorkha earthquake stopped in the subsurface, coincidentally near Kathmandu where the

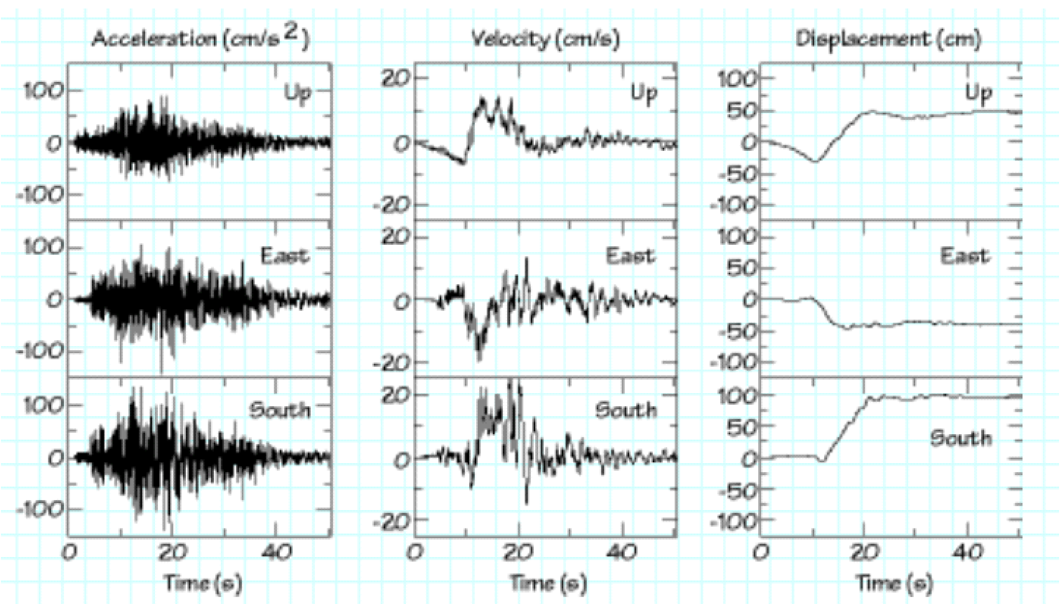

Figure 4. Strong motion record from the 1985 Mw=7.6 Michoacan earthquake with >6 m of slip, showing acceleration, velocity and displacement ≈20 km above the rupture.

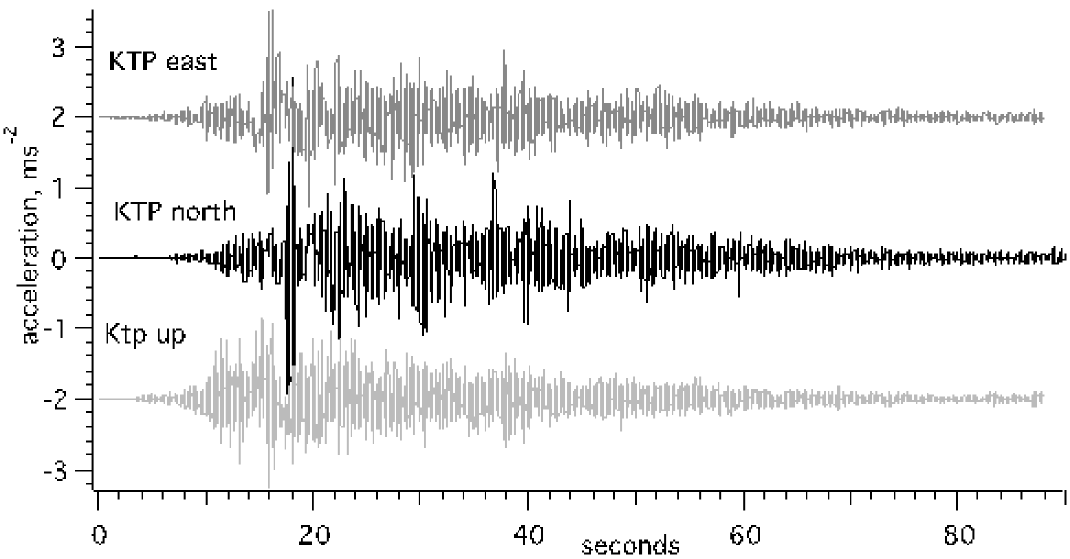

Figure 4. Accelerogram from bedrock site KTP on the SW edge of the Kathmandu Valley for the Gorkha earthquake (Takai et al., 2016). The north and up traces have been offset by 2 ms^{-2}, and advanced 2 s to reduce peak overlap between traces.

only strong motion records from the Gorkha earthquake were obtained. The effects of fling and directivity are discussed below.

In figure 2b the Gorkha earthquake rupture is superimposed on the Garhwal Himalaya with its northern edge corresponding to the zone of seismic decoupling there. In this case, the Tehri Dam has approximately the same geometrical relationship to this hypothetical rupture, as does Kathmandu. Hence the strong motion records available from the Gorkha earthquake are almost ideally suited to examining the response of the Tehri Dam to a Mw7.8 incomplete rupture in the Garhwal Himalaya, with rupture terminating near the dam. Accelerations approach 0.25 g for only a few tens of seconds. Thus, given the 2d simulations of Sengupta (2010), the Tehri Dam's design criteria are adequate to survive a Gorkha type earthquake. This must be considered good news. But how representative are the acceleration records for the Gorkha earthquake? Can they be scaled for a larger earthquake, and if so how?

Gorkha ground motions

As mentioned earlier, the instrumental accelerations observed in the Gorkha earthquake (Figure 4&5) were lower than expected for a Mw=7.8 earthquake elsewhere (Martin et al, 2015; DIxit et al., 2015; Goda et al., 2015). Of the five strong motion accelerometers that recorded the earthquake, four were on the thick sediments of the Kathmandu valley and recorded

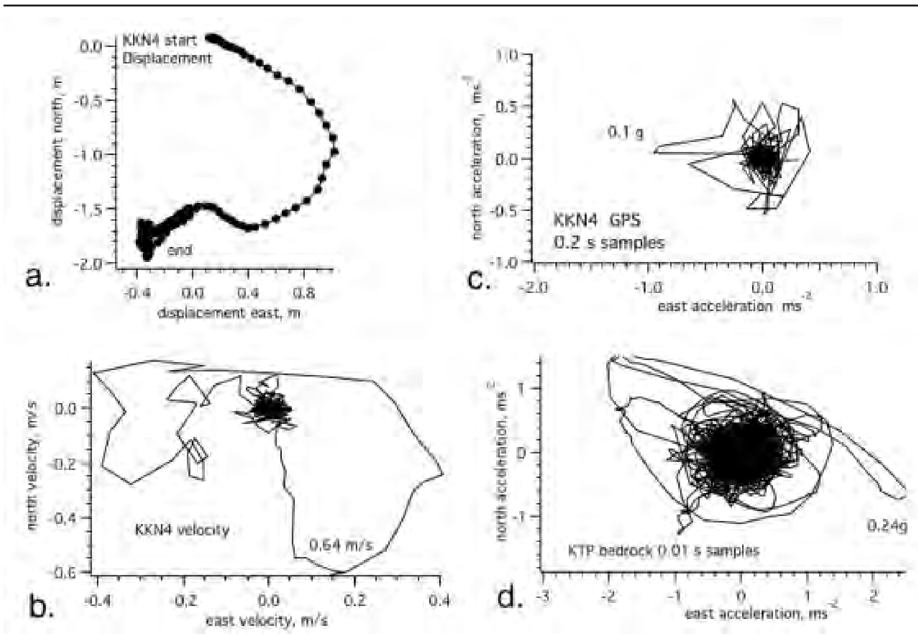

*Figure 5 Bedrock motions from the southern edge of the Gorkha rupture displayed as hodograms- map views of horizontal components. **a.** GPS positions sampled every 0.2s at point KKN4 reveal the wholesale 1.5 m SSW displacement of the surface rocks of Nepal to the SSE over the Indian plate. Simultaneously the point rose 1 m (Galetzka et al. 2015). The horizontal displacement is known as "fling" because of its non-recoverable offset. **b**. KKN4 GPS velocity, the derivative of 5a. **c**. KKN4 acceleration, the derivative of 5b **d.** KTP acceleration (Takai et al., 2016).*

amplified and prolonged resonance (Dixit et al., 2015; Takai et al, 2015), and are thus of no utility for Tehri Dam studies. Accelerations from the one instrument on bedrock (KTP in Takai et al, 2015), on the SW edge of the valley are shown in Figure 5 as an east-north vector hodogram (Figure 5d). The plot indicates the relatively brief period near the time of the fling pulse when accelerations significantly exceeded 0.1 g. A GPS unit at KKN4 sampling positions 5 times per second on the northern edge of the valley recorded somewhat lower accelerations (Figure 5c) due its lower sampling frequency.

Unfortunately there were no instrumental records above the rupture zone of the Gorkha earthquake except those near Kathmandu. Goda et al. (2015) illustrate the difficulty in emulating mezzocentral accelerations from existing theory, which are uniformly unable to accurately characterize accelerations above a subhorizontal rupture. Could the low observed accelerations have been confined to the Kathmandu basin and its environs, where the influence of basin interaction was large? Could the accelerations have been influenced anomalously by the mechanics of stopping phases near the southern edge of the rupture?

A partial answer this question comes from observed macroseismic intensity observations from above the rupture. With the exception of the study of Martin et al., (2015), at the time of writing no comprehensive survey of intensities of the mezzoseismal zone has been published. Figure 6 shows some of their data omitting many from the Kathmandu Valley. Observed intensities (EMS 6.6±0.5) made in traverses across the rupture indicate that accelerations probably did not exceed 0.2 g except on ridges (EMS≤8). Thus both instrumental data and macroseismic data indicate that accelerations above the Gorkha rupture on average did not exceed 0.25 g. This bodes well for the Tehri Dam should it enjoy a similar earthquake.

Fling pulse and the rupture process

The most significant difference between seismograms used in previous simulations of Tehri Dam integrity and the newly available strong motion records from the Gorkha earthquake is the presence of the large translation pulse (fling) that arrives 27 seconds after the mainshock. In the Michoacan record it approaches 1 m. In the Gorkha record it exceeds 1.5 m (Fig 5a). Unfortunately the double-integrated time series emulation of horizontal displacements from data in Figure 4 (not shown) is marred by apparent tilt of the sensor during the record.

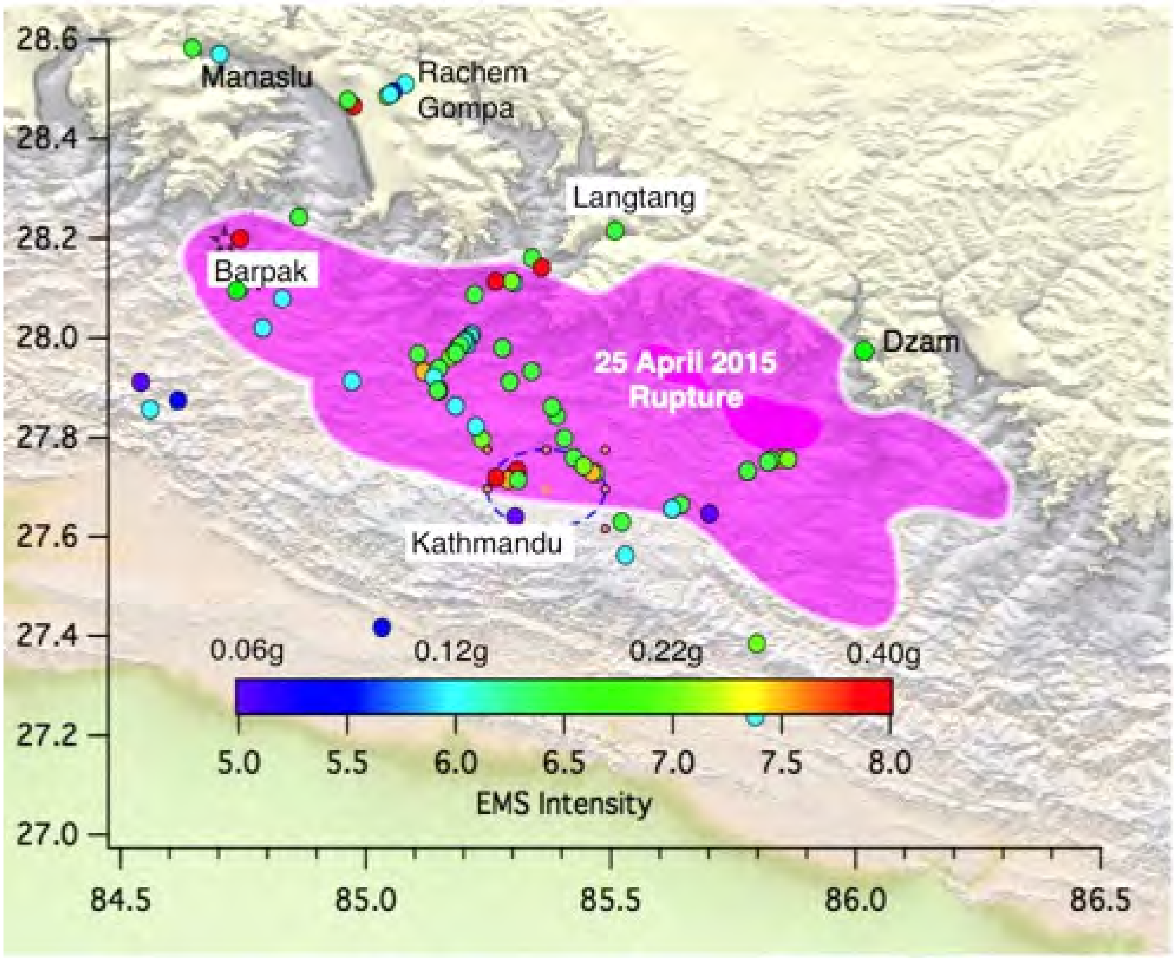

Figure 6. EMS Intensities assessed in traverses across the Gorkha rupture. The mean EMS value for 45 observations above the rupture is EMS 6.6±0.5 The data suggest accelerations of 0.2± 0.1g prevailed above the rupture.

The fling pulse is caused by the southward motion of the Himalaya over the Indian plate, the process that permits the Indian plate to descend beneath southern Tibet. The fling is manifest as an incremental displacement at every point above the rupture, which in the case of the Gorkha earthquake has been precisely quantified by geodetic measurement before and after the earthquake (e.g. Lindsey et al., 2015). Maximum displacement occurs on the rupture surface at 10-15 km depth, and had the rupture propagated uniformly to the frontal thrusts at the surface, displacements throughout the rupture (with the exception of regions near the northern, eastern and western edges) would have closely matched these maxima because the earth's surface is traction-free.

Rupture propagated from the nucleation region 80 km to the NW of Kathmandu towards the city at 2.5-3 km/s slowing as it passed (Hayes et al., 2015). The physics of the slip process is currently the subject of much research and no single mechanism is presently preferred (e.g. Andrews and Ben Zion, 1997, Anooshehpoor and Brune 1999). Melting, dynamic separation, and fluid pressurization have been invoked as a mechanism for an instantaneous reduction in friction to close to zero. Very low friction is necessary for the base of the Himalaya to be momentarily freed from the Indian plate beneath it and to respond to the compressional strain that propels overlying rocks southward. It is certain that only a small patch of the entire rupture surface is in motion at any one moment, and that displacement is rapid and able to nucleate additional seismic waves as it slips. Thus during rupture, myriads of smaller shocks are generated (Avouac et al., 2015). These initiate compressional p-waves and transverse shear s-waves that radiate from breaking points on the rupture zone in all directions at velocities of 3.5-7 km/s. Waves radiating in the direction from which the propagating rupture arrived fall far behind the rupture front, but those heading in the direction of the rupture are barely able to outpace the rupture, and hence may arrive as a burst of energy slightly before the arrival of the rupture front. Should the s-wave velocity equal the propagation velocity a extreme scenario in which the slow moving s-waves from numerous secondary propagation shocks all arrive at the same time at a point resulting in constructive interference and significantly amplified surface displacement. The doppler-like compression of forward propagating energy is known as *directivity*.

Despite the propagation front traveling SE towards Kathmandu, a condition optimum for directivity amplification near Kathmandu, accelerations recorded near the city before the arrival of fling pulse remained below 0.1g.

In the case of the Gorkha earthquake the rupture did not continue to the southern edge of the Himalaya and instead terminated near the southern edge of the Kathmandu valley. Maximum displacements on the rupture north of Kathmandu locally attained 7 m but horizontal surface displacements above this maximum were reduced by elastic compression of surface rocks, which resulted in Poisson's-ratio elastic thickening near Kathmandu, and hence the observed uplift of more than 0.8 m

The effect on the Tehri Dam of a replica of the Gorkha earthquake.

We concluded above that the accelerations to which Tehri Dam will be subjected are within the design acceleration of 0.25g if we assume that an incomplete rupture of the Garhwal Himalaya occurs as depicted in Figure 2b. However, the horizontal and vertical fling pulse evident in Figure 5a is larger than in previous emulations - would this not result in unpredictably high stresses in the dam? The following reasoning shows this unlikely. The sickle-shaped translation of the rocks surrounding the Kathmandu basin took 9 s to shift the city 1.5 m to the south, but was mostly complete in 5s seconds (the dots indicate 0.2s increments). This low frequency non-recoverable pulse stimulated the Kathmandu valley sediments into resonance. However, the fundamental resonance of the Tehri Dam is 1-2s. The effect of translating the Tehri Dam 1.5 m to the south in 5-9 s periods is thus unlikely to stimulate failure modes within the dam. Just as a short stubby building resists damage, the Tehri Dam is apparently immune to the direct effects of a Gorkha-type slip pulse. The pulse will, however, stimulate a tsunami in the Tehri reservoir which may result in overtopping if the reservoir is close to capacity. It has a 9.5 m freeboard when full, (Figure 1a) but this could increase to more than 100 m if an earthquake occurs when the reservoir is low.

The net conclusion is that were a Gorkha type Mw=7.8 earthquake to occur near Tehri it would survive both the high frequency accelerations and its fling pulse. Not considered is a conspiracy of directivity effects, and abutment convergence displacements (Figure 1 c) caused by topographic amplification in the valley that may result in unusually high local amplifications.

II. DISCUSSION: Scaling the Gorkha earthquake upward to Mw=8.5.

It is now possible to develop a synthetic seismogram of arbitrary complexity above a rupture (Rai and Beroza, 2002; Erdik and Durukal, 2003, Raghu Kanth and Iyengar), and such models permit a wealth of conjectural ruptures to be synthesized. However, such models are limited by the precision of physical insights into to the rupture process.

The traditional method that has been used to increase the magnitude of an earthquake input to a synthetic models is to multiply the amplitude of an available strong motion record by a linear scaling factor. The pitfalls and methods of undertaking scaling are discussed by Bommer & Acevedo (2004) and Watson-Lamprey & Abrahamson (2006). The conversion of the Gorkha earthquake record into a plausible simulation of a Mw=8.5 Himalayan earthquake must realistically account for increased along-strike rupture duration, and the correct adaption of the record for complete up-dip rupture to the frontal thrusts. The 2015 rupture length (≈150km) is only 30-50% shorter than the MCE rupture of 200-300 km, and is very similar to that associated with the 1934 earthquake. Hence along strike duration requires padding, but does not constitute a significant spectral problem. It is, however, essential to apply a realistically long duration period of strong synthetic shaking to the dam to model such factors such as a pore pressure weakening in the dam structure (Bureau, 2003).

A less easy problem to solve is how to correctly emulate the possibly modified period of the fling pulse. If the duration of the slip pulse in a 10-15 m rupture Mw=8.5 rupture occurred in the same 5-9 s duration as seen in the Gorkha earthquake, the accelerations (and decelerations) would obviously increase accordingly. However, we have some evidence that this does *not* occur from the intensity data in Figure 3. Although mean slip was approximately 3.5 m, and slip locally exceed 6 m, surface displacements on the rupture surface north of Kathmandu did not much exceed 3 m (Lindsey et al., 2015) roughly double that seen in Kathmandu. Despite double the surface slip, EMS intensities near and north of Kathmandu remained the same (Figure 6). A caveat remains, of course, in that the EMS damage scale may not provide a sufficiently precise constraint of both high frequency shaking and monotonic fling.

A second constraint on whether slip pulse accelerations scale with Himalayan earthquake magnitude comes from the 1934 Mw=8.4 earthquake. The magnitude reported for this earthquake depends on the dip applied to the calculation of magnitude. Earlier magnitudes adopted much larger dip than we now know is appropriate and values of the order of 8.1<Mw<8.2 have commonly been cited. The value used here is that derived by Chen and Molnar (1989). Unfortunately, however, only eight observations are available from above the rupture of the 1934 earthquake. They yield an average value of EMS 7.6±0.7 including one outlier, a single EMS=9. Based on this magnitude and the rupture area shown in figure 7 we infer that mean slip in the earthquake as 8m.

From this we conclude that an approximate doubling in décollement slip from ≈3.5m to ≈8m results in a one unit increase in EMS Intensity which corresponds to an approximate doubling in acceleration. The sparsity of macroseismic data for the 1934 earthquake, and the tendency for reports of shaking to focus on the worst-damaged structures in villages, suggests that the factor may be less than 2. In contrast, a doubling in surface slip between the southern edge of the rupture, and the central rupture in the Gorkha earthquake was accompanied by no discernable increase in EMS Intensity, possibly related to the absence of a diversity in building styles above rupture suited

Figure 7 Reported EMS intensities for the Nepal 1934 Mw=8.4 earthquake (Martin and Szeliga, 2004) with a conjectural rupture zone (dashed). Epicenter is from Chen and Molnar (1987) and MFT rupture from Sapkota et al., 2012. Intensities above the 1934 rupture are sparse but average EMS 7.6±0.7. With the conjectural rupture shown, mean slip in 1934 would be ≈8 m.

to identifying degrees of damage near EMS intensity 7. For the 1934 Mw=8.4 earthquake a single intensity 9 observation is reported (PGA of 0.65-1 g) and this value, as in anomalously high values observed for the Gorkha earthquake, may be due to the effects of ridge amplification.

An implication of acceleration being proportional to slip amplitude is that the slip pulse duration is independent of slip. Alternatively if acceleration remains constant in the presence of increasing slip, slip pulse duration must increase with slip amplitude. It would be of value to apply additional data to distinguish these two end members. In view of the conflicting conclusions related to scaling the fling pulse in the Gorkha record to a Mw=8.5 earthquake, a conservative approach would be to double the amplitude throughout of the record. This would peak raise accelerations to ≈0.5 g while maintaining the same spectral content.

III. CONCLUSIONS

In this article I review the potential utility of using the Gorkha earthquake as a template for future shaking of the Tehri Dam in the Garhwal Himalaya. Based on the results of published 2d synthetic models, I conclude from the one available rock accelerogram for the Gorkha earthquake (KTP, Takai et al., 2016), that the dam will survive a similar Mw7.8 earthquake unscathed. The possibility of the dam sustaining damage in a much larger earthquake requires using a version of the Gorkha record appropriately scaled to longer duration and significantly larger fling. In the Gorkha record fling was 1.5m, but the non-recoverable southward translation of the dam in a future great earthquake may exceed 15 m.

Inferred slip in the Mw8.4 Nepal 1934 earthquake was roughly double that recorded in the Mw7.8 Gorkha 2015 earthquake and mean rupture intensities were EMS 7.6±0.7 and EMS=6.6±0.5 respectively. From these ratios we deduce that fling pulse accelerations may scale linearly with earthquake magnitude. In contrast, no difference could be discerned between intensities in the Gorkha earthquake where surface displacements varied by a factor of two. A conservative approach would be to adopt a linear scaling relation relationship since EMS intensities above the Gorkha earthquake are derived from a limited range of building styles, with consequent difficulties in discerning subtle difference in shaking intensity.

Hitherto synthetic 2D simulations of Tehri Dam shaking have been limited to durations of 23s or to artificially unrealistic shaking with 42s duration. Prolonged cyclic shaking for 60 s or more may lead to progressive failure associated with non-linear deformation, and pore pressure increase that may or may not occur. It is timely to subject the Tehri Dam to a full 3D synthetic computer simulation. Scaled and lengthened versions of the Gorkha earthquake accelerograms have known inadequacies, but they appear currently to provide a more realistic input to such models than any available hitherto.

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Systematic Approach to Post Earthquake Repair & Rehabilitation of Structures

E.GOPALKRISHNAN
Head,Dr. Fixit Institute of Structural Protection & Rehabilitation,Mumbai,India

S.C.PATTANAIK
Team Head,Dr. Fixit Institute of Structural Protection & Rehabilitation,Mumbai,India
Email ID:e.gopalkrishnan@pidilite.com

ABSTRACT

Post earthquake cracks in masonry and concrete structures need to be diagnosed properly & to be repaired with highly flexible materials. The structural cracks need more attention than non structural cracks. The repair materials and methodology are different depending upon types of cracks, their locations, joints, structural members etc.The structural audit with non-destructive tests are very much helpful to understand & diagnose the problems & defects. The present paper focuses the various types of flexible polymeric materials that are being used for repair of post earthquake cracks in structures.

Key words: *Cracks, Crack repairs, Cementitious grouts, Polymer modified cementitious grouts, Epoxy injection& Micro concrete*

I. INTRODUCTION

Concrete structure develops some typical types of cracks during the earthquake wherever adequate seismic retrofit is not being considered. Most of the cracks happen at the junction of column & beam radiating diagonally from junction towards column bottom and towards beam at bottom surface. The width and depth of these cracks depend on movement of the structural members. While concrete becomes older, these cracks become wider during earthquake leading to serious degradation and subsequently structural failure of the member causing damage to structural strength, durability and serviceability.

II. CONDITION EVALUATION

The visual inspection and selected non-destructive tests should be carried out to test various parameters to understand the level and extent of damage in RCC structures. These tests are given as follows:

- Homogeneity of concrete by Ultrasonic pulse velocity (UPV)
- Concrete surface hardness by Rebound Hammer
- In-situ strength of concrete by core testing
- Petrography examination of concrete and its constituents
- Infrared thermography for detecting voids, displacement of the structure

It may not be required to all the NDT tests at the site. It is very much important to understand the pattern of damage by observing all the cracks and to find out the inadequacy in designing and detailing. The structural drawings and architectural drawings need to be checked and evaluated for the inadequacy in reinforcement and cross section of the structural member. The residual strength of the structural member should be given as input to analysis and design of the structures for seismic stability.

III. SEISMIC STABILITY METHODS

There are various seismic stability methods available that can be introduced as per the requirement. However, post-earthquake we often come across a lot of cracks in structural members and bulking of column. While the joints are more vulnerable for development of cracks. Instead of making joint more rigid, it should be more flexible to accommodate the movement of the structures. While it may not be possible to make joint as more flexible but the cracks in those joints can be retrofitted with polymeric crack filling materials. Some of the seismic stability methods are as follows:

- Modification of roofs
- Substitution or strengthening of floors

Strengthening of walls including provision of horizontal and vertical bands or belts, introduction of 'through' or header stones in thick stone walls, and injection grouting etc.

- Adding to the sections of beams and columns by casing or jacketing etc.
- Adding shear walls or diagonal bracings
- Strengthening of foundations if found necessary

Repairs can be differentiated into structural repair and non-structural repair. Structural repairs are the systems used to strengthen the building to increase its performance life and may be re-instating it into stable condition due to distress happened by earthquakes. While, general repair is that which oneadopted for cosmeticrepairtobringthestructurebackintogoodcondition.

IV. CRACK REPAIRING METHODS

While selecting the repair material for a particular type of crack the following points should be considered. Materials for non-structural crack repair of dormant nature should be a rigid material. The crack should be three to four times wider than the largest aggregate particle. Cementitious, polymer modified cementitious grouts of acrylic, styrene-acrylic and styrene-butadiene should be used for wider cracks. However polyester and epoxy resins should be used for injection of dormant cracks. For much wider cracks flexible material of polysulphide or polyurethane sealant should be used [1, 2, 5]. Before repair of any non-structural cracks the factors have to be considered are: whether the crack is dormant or live; the width and depth of the crack; whether or not sealing against pressure is required, and, if so, from which side of the crack will the pressure be exerted and whether or not appearance is a factor.

Non-structural cracks may range in width from 0.05 mm or less (crazing) to 5 mm or more. The width of the crack has a considerable influence on the materials and methods to be chosen for its repair.The fine cracks are repaired by low viscous epoxy resin and other synthetic resin by injecting. Wider cracks on a vertical surface are also repaired by injection methods. Cracks on horizontal surface can be repaired by injection or by crack filling by gravity.

Non-structural cracks, where the repair does not have to perform a structural role, can be repaired by enlarging the crack along the external face and filling and sealing it with a suitable joint sealer. This method is commonly used to prevent water penetration to cracked areas. The method is suitable for sealing both fine pattern cracks and larger isolated defects. Various materials are used, including epoxies, urethanes, silicones, polysulphides, asphaltic materials and polymer mortars [4]. Polymer mortars are used for wider cracks. The crack is routed out, cleaned and flushed out before the sealant is placed. It should be ensured that the crack is filled completely. Where ever a cementitious material is being used, dry or moist crack edges must be wetted thoroughly.While the structural and non-structural cracks can be repaired with different polymeric materials but the strengthening of excessive damaged structures can be made with jacketing and micro- concrete. The following repair methods are being discussed herewith:

- Masonry repair by grouting of cracks
- Non-structural crack repair
- Structural strengthening by injection grouts
- Structural strengthening by jacketing with micro concrete
- Sealing the joints with flexible sealants

4.1. Masonry Repair by Grouting of Cracks

While cracks in any masonry structures cause different pattern of cracks. The repair of such cracks needs to be grouted or injected with polymer modified cementitious grouts. It is used for repair of cracks that are 5 mm and greater in width. It is a mixture of cementitious material and water, with or without aggregate that is proportioned to produce a pourable consistency without segregation of constituents.

Cement-based grouts are available in a wide range of consistencies; therefore, the methods of application are diverse. These materials are the most economical of the choices available for repair. They do not require unusual skill or special equipment to apply, and are reasonably safe to handle. These materials tend to have similar properties to the parent concrete, and have the ability to undergo autogeneous healing due to subsequent hydration of cementitious materials at fracture surfaces. Shrinkage is a concern in such type of grouts. These are not suitable for structural repairs of active cracks.

It is a mixture consisting primarily of cement, fine aggregate, water, and a polymer such as acrylic, styrene-acrylic, styrene-butadiene, or a water-borne epoxy. The consistency of this material may vary from a stiff material suitable for hand-packing large cracks on overhead and vertical surfaces to a pourable consistency suitable for gravity feeding cracks in horizontal slabs. These materials are generally more economical than polymer grouts, and the performance, with respect to bond strength, tensile strength, and flexural strength, are improved compared with cement-based materials that do not contain any polymers. These materials are filled in the cracks by some form of routing and filling the crack (Figure 1).

Figure 1. Crack to be filled with ready to use polymeric repair grout

For application of polymer modified cementitious grouts generally, some form of routing and surface preparation, such as removal of loose debris are needed. Pre-wetting should be done to achieve a saturated-surface-Dry (SSD) condition. Grouts are generally to be mixed to a pourable consistency by using a drill and paddle mixer, and the consistency may be adjusted thereafter. Application should be done by hand troweling or dry packing into vertical and overhead cracks to fill all pores and voids.Finally, a suitable coating to be applied on the repaired surfaces.The detail steps are given as follows:

4.1.1 Minor and medium cracks (crack width 0.5 mm to 5.0 mm) in masonry buildings

Material/equipment required

(i) Plastic / aluminum nipples of 12 mm diameterand 30 to 40 mm long.

(ii) Cement slurry admixed with polymer

(iii) 1:3 cement sand mortar with polymer for sealing of the cracks.

(iv) Compressor for injecting the slurry.

The step by step method of crack filling in masonry structures given as follows:

- Remove the plaster in the vicinity of crack exposing the cracked bare masonry.
- Make the shape of crack in the V-shape by chiseling out.
- Fix the grouting nipples in the V-groove on the faces of the wall at spacing of 150-200 mm C/C.
- Clean the crack with the compressed air through nipples to ensure that the fine and loose material inside the cracked masonry has been removed.
- Apply a bonding coat then seal the crack on both faces of the wall with polymer modified mortar (cement mortar 1:3) and allowed to gain strength.
- Inject water starting with nipple fixed at higher level and moving down so that the dust inside the cracks is washed off and masonry is saturated with water.
- Make cement slurry with 1 : 1 (1cement : 1 water) with polymer and start injecting from lower most nipple till the cement slurry comes out from the next higher nipple and then move to next higher nipple.
- After injection grouting through all the nipples is completed, replaster the surface and finish the same.

4.1.2 For Wider cracks more than 5 mm

Wherever the cracks are wider than 5 mm,it is not possible to repair the same cracks with injection grouts.Additionally such cracks further need to stitching with galvanized steel clamping(Figure 2).

Material/equipment required

(i) Plastic / Aluminum nipples of 12 mm dia (30 to 40 mm long)

(ii) Sealing with polymer modified mortar.

(iii) Compressor for injecting the slurry.

(iv) Galvanized steel wire fabric (16 to 14 gauges i.e. 1.5 to 2.03mm diameterwire) with 25 mm x 25 mm mesh size.

(v) Galvanized steel clamping rod of 3.15 mm or 5 mm diameterand150 mm long wire nails.

- Remove the plaster in the vicinity of crack exposing the cracked bare masonry.
- Make the shape of crack in the V-shape by chiseling out.
- Clean the crack with compressed air.
- Fill the crack with polymer modified from both sides as deep as feasible.
- Provide wire mesh on both the faces of wall after removal of plaster in the region of repair to a width of 150 mm on each side of the crack.
- Clamp the mesh with the wall using clamps or wire nails at the spacing of 300 mm c/c.
- Plaster the meshed area with Polymer Modified Cement Mortar (1:3 cement: sand), covering the mesh by a minimum of 12 mm.
- Stitching by steel clamping

Figure 2. Stitching of cracks in masonry structures

4.2. Repair of Non-structural cracks

Non-structural cracks of dormant nature need to be sealed with flexible ready to use polymeric crack filling materials of acrylic based. While Dr. Fixit Crack X Paste should be used to seal the cracks upto5mm wide but Dr. Fixit Crack X shrink free should be used to seal the cracks upto10mm width. Dr. Fixit Crack X paste is a single pack, ready to use flexible putty for filling the cracks up to 5 mm wide. The method is suitable for sealing both fine pattern cracks and larger isolated defects.

- The crack is routed out, cleaned and flushed out before filling it with ready to use crack filling material.
- It should be ensured that the crack is filled completely. Where ever a cementitious material is being used, dry or moist crack edges must be wetted thoroughly.
- Surface must be free from dust, oil, grease, and loose particles etc. Moisten the surface before applying Dr. Fixit Crack-X Paste.
- For porous surfaces, apply primer coat prepared with Dr. FixitCrack-X Paste and water in 1:1 proportion over the crack.
- Fill Dr. Fixit Crack-X Paste when the surface is tacky and not dried completely.
- Press Crack-X Paste firmly into the crack with a spatula or putty knifes and level with the surface
- Care must be taken to avoid formation of cavities or bubbles during application.
- Allow it set for 24 hours and then apply another coat of Dr. FixitCrack-X Paste

4.3. Structural Strengthening by injection grouts

Epoxy is the strongest material for injecting very fine cracks in structural members which makes the structures rigid and arrest all the fine cracks inside the concrete structural members. It is being used for grouting the cracks having width 0.5 mm to 2 mm. However the material should be100% solid resinous solvents or non-reactive diluent should be present in the resin.

One of the potentially effective repair procedures is to inject epoxy under pressure into the cracks.Dr. Fixit epoxy injection grouts of two component based should be used for fine structural crack repairs. The injection procedure will vary, subject to the application and location of the crack(s), with horizontal, vertical, and overhead cracks requiring somewhat different approaches. The approach used must also consider accessibility to the cracked surface and the size of the crack. Cracks can be injected from one or both sides of a concrete member. If access is limited to only one side, installation procedures may include variations in epoxy viscosities, injection equipment, injection pressure, and port spacing to ensure full penetration of epoxy into the crack. Depending on the specific requirements of the job, crack repair by epoxy injection can restore structural integrity and reduce moisture penetration through concrete cracks of 0.05 mm in width and greater. However, before any concrete repair is carried out, the cause of the damage must be assessed and corrected and the objective of the repair understood. If the crack is subject to subsequent movement, an epoxy repair may not be applicable.

Hence any structural crack repair should be injected with an epoxy resin. The compressive and bond strength of epoxy resin is higher than concrete itself. While injecting with epoxy two parameters are more important; viscosity and pressure required for pumping. Depending upon the width of the crack the viscosity and pressure has to be selected. In general the manufactures supply as low viscous and very low viscous resins as injection material. Epoxy is also available as moisture sensitive and moisture insensitive crack repair material.

4.3.1 Material Properties and Equipment

Epoxy resins are injected for repair of hair line cracks and fissures as narrow as 0.05 mm due to their unique property of super low viscosity. The appropriate viscosity of the epoxy will depend on the crack size, thickness of the concrete section, and injection access. For crack width of 0.3mm or smaller a low viscosity epoxy injection can be used. For wider cracks, or where injection access is limited to one side, a medium to gel viscosity material may be suitable. Injection can be made of low pressure (Figure 3) or high pressure system (Figure 4) depending on the nature of cracks. It is better to use two-component pumps with a static mix head to prevent premature reaction. The requirement of epoxy resin to bond hardened concrete to hardened concrete as per ASTM C881 "Standard Specification for Epoxy-Resin-Base Bonding Systems for Concrete," [3] identifies the basic criteria for selecting the grade and class of epoxies.

Figure 3. Hand pump for injection

Figure 4. High pressure injection pump

4.3.2 Application of Epoxy Injection

Surface preparation

The cracks should be cut and cleaned properly. Any contamination should be removed by flushing with water or some especially effective solvent. Then the solvent should be blown out with compressed air, or adequate time should be given for air drying. The surfaces should be sealed. This keeps the epoxy from leaking out before it gelled. A surface can be sealed by brushing an epoxy over the surface of the crack and allowing it to harden. If extremely high injection pressures are needed, then the crack should be cut into a V-shape, filled with an epoxy, and should be stroke off flush with the surface. The entry ports should be installed thereafter.

Fixing of injection ports/nozzles

There are three ways to do this. Fitting of nozzles to be inserted in drilled holes should be made by drilling a hole into the crack for 8 mm dia injection packers @ 200 to 300 mm c/c, penetrating below the bottom of the V-grooved section. A fitting such as a pipe nipple should be inserted or tire valve stem should be inserted into the hole and bonded with an epoxy adhesive. A vacuum chuck and bit will help to keep the cracks from being plugged with drilling dust. The second method is by bonded flush fitting. When the cracks are not V-grooved, a common method of providing an entry port is to bond a fitting flush with the concrete face over the crack. Last method is by interruption in seal. Another way to allow entry is to omit the seal from part of the crack. This method uses special gasket devices that cover the unsealed portion of the crack and allow injection of the adhesive directly into the crack.

Mixing

Mixing the two components of epoxy injection grout of base and hardener should be done in a suitable container with heavy duty slow speed drilling machine with paddle attachment. Mixing should be made for 2 to 3 minutes to obtain a uniform colour.

Injection of Epoxy

For smaller area or isolated crack a hand pump may be used for injection. Hydraulic pumps, paint pressure pots, or air-actuated caulking guns can be used for larger cracked areas. The pressure should be selected carefully, because too much pressure can extend the existing cracks and cause more damage. If cracks are clearly visible, injection ports can be installed at appropriate interval by drilling directly into the crack surface. The surface of the crack between ports is allowed to cure. For vertical cracks, pumping of epoxy into the entry port should start at the lowest elevation until the epoxy level reaches the entry port above. Then the lower injection port is caped and the process is repeated at successively higher ports until the crack has been completely filled in. For horizontal cracks, injection starts from one end of the crack to the other in the same way. When the pressure is maintained, the crack is filled completely. For injection from underside of ceiling of flat roof a lot of pressure is being exerted. Hence care should be taken while injecting from underside. Figure 5 shows epoxy injection in ceiling surface of a roof slab.

Figure 5. Epoxy injection in ceiling surface of a roof slab

Removal of the Surface Seal

After the injected epoxy has cured, the surface seal is being removed by grinding or some other appropriate means. Fittings and holes at entry ports should be painted with an epoxy patching compound.

Advantages of epoxy injection

The various advantages of epoxy injection grouts are as follows:

Epoxy resins cure to form solids with high strength and relatively high moduli of elasticity.

- It can be applied on negative side.
- It can penetrate deep into the cracks
- It does not cure very quickly particularly at low temperatures
- It can have better bond with dry surface than wet surface.

4.3.4 Structural strengthening by jacketing with micro concrete

Repair to damaged reinforced concrete elements like beams, columns, wall etc., where excessive damage has occurred due to earthquake can be repaired and strengthened by jacketing with micro concrete. Dr. Fixit micro concrete can be used for structural strengthening.

The step by step approach for repair& strengthening by jacketing with micro concrete is given as follows:

Supports

The structural members should be properly supported before chipping the spalled / loose concrete. The props provided shall be adequate to provide sufficient structural support to the load carrying members (Figure 6).

Figure 6. Props for micro concrete

Surface Preparation of concrete

All the spalled cracked concrete or any other pre-applied mortar shall be removed by chipping to expose the reinforcing bars. The concrete shall be chipped to a minimum depth of 10mm behind the reinforcing bars. The areas to be repaired shall be profiled to get rectangular or square shape with an inward tapering edge.

Surface preparation of reinforcement

The exposed reinforcing bars should be cleaned thoroughly to remove all traces of rust, scales, etc., by using wire brush, emery paper etc. The lateral ties/stirrups shall also be cleaned in the same way. After removal of corroded portion, the diameter of the reinforcement shall be checked and compared with the drawings.

Provision of additional reinforcement

As the diameter of reinforcing bars is reduced substantially (say >20%) additional bars shall be provided as per the design. This additional reinforcement shall be properly anchored to the existing concrete by providing adequate shear connectors (Figure 7). Weld mesh may also be provided if found necessary along with additional reinforcement (Figure 8).

Provision of shear connectors

Shear connectors of 8mm diameter shall be provided in holes of 14mm diameter and 75 mm deep. These shall be provided at every 500 mm c/c on all the faces of the beams in staggered form. The holes shall be cleaned with compressed air or water jet to remove all the dust etc. and then the shear connectors (Figure 7) shall be fixed in the holes using polyester resin anchor grout.

Priming of reinforcement bars

Figure 7. Additional reinforcements and shear connectors provided

The exposed and cleaned reinforcing bar shall be provided with a coat of epoxy zinc primer such that the coated film will have a dry film thickness of 40 microns. The film shall be continuous especially in the regions where pitting, imperfections etc., are present on the surface of the bars. It is important that the rear portion of the bars should not be left without coating. A second coat if needed may be provided to achieve a uniform and continuous film. The additional reinforcement provided and also the shear connectors shall be coated with epoxy zinc primer. The weld mesh if provided shall also be coated with epoxy zinc primer.

Provision of Epoxy based bonding agent

The base and hardener component of epoxy resin based bonding agent must be mixed well to get a uniform grey coloured mix. Apply the material to properly cleaned and dry concrete substrate using stiff nylon brush by scrubbing it well into the substrate. The coat should be uniform and well spread on the entire surface area of the repair patch. The mixed material must be applied before the elapse of its pot life and the new repair mortar must be applied before the elapse of overlay time. As a fully dried epoxy resin coat acts as debonding layer, the repair material should be applied whilst the bonding coat is tacky. In case the applied epoxy bond coat gets dry, an extra coat should be applied before application of repair mortar.

Formwork and shuttering

Slurry tight and strong form work shall be provided. The shuttering for encasement shall be kept ready such that the formwork shall be placed in position and fixed such that the micro concrete can be poured into the formwork within the overlay time of the bonding agent (5 hours). Adequate supports shall be provided for the formwork. Care should be taken to ensure leak proof shuttering (Figure 9). Under no circumstance the slurry should flow out of the shuttering during pouring of micro concrete.

Mixing of micro concrete

It should be mixed using the appropriate water powder ratio as mentioned in the product data sheet. The mixing shall be done mechanically and under no circumstance hand mixing shall be done. Mixing shall be carried out for 3 to 5minutes to ensure that homogeneous mix is obtained without any bleeding or segregation. In hot climate ice cooled water shall be used to maintain the temperature of mixed material. If the encasing thickness is more than 100 mm, add stone aggregates up to 50 % by weight of micro concrete to the mixed micro concrete directly into the mixer hopper. The stone aggregates must be 10 mm and down and shall be clean, washed and dried. The mixing should be done for 3 minutes in mixer and then pre weighed stone aggregates into the mixer. Mix further for 2 minutes till lump free mix is obtained.

Figure 8. Additional reinforcements provided for jacketing

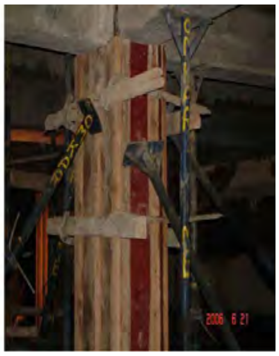

Figure 9. Strengthening of column with micro concrete

Deshuttering

The shuttering from the sides of the R C members shall be removed after a period of 24 hours. However, the formwork of the soffit shall be retained and removed after 3 days.

Pouring of micro concrete

The mixer should be poured into the formwork using a suitable funnel or through a hose pipe. It must be poured from one end only. A suitable hopper / funnel arrangement shall be made at site to facilitate the pouring operations(Figure 10 & 11). The pouring operation shall be continuous and it shall not be stopped unless the job is completed. To achieve this sufficient mixers / drilling machines and work force shall be arranged at site.

Figure 10.Preparation for micro concrete of a column

Figure 11. Repair of a column with micro concrete

Curing

All the repaired and encased area shall be fully cured as per standard concrete practices. Curing compound shall be used for effective curing of sides and soffits of beams. If a curing compound is applied, care shall be taken to ensure that proper surface preparation is carried out so as to remove any traces of curing compound on the surface. If this is not done, it may lead to debonding of any protective coating applied on top.

Figure 12. Preparation for micro concrete of a beam

Figure 13. Repair of a beam with micro concrete

V. CONCLUSION

For any nonstructural crack any cementitious, or polymer modified cementitious will be more suitable. The epoxy is the best material for injection in to cracks in structural members. But for densifying and treatment of honeycombs, the cementitious grouts will not only be suitable but also economical. The wider cracks need to be sealed with a Polyurethane sealant. But if the structural members are having excessive damage then jacketing with micro concrete is best suitable.

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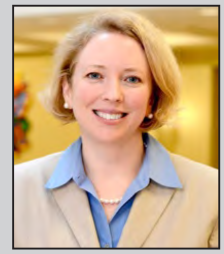
Guidelines for Preliminary Evaluation of Existing Rc Buildings

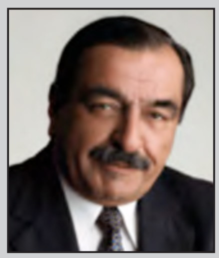
Santiago PUJOL
Professor, Lyles School of Civil Engineering, Purdue University, U.S.A.
spujol@purdue.edu

Prateek SHAH
Undergraduate Researcher, Lyles School of Civil Engineering, Purdue University, U.S.A.
shah151@purdue.edu



JoAnn BROWNING
Dean, College of Engineering, The University of Texas at San Antonio, U.S.A.

Michael KREGER
Drummond Endowed Chair in Civil Engineering, Department of Civil, Construction and Environmental Engineering, The University of Alabama, U.S.A.



Luis GARCIA
Professor, Department of Civil and Environmental Engineering, Universidad de los Andes Bogotá, Colombia

Steven MCCABE
Group Leader, Earthquake Engineering Group, National Institute of Standards and Technology, U.S.A.

ABSTRACT

American Concrete Institute (ACI) committee 133 commissioned the writers to visit Kathmandu after the 2015 April Earthquake. The writers worked with local engineers and Department of Urban Development and Building Construction (DUDBC) to inspect 176 buildings. Using these observations, and in collaboration with ACI 133, guidelines for seismic evaluation were produced. The guidelines are reproduced here hoping the readers may have the time to share their reactions with the writers.

I. INTRODUCTION

If a building survives an earthquake, one cannot conclude that it will also survive other (future) earthquakes. Even earthquakes produced by the same geologic faults may cause different ground motions at a given site. For this reason, to assess damage caused by ground motion is not the same as assessing the likelihood that a building is safe now and in the future.

The guidelines reproduced here were conceived:

1. Realizing that they needed to be simple because Kathmandu has too many buildings in need of evaluation,
2. On the basis of quantitative observations made by Hassan, Dönmez, O'Brien, Zhou, in Turkey, Haiti, and Wenchuan (Hassan & Sozen, 1997; Dönmez & Pujol, 2005; O'Brien, et. al., 2011; Zhou, Zheng & Pujol, 2013),
3. Using the experiences of ACI committees 133 and 314, With the expectation that the user of the guidelines understands their limitations,
4. Understanding that much of the damage in taller RC buildings in Kathmandu was related to problems with partitions instead of problems with the structures,
5. Realizing that the stiff but brittle brick partitions used in Kathmandu play two contradicting roles: a) they help control drift, b) they fail at small drifts. Using different

partitions could be an option, but the engineer must evaluate first to what extent this change may cause increases in drift demand that may compromise the structure.

6. Knowing that there are other alternatives for evaluation and retrofit, but Kathmandu is in need of guidelines that are simple enough to avoid inaction because of lack of resources.

II. GUIDELINES FOR PRELIMINARY EVALUATION OF EXISTING RC BUILDINGS

1. If the building has severe structural damage (evidenced by exposed reinforcing bars and/or cracks with thicknesses exceeding 3mm) in structural walls or columns, the building should be given high priority in an evaluation or strengthening schedule coordinated by the responsible local building authority. Inclined cracks of any thickness should be classified as severe damage in columns with transverse reinforcement ratios not exceeding the larger of r_{min} and 0.3%:

Eq. 1 $$r_{min} = 1.5\frac{\rho \times h}{H_{col}}$$

r_{min} = minimum transverse reinforcement ratio (cross-sectional area of hoops and ties divided by product of width –dimension perpendicular to h– times hoop or tie spacing).

ρ = longitudinal reinforcement ratio (total cross-sectional steel area divided by gross cross-sectional area)

h = larger cross-sectional column dimension. Use the same units for both h and H_{col}.

H_{col}: clear (unrestrained) column height (from slab top to beam bottom, or from sill top to spandrel bottom). Use the same units for both h and H_{col}.

2. If the building has fewer than eight stories:

2.1. Calculate the Column Index *CI*:

Eq. 2 $$CI = \frac{1}{2}\frac{\Sigma A_{col}}{\Sigma A_{floor}}$$

ΣA_{col} = Gross cross-sectional area of all columns in ground story

ΣA_{floor} = Total floor area above ground (approximately equal to typical floor area times number of stories)

Use the same units to calculate all areas.

2.2. Calculate the Wall Index *WI*:

Eq. 3 $$WI = \frac{\Sigma A_{RC\ Wall} + \frac{1}{10}\Sigma A_{Masonry\ Wall}}{\Sigma A_{floor}}$$

$\Sigma A_{RC\ Wall}$ = Gross cross-sectional area of all RC walls in ground story

$\Sigma A_{Masonry\ Wall}$ = Gross cross-sectional area of all clay masonry infill walls in ground story. Exclude walls with window and door openings.

ΣA_{floor} = Total floor area above ground (approximately equal to typical floor area times number of stories)

Use the same units to calculate all areas. The numerator should be calculated for parallel RC and masonry walls and for the direction producing the smallest value of *WI*.

2.3. Estimate whether the total cross-sectional area of elements resisting lateral loads exceeds the minimum described by Eq. 4:

If Eq. 4 $$CI < \frac{1}{250} - 2\,WI$$

then the building should be given high priority in an evaluation or strengthening schedule coordinated by the responsible local building authority.

3. If the building has eight or more stories:

3.1. Estimate first-mode building period T_o using gross-section properties and linear analysis based on structural mechanics. Do not use approximate expressions used in design of new buildings.

3.2 Calculate index *MDR*:

Eq. 5 $$MDR = 0.5\frac{m}{4} \times T_o \times \frac{1}{H}$$

H is the height from ground to centerline of roof beams (in meters).

3.3. If *MDR* exceeds 1.5%, the building should be given high priority in an evaluation or strengthening schedule coordinated by the responsible local building authority.

3.4. If *MDR* exceeds 0.5% and any of the following conditions <u>are not met</u>, the building should be given high priority in an evaluation or strengthening schedule coordinated by the responsible local building authority:

a) The longitudinal reinforcement ratio (Sec. 1) in all structural walls exceeds 0.5%.

b) The transverse reinforcement ratio (Sec. 1) in all structural walls exceeds 0.25%.

c) Lap splices at the base of any structural wall are longer than 40 bar diameters and are confined by 8-mm or larger diameter hoops or closed ties with a vertical spacing not exceeding 150 mm.

d) The volumetric ratio of transverse reinforcement (volume of steel divided by volume of confined concrete) within the outer fourths of the wall cross section exceeds 0.75%.

e) The transverse reinforcement ratio (Sec. 1) in all columns exceeds 0.3%.

f) All longitudinal reinforcement consists of standard deformed steel bars with a minimum yield strength of 250 MPa.

3.5. If *MDR* exceeds 0.5%, clay masonry and similar partitions and infill walls should be reinforced with Fe 250 (MPa) or stronger horizontal steel bars embedded in bond beams, mortar joints, or plaster. The ratio of steel reinforcement cross-sectional area to masonry gross cross-sectional area should exceed 0.25%. The clear distance between each reinforcing bar and masonry units should exceed one bar diameter. This recommendation is satisfied by reinforcing bars at midheight of mortar joints with thicknesses exceeding three bar diameters. The clear distance between each bar and the finished plaster surface should exceed the larger of two bar diameters and 15 mm. Reinforcement in plaster exposed to weather should be stainless or galvanized.

An equivalent reinforcing system such as textile reinforcement could be used instead of steel reinforcement if shown through tests to be as effective in controlling crack widths at drift ratios up to 1.5%. Welded-wire fabric with wire spacing smaller than 250mm should not be used as reinforcement.

3.6. In all cases, masonry walls (of any type) on the exterior of the building shall be reinforced with bars anchored in structural elements (RC columns, walls) unless the masonry wall has no openings and is confined by structural elements along its entire perimeter. Anchorage should be provided through a hook, an embedment of at least twenty bar diameters or a mechanical device such as a steel plate or head.

4.1. If the building has a single group of elevator or stairwell RC walls on its periphery or more than half of its structural walls in one direction are near one of the building edges, the building should be given high priority in an evaluation or strengthening schedule coordinated by the responsible local building authority.

4.2. If more than half of the masonry walls (of any type) in one direction are near one of the building edges in any story, the building should be given high priority in an evaluation or strengthening schedule coordinated by the responsible local building authority.

5. If the building has captive columns, the building should be given high priority in an evaluation or strengthening schedule coordinated by the responsible local building authority. A captive column is a column constrained against lateral displacement along part of its floor-to-ceiling height (by a parapet, window sill, or deep spandrel for instance). A column constrained by a masonry infill wall with a window opening should be classified as a captive column if the distance *x* from column face to window is shorter than twice the distance *y* from window sill to beam bottom (Figure 5.1).

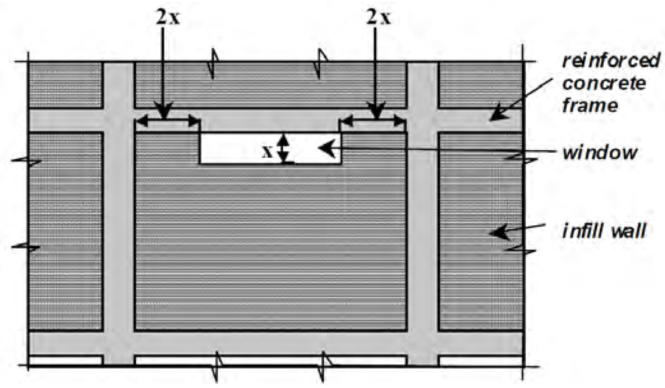


Figure 5.1 Dimensions related to recommended definition of captive column *(Based on ACI 314)*

6. If the total cross-sectional area of structural walls in one direction for a given story is 20% smaller than in the story above or the story below, the building should be given high priority in an evaluation or strengthening schedule coordinated by the responsible local building authority.

7. If the total cross-sectional area of masonry walls (of any type) in one direction for a given story is 30% smaller than in the story above or the story below, the building should be given high priority in an evaluation or strengthening schedule coordinated by the responsible local building authority.

8. Buildings with significant framing irregularities should be given high priority in an evaluation or strengthening schedule coordinated by the responsible local building authority.

III. CONCLUSION

If a building gets damaged in an earthquake, it is clear that the same building may get damaged again in a future earthquake. What is often ignored is the fact that if a building survives an earthquake without damage, one cannot conclude that it will also survive a future earthquake in good shape. All earthquakes are different. For this reason, to assess damage caused by ground motion is not the same as assessing the likelihood that a building may remain safe during a future (unknown) ground motion. The guidelines by ACI 133 reproduced here were meant to help Kathmandu evaluate as many of the buildings that survived the 2015 April earthquake as possible.

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Safe Anchor Designing in Structural Retrofitting

Prashant ANAND
National Technical Manager at Hilti India P. Ltd.,
B.E. (Civil), MBA
Email: prashant.anand@hilti.com

ABSTRACT

Retrofitting of RCC done by steel or concrete jacketing methods need steel elements (plates, angles, flats etc) to be *anchored* or *rebars grouted* to parent concrete. These anchors are used to transfer intended loads to the base concrete. Proper *selection, design and usage* of anchors is essential to avoid unintended failures. The failures can be in form of concrete breaking or splitting or anchor simply coming out in future. Currently there are no mandatory codes on "anchor designing" and "post-installed rebar grouting" but codal guidelines as well as state of art research exist which must be used to ensure safety.

I. INTRODUCTION

After the devastating earthquakes in Nepal last year, many of the buildings and other structures would need retrofitting as a remedial measure. Equally important is proactively retrofitting structures which may not be safe as per the current seismic code requirements. The article tries to introduce the reader to the basics of post-installed anchoring and post-installed rebar grouting (rebarring) which are extensively used in retrofitting methods. It's imperative for engineers to know the following before making recommendations:

- What kinds of anchor (also called fasteners) exists, how do they work and how to select right type for any specific case.
- How to design capacity of anchors to ensure no failures.
- What are the must DOs and DONTs while executing the job.

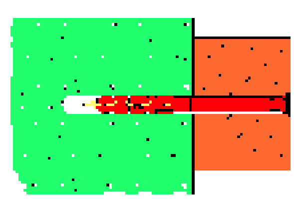
The article is

II. ANCHORING & REBARRING TECHNOLOGY BASICS

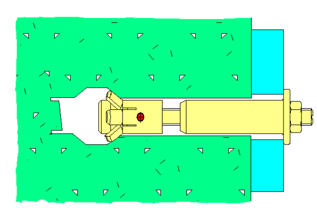
How do anchors work?

Anchors work on any or combination of the 3 working principles:

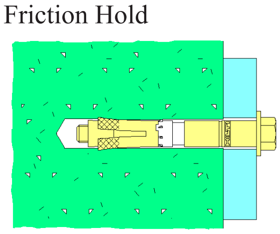
- Friction hold: anchor sleeve presses against concrete to create friction hold (all plastic anchors used in concrete)
- Keying hold: anchor sleeve cuts into concrete and expands inside to create hold
- Combined friction and keying hold: most mechanical expansion anchors work on this principle
- Adhesive or chemical bonded anchors: chemical adhesives are used to create bond between the steel element and concrete



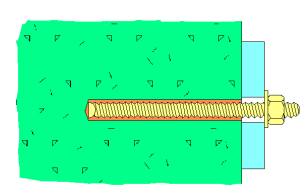
Friction Hold



Keying Hold



Combined



Adhesive

Basic selection guidelines

The right selection of anchors depend on many criteria which include condition of concrete, size of concrete, load to be transferred, nature of load and finally cost involved. The below table give some 'rule of thumb' guidelines for easy selection

	Anchor Types			
Application Requirement	Friction	Keying	Comined Friction & Keying	Adhesive
Load capacity	Light	Heavy	Medium	Medium- Heavy
Fixing in weak (< M20 grade) concrete	X	0	X	Y
Usage in small concrete members (eg. 9"X9" column)	Y	X	X	Y
High seismic load transfer	X	Y	0	0
Average costing	Very economical	Medium - Expensive	Medium	Medium

X Not Usable Y Preferred 0 Use with caution

What is anchor designing and why is it necessary?

Anchor manufacturers publish the anchor capacities under a set of conditions eg. Type XYZ anchor has 10kN capacity in M25 concrete grade when not in group and not close to concrete edge. The actual condition at site would almost always be different. Say, we may want to fix a plate with 4 anchors on a column face where inputs are- M20 grade concrete, group of 4 anchors with spacing of 200mm vertically and 150mm horizontally, anchors placed 75mm from concrete edge. As conditions vary, the actual capacity of anchors would vary from the published loads. Thus capacity of 4 anchor < 4X10=40kN. Clear guidelines are laid out to calculate the capacity under changing inputs which has to be calculated.

Only through designing can final recommendation be made which has to show:

- Type of anchor
- Diameter and embedment depth required
- Layout of anchor

What are applicable codes and guidelines?

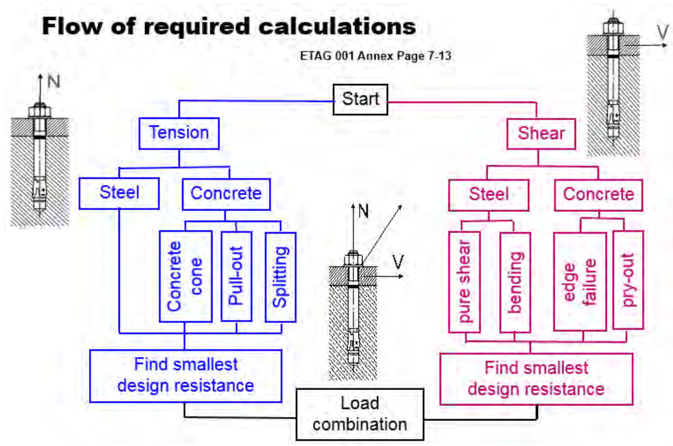
European Organization for Technical Approval (EOTA) and American Concrete Institute (ACI) have laid out design and anchor selection guidelines. The current design method known as Concrete Capacity Design (CCD) below is further detailed in the table below.

Design Type	ACI		ETA	
	Mechanical	Chemical	Mechanical	Chemical
Static Anchor Design	ACI 318 App D	ACI 318 App D; AC 308	ETAG 001 Annex C	EOTA-TR 029
Seismic Anchor Design	ACI 318 App D	ACI 318 App D; AC 308	EOTA-TR 045	EOTA-TR 045
Rebarring	NA	No Guidelines	NA	EOTA-TR 023

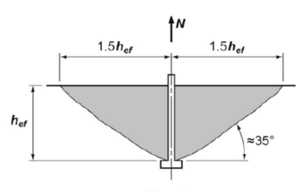
ETA guidelines are available at www.eota.eu

What checks are done in the designing?

Anchors when overloaded may fail in one of the various possible failure modes. Some failures will happen to individual most stressed anchor while some failure are dependent on group behavior. A designer has to ensure that there is no possibility of failure within design load being transferred to anchors. The below flowchart provides the failure checks which need to be done in designing.



Any tensile load applied to anchor is transferred again as direct tension to concrete. As concrete is not very strong to take tension force, *concrete failure modes like breakout of splitting are generally most critical failure modes.* The capacity for concrete failure are dependant on anchor layout (spacing and edge distance), embedment depth and concrete grade.



Ideal concrete cone formation Actual cone formation with overlaps

Technical parameters of anchors (load capacity, minimum spacing and edge distance requirement, fixing paramerters like drill dia & depth, torque to be applied etc) vary for each anchor type. Unlike building materials like rebars or cement whose specifications are standardized as per the existing codal requirements, anchors don't have any standardized parameters. Thus all details are published by the manufacturers which should be used for design and adhered to for fixing.

Manuals published by manufacturers like Hilti can be used by engineers. Alternatively, anchor design or rebarring design software can be used. Using software has the advantage of faster calculation, easier selection and avoiding errors of breaching boundary conditions.

Hilti design software can be downloaded free of cost from https://www.hilti.in/engineering/Design-and-Software/software

DOs and DONTs in anchor designing

- Don't suggest anchors just based on individual capacity as published by manufacturer
- Optimize group capacity by changing layout (increasing spacing and edge distances) wherever possible. This will often enhance capacity more that increasing number or diameter of anchors
- Check every failure mode- a design passes only if capacity is higher than load for every failure mode
- Better err on safer side- be conservative on assuming concrete grade if it's not known

Provide clear anchor details and layout on drawings

DOs and DONTs during anchor fixing

- All anchors must be loaded simultaneously else some anchors can be subjected to additional loading resulting in failure
- When using chemical anchors, transfer loads only after full curing of chemical. Also anchors must not be touched after 'gel time'.
- Proper installation procedure as provided by manufacturer must be followed. It includes using specific diameter drill bits, drilling depth, torque to be applied etc.
- Follow the layout as per the drawing from designer. In case the same is not possible, alternate layout shall be vetted by designer.
- It is advisable to use concrete scanner to find rebar locations when drilling in heavily reinforced members like column so that rebars are not hit during drilling.

III. RECOMMENDATIONS

Engineers should acquaint themselves about the basic design principles of anchoring/ rebarring when using them in retrofitting or other applications. As these are not part of national codes currently, ACI or ETA guidelines can be referred for designing. Special care has to be taken to check for possible concrete failure like concrete cone breakout. As current cast-in bolt design practice is often to check only for bond and steel capacity, the tendency to do the same has to be avoided.

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Seismic Strengthening of Reinforced Concrete Structures by Post-Tensioned Metal Straps (PTMS) Technique

Pramod Neupane
Urban Governance and Development Program (UGDP), Emerging Towns Project (ETP), Dhankuta Municipality Office, Dhankuta, Nepal
Email: pneupane0124@gmail.com

Dipendra Gautam
Structural and Earthquake Engineering Research Institute, Kathmandu, Nepal

Abstract

Seismic rehabilitation of structures is dragged attention of many researches worldwide and accordingly several techniques are developed till date. However the major shortcomings correlate with high cost, long time for construction and special knowledge and skill. This paper focuses on seismic strengthening of reinforced concrete structures by Post-Tensioned Metal Strap (PTMS) technique. By strengthening beam-column joints and critical elements using this technique, a considerable gain in strength and ductility of the whole structure is achieved. To evaluate seismic response and overall structural capacity, the results from a bare and strengthened frame tested under the BANDIT project were analysed using DRAIN-3DX analysis. The analytical models were studied under varying peak ground accelerations of 0.05 g to 0.35 g. From several analytical results and their correlation with experimental results from previous works, strengthening by PTMS technique is found to be efficient in increasing overall structural strength, stiffness and ductility of the RC structure. In the aftermath of Gorkha earthquake in Nepal, the RC structures sustaining minor damage could be effectively rehabilitated by PTMS technique in relatively lower cost assuring higher level of structural strength, stiffness and ductility.

Keywords: *seismic strengthening; PTMS technique; RC Structure; DRAIN-3DX.*

I. INTRODUCTION

After occupying for several years, substandard construction or due to dynamic loads imposed on structures, there will be subsequent deterioration in the strength, ductility and stiffness of RC structures. However, building rehabilitation may be largely deterred by the huge repair cost (Frangou 1996). Till date practices are not compliant with the demands of occupants in terms of economy and acceptable level of safety, thus people continue to use the structures for many years without repairing and thus lives and property loss during earthquakes becomes obvious in all seismic zones. Most of the available rehabilitation techniques are labor intensive, time consuming, prohibit use of structural whilst works are carried on and unaffordable (Pilakoutas and Dritsos 1992).

Thus it is imperative to develop an effective, easy and low-cost technology so that people could immediately incorporate in household level.

Strengthening by post tensioned metal straps (PTMS) is relatively new and innovative technology developed by The University of Sheffield, which tries to incoprorate the deficiencies of conventional technologies in terms of economy, time bound and cost reductions aspects. This technique can increase strength and stiffness of the existing structures, making them seismically strong enough to resist moderate earthquakes (Frangou 1996). Apart from this, the technology involves confining RC members by tightly strapping high strength metal straps throughout the critical joints in the structure. Frangou et al. (1995) concluded from experimental research that the PTMS strengthening technique can enhance member strength and ductility to a higher level than that possible by conventional reinforcement.

The PTMS technique is more pronounced for structures when it is required to increase the overall strength and ductility of a structure by increasing ductility of critical elements (Frangou 1992). This ultimately helps in energy dissipation in structures leading to reduction in seismic forces on elements (Frangou 1996). Lateral confinement is achieved by strapping elements with high strength metal straps which are commonly used in packaging industries. Strapping equipment apply stress in the strap while simultaneously clipping them in position by using special clips. Straps can easily bend around corners and transmit high strength and ductility to RC structures (Frangou 1992). The low cost of straps and easy operation techniques make this method of strengthening practical and it is preferred over other prevailing strengthening techniques.

Short splices strengthening in RC beams using PTMS technique is successfully implemented by Helal et al. (2014) inferring that the bond strength was significantly enhanced (upto 58%) due to use of PTMS confinement. Moreover, the full scale testing of substandard structure rehabilitated with PTMS technique sustained PGA of 0.35 g without any significant damage (Garcia et al. 2014) thus PTMS has been accepted as economic and efficient strengthening technique for RC structures nowadays (For details see: Frangou et al. 1995; Frangou 1996; Poulopoulou 2006; Moghaddam et al. 2008; Garcia et al. 2014; Helal et al. 2014; Garcia et al. 2015).

During 2005 Kashmir, 2008 China, 2009 Indonesia, 2010 Haiti and 2015 Gorkha (Nepal) earthquake, many of the RC building collapse was attributed by the inadequate behaviour of beam-column joints. The deficiencies in beam-column may be due to: a) lack of confinement in joint core, b) short anchorage length of bottom beam reinforcement in joint core and c) construction joints above/or below beam-column joints (Beres et al. 1996). In case of Gorkha earthquake in Nepal majority of the partly damaged RC structures are constituting damage in beam-column joint (Gautam et al. 2015; Gautam et al. 2016a) and are in dire need of strengthening. Due to Gorkha earthquake, 8,790 casualties and 22,300 injuries were reported. As many as 498,892 buildings were completely collapsed and 256,697 buildings were damaged due to main shock and strong aftershocks (NPC, 2015). As reported by the National Planning Commission, 6,613(1.7% of total damage) RC buildings are completely collapsed and another 16971(6.7% of damaged structures) are partly affected. However, seismic strengthening is broadly felt in public level across Nepal after the Gorkha earthquake and jacketing is commonly applied as seismic strengthening of substandard structures nowadays. Chaulagain et al. (2014) compared various strengthening solutions and justified that retrofitting could significantly enhance the performance of structures during earthquakes. Previous contribution from Gautam et al. (2016b) identified some of aseismic features in local buildings technologies which are economic though not widespread and do not in corporate present trend of RC construction. This paper highlights the efficacy and economy of PTMS technique so as to recommend Nepal for enhancing ductility, stiffness and structural strength for the substandard, affected and structures needing seismic improvement in long run as PTMS is never dealt before for strengthening solutions. The analytical results carried out in DRAIN-3DX platform and other experimental works performed in The University of Sheffield are presented.

II. Materials and Methods

Nepal needs some immediate efforts to strengthen many of the existing structures after the 2015 Gorkha earthquake. After 1980s, people in Kathmandu valley and other areas started practicing RC constructions and most of these structures are substandard constructions or are exposed to at least two big earthquakes of 1988 and 2015. As metal straps are very cheap and may be easily available in Nepalese market, moreover for this technology, not any sophisticated technology and skills are needed. This could be viable solution so as to improve the building performance and improving seismic safety of the damaged structures. By 1995, Frangou developed novel strengthening technique for RC beams and columns using Post-Tensioned Metal Strapping (PTMS) and later on The University of Sheffield has done a lot of works regarding seismic improvement of structures in terms of strength, ductility and stiffness. Figure 1 shows the general outline of the PTMS technology used for improving the beam-column joint.

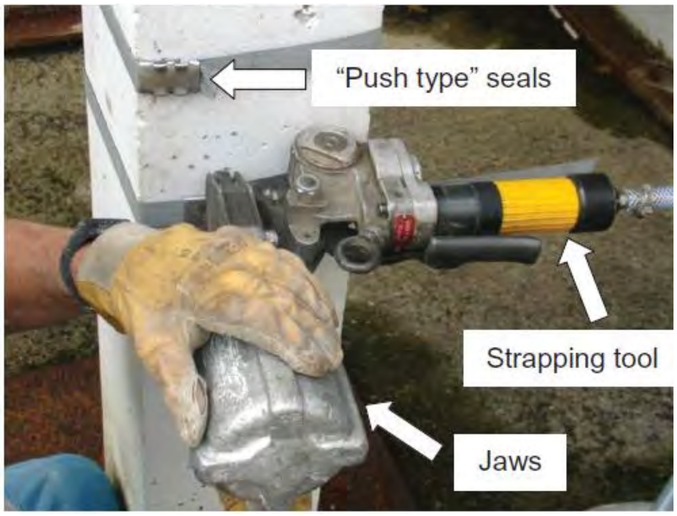

Fig. 1 Metal straps fixed in the beam-column joint (adopted from Garcia et al. 2014)

PTMS technique relies on the post-tensioning of high-strength steel straps around the RC members using hydraulically-operated steel strapping tools, which adhere with the packaging industry. Thereafter the straps are fastened using 'push-type' seals to maintain the tension force. Time history analyses are used for to compare and identify the performance of structure with incremental PGA range of 0.05 g to 0.35 g for each 0.05 g increment. The floor displacement records of strengthened frames are estimated in each analysis. During the nonlinear time history analysis, the deficient frame showed high deformation and was unable to withstand the load of more than 0.15 g PGA as suggested by the BANDIT experiment, however the strengthened frame are increased upto 0.35 g PGA and analyses re performed. The experimental results from BANDIT experiment are congregated and compared with the time history analyses results.

III. Results and Discussion

A comparison between maximum displacement records in the bare and PTMS strengthened building from the experimental results obtained from (Garcia et al. 2012) and analytical results obtained from DRAIN analyses are shown below in table 1. The maximum displacements give an overview of the global behavioural response of the structure under different peak level of ground accelerations.

Table 1 Maximum floor displacements obtained from BANDIT tests

PGA level	Floor no.	Bare building (mm)		PTMS strengthened building, (mm)	
		Experiment	Analysis	Experiment	Analysis
0.05g	2	17.5	16.1	20.5	20.76
	1	10.5	10.86	13.3	13.78
0.10g	2	44.7	45	52	48.72
	1	24.8	23.61	29.4	26.99
0.15g	2	81.8	74.7	78.9	66.5
	1	31.3	35.2	41.6	40.85
0.25g	2	-	-	125.9	110.23
	1	-	-	60.7	58.67
0.35g	2	-	-	162.3	146.57
	1	-		75.3	78.8

Table 2 shows comparison between analytical frequencies obtained from strengthened frame. First and second frequency modes obtained from analysis were tabulated to compare the frequency loss in bare and PTMS strengthened frame.

Table 2 Modal Frequencies for PTMS strengthened frame

Condition	First mode f_1 (Hz)		Δf_1	Second mode f_2 (Hz)		Δf_2
	Experiment	Analysis		Experiment	Analysis	
Initial	1.65	1.65	NA	5.01	5.01	NA
After PGA = 0.05g	1.56	1.53	-7.3	4.66	4.62	-7.8
After PGA = 0.10g	1.5	1.49	-9.7	4.59	4.57	-8.8
After PGA = 0.15g	1.47	1.47	-10.9	4.33	4.32	-13.8
After PGA = 0.25g	1.28	1.26	-23.6	3.9	3.8	-24.2
After PGA = 0.35g	0.99	0.97	-41.2	3.7	3.6	-28.1

Table 2 shows 41% loss in first modal frequency of strengthened frame obtained at fifth test of PGA level of 0.35 g, while 28% frequency loss in second mode. The frequency loss in strengthened frame at 0.35 g PGA level is nearly equal to that obtained with bare frame for 0.15 g. Results show significant reduction in frequency loss in second mode with strengthened frame. This shows that PTMS strengthening in beam-column joints have been able to regain overall stiffness of the structure effectively. Figures 1 and 2 show the comparison of displacement time history of analytical response with experimental time history at 0.05 g PGA. The analysis has moreover been able to predict the structural behaviour of the strengthened frame; however variations in peak responses were observed along with frequent phase differences. This could be because of loss in frame's strength and stiffness after first set of tests experimented with bare frame, where concrete cracks cured might not have been able to gain full strength. This could have altered damping in concrete, as a result, variations are observed.

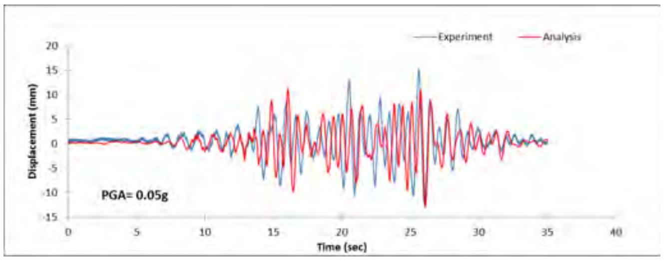

Fig 1. Time history records of first floor at 0.05 g

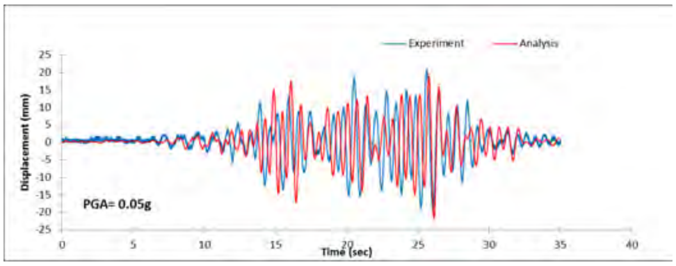

Fig 2. Time history records of second floor at 0.05 g

The analytical time history result of first floor at 0.10 g is shown in figure 3. Analytical result showed slightly stiffer response; however has followed experimental results with slight variance in phase. Similar response was observed in second floor displacement as shown in figure 4. Cracked concrete and degraded concrete capacity might have altered the damping level in concrete which showed variable responses in comparison with the experimental results. The analysis results showed correlation with the experimental results until the first 10 seconds and slight variations in peak response.

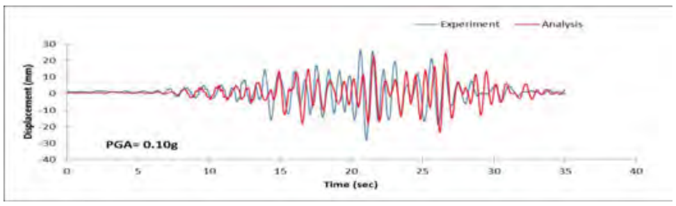

Fig 3. Time history records of first floor at 0.10 g

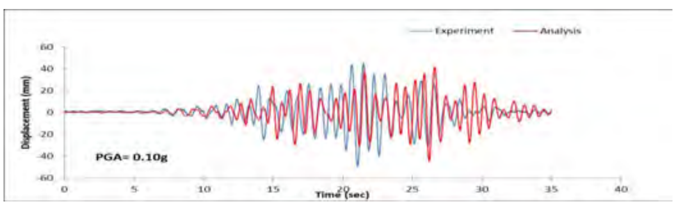

Fig 4. Time history records of second floor at 0.10 g

Figures 5 and 6 show the displacement time histories of first and second floor of the test frame at PGA level of 0.15 g. Very good correlation was observed between analytical and experimental results suggesting better analytical prediction of structural response. Slight difference in peak response of first floor at about 21 seconds was observed while rest of the responses were up to best correlation suggesting DRAIN analysis has predicted the response of the strengthened frame precisely at this PGA level. Precise analytical correlation with the experimental results shows that analysis has been able to predict the response of the strengthened frame.

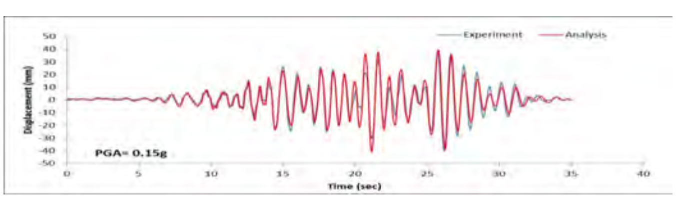

Fig 5. Time history records of first floor at 0.15 g

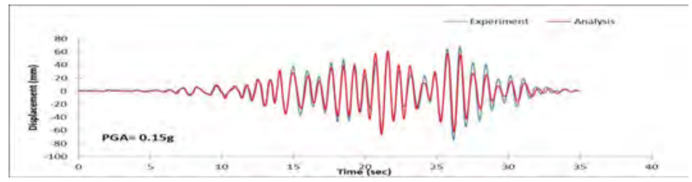

Fig 6. Time history records of second floor at 0.15 g

The analytical responses of frame at first and second floor are shown below in figures 7 and 8 at

0.25g PGA level. The analytical result has shown slightly stiffer response than the experimental result in the first floor, however the analytical result have followed the experimental results well. Slightly stiffer response in analytical results was obtained at first floor which improved in the second floor. Moreover, analytical results have been able to predict the response of the structure.

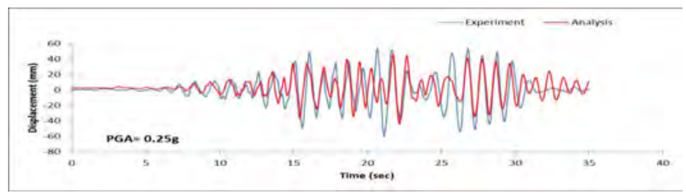

Fig 7. Time history records of first floor at 0.25g

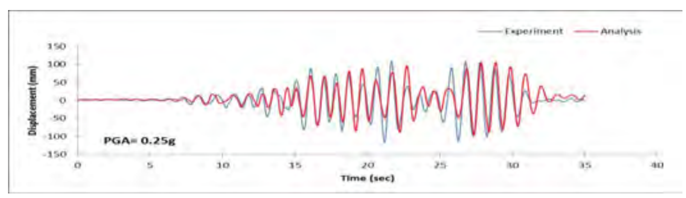

Fig 8. Time history records of second floor at 0.25g

Figures 9 and 10 below show the time history displacements for first and second floor respectively at 0.35 g. The analytical results show correlation with experimental results. This signifies the confinement level achieved in the experimented frame has been predicted well by the analysis. Moreover the analysis has been able to predict the structural response of the strengthened frame.

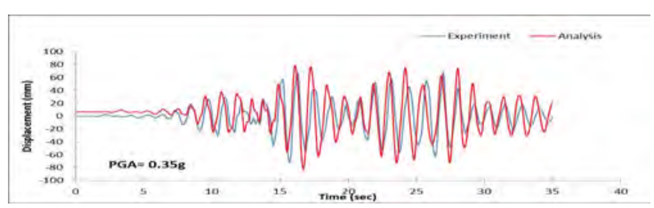

Fig 9. Time history records of first floor at 0.35g

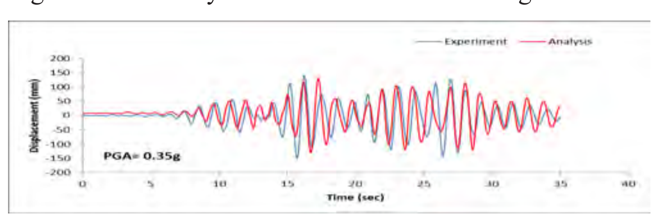

Fig 10. Time history records of second floor at 0.35 g

IV. CONCLUDING REMARKS

After the BANDIT project, PTMS technique for seismic strengthening is gaining attention in terms of low cost and viable solution in terms of structural performance evaluation even in exposure to large PGA (upto 0.35 g). In order to simulate behavioural response of strengthened frame and to detect the significance of proposed PTMS strengthening technique, frame experimentally tested under the BANDIT project was analysed using time history approach. The frame had structural deficiencies in beam-column joint and column splice. This deficient frame was first analysed to get the behavioural responses of bare frame and later re-analysed by introducing PTMS intervention at deficient joint regions to obtain response of strengthened frame. The comparison between first and second modal frequencies showed significant gain in stiffness in the strengthened frame. This was evidenced by the frequency loss in first mode of bare frame at 0.15 g was 44%, which was equal to the first modal frequency loss of strengthened frame at 0.35 g PGA.

Further, Strengthened frame showed 28% loss in frequency of strengthened frame while 40% loss was obtained in bare frame. From the analytical results obtained from PTMS strengthened frame and their comparison with the level of performance of bare frame, significant enhancement in structural strength, stiffness and ductility was achieved. This is attributed to the active confinement of post tensioned metal straps in the beam column joints. Thus, seismic strengthening of deficient RC frame was successfully achieved by PTMS strengthening technique. PTMS relies on very simple technology and even metal straps conventionally used for packaging industry could be used, the substandard structures existing in Nepal after Gorkha earthquake can be effectively strengthened using this technology. In addition to this, structures that sustained considerable damage and are weakened in joints may need serious repairs and conventional retrofitting techniques may not assure the required level of seismic safety and performance, however from the results of BANDIT project and analytical approaches it is well justified that the structural performance will be well assured in low cost-which is direly needed for Nepal as almost 80-90% of RC structures in Nepal are pre-engineered constructions and primarily compromised in terms of assuring ductile behaviour. PTMS technique could be undoubtedly useful for assuring seismic safety of structures

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Testing Fibre Stabilisation for Earthquake Resilience of Earth Mortar in Stone Masonry Construction

Dr Martin HEYWOOD
Senior Research Fellow; Faculty of Technology, Design and Environment; Oxford Brookes University UK
Email: mheywood@brookes.ac.uk

Charles PARRACK
Senior Lecturer Faculty of Technology, Design and Environment; Oxford Brookes University UK

Dr Bousmaha BAICHE
Research Fellow; Faculty of Technology, Design and Environment; Oxford Brookes University UK

Loren LOCKWOOD
Shelter & Housing Technical Advisor, Catholic Relief Services

Jamie RICHARDSON
Shelter & Housing Technical Advisor, Catholic Relief Services

ABSTRACT

Earth mortar stone masonry construction in seismically active zones present a risk to life. This paper documents a laboratory mortar test which demonstrates that fibre reinforcement of the mortar is likely to have some benefit in terms of the earthquake resilience of the masonry, by improving the structural integrity of the building and reducing the risk of total collapse during a seismic event. Further research is needed to quantify the required reinforcement density and to provide guidance on achieving the required density and distribution in practice.

Key words: Earthquake Resilience Earth mortar masonry

I. INTRODUCTION

Background

Stone masonry buildings with low strength mortar are particularly vulnerable to damage and collapse during seismic activity. This risk becomes more acute when unshaped stones are used and skilled craftsmen are not available. The Earthquake Engineering Research Institute (USA) identifies 'economic constraints and lack of proper training for local artisans' (Bothara & Brzev 2012, p15) as contributory factors to this vulnerability.

In response to the 2015 earthquake in Nepal, Arup engineers commented (Arup 2015 pers. comm. 15 December) on the challenges for rebuilding

'The most challenging places for rebuilding seem to be the remote and often impoverished areas where better materials are both too costly and difficult to transport to the sites. Here, various types of local stone and mud mortar (and sometimes timber) have been used for construction. In addition, there are many site related challenges: site amplification of ground motions, landslides, rock falls, irregularity in the structure created sloping sites and localized seasonal flooding.

As was demonstrated in the recent earthquake, stone masonry typically performs very poorly under earthquake loading and poses a significant risk to Life Safety. Ideally, we would avoid this type of system in high seismic hazard areas such as Nepal as even if the structure remains standing, dislodged bits of stone during the earthquake can still injure or kill the occupants.' (Arup 2015 pers. comm. 15 December)

Guidelines (for example: Arya, Boen, & Ishiyama, 2014; Bothara & Brzev 2012) and standards (for example: Government of Nepal 1994) offer a number of interventions which are able to increase seismic resistance of stone masonry construction, but this paper focuses on communities, unable to afford even modestly priced materials, and may not have access to stone masonry skills. It poses the question of whether vulnerability can be lessened for this group by addition of fibrous materials to earth mortar.

Strength of earth mortar can be increased by a process termed 'stabilising'. Additives such as 'ash, lime, cement, fibres, or cow dung' (Bothara & Brzev 2012, p49) are used in this way. Whilst these all increase strength of the mortar, this does not necessarily increase seismic resistance:

The use of cement or cement/lime mortar has been recommended by various codes and guidelines. A recent research study by Ali et al. (2010) has shown that use of cement mortar does not necessarily lead to improved seismic resistance of stone masonry buildings unless earthquake- resistant provisions are also incorporated (Bothara & Brzev 2012, p48)

No data was found on the seismic resistance behaviour of fibre stabilised earth mortar, therefore a series of laboratory tests were proposed to investigate this quality.

Fibres commonly used in earth mortar include straw, hay, hemp, sisal, and elephant grass (Bothara & Brzev 2012). This type of fibre was tested, along with man-made fibres which might commonly be available for low or no cost in a post disaster context: rope and tarpaulin.

Aim and Purpose

This paper presents the results of a series of tests undertaken on mortar samples. The aim of the research was to investigate the impact of various forms of reinforcement on the strength and integrity of mortar in traditional buildings (stone and mud mortar) and to present recommendations on how the mortar may be improved in practice.

Test samples were provided in grey cement mortar and traditional mud mortar with the following types of reinforcement:

- None (control)
- Rope
- Straw
- Shredded tarpaulin

The tests were performed by the Architectural Engineering Research Group, Oxford Brookes University UK under the supervision of Mr Ray Salter.

II. CORE OF THE DELIBERATION

Test apparatus and procedure

The aim of these tests was to determine the shear resistance of mortar samples with and without reinforcement. The tests were undertaken in the Lloyd compression testing machine on 350mm x 100mm x 25mm (approx.) samples supported on 100mm square steel blocks, such that the samples spanned approximately 150 mm between supports. The load was applied at a steady displacement rate at mid-span through a 100mm x 100mm steel bearing. The aim was to produce shear failure in the samples, although in practice failure was likely to be a combination of shear and bending (due to the geometry of the samples). The test set-up is shown in Figure 1.

- Two types of test were undertaken:
- Test to initial failure (drop in load)
- Test to total failure (sample breaks in two)

The aim of the first test was to determine the load and associated deflection at which the mortar failed in shear through cracking. This corresponds to the point at which the mortar joint may no longer be serviceable in the long term, but retains sufficient integrity to prevent collapse of the wall. During this test, the displacement of the bearing relative to the supports was increased at a steady rate (0.25 mm/min) until a reduction in load was observed. In the case of the unreinforced control samples, collapse of the test specimen occurred at this point.

Figure 1. Test set-up
Figure 1. Photograph of testing apparatus:
Source Oxford Brookes University Laboratory

The aim of the second test was to determine the degree of structural integrity provided by the reinforcement after the initial failure of the mortar. Following completion of the initial test, each sample was removed from the testing machine, inspected and photographed and then reinstalled into the machine. Load was applied for a second time until total failure was observed, at which point the load reduced rapidly to zero and the sample broke into two pieces. In some cases, where the quantity of fibres was sufficient to prevent separation of the two halves of the sample, the test was terminated at a maximum displacement of 10 mm.

Note:

The following notation is used in the test references:

C = control (no reinforcement)

R = rope

S = straw

T = tarpaulin

Test results and analysis

Prior to testing, the samples were inspected for damage and photographed. The initial condition of the test specimens is shown in Figure 2.

Figure 2. Photograph of Initial condition of samples prior to testing. Source Oxford Brookes University Laboratory

It is apparent that the earth mortar specimens were in noticeably worse condition than the normal mortar.

The results of the initial shear tests are presented in Table 1.

Table 1. Initial shear tests Source Oxford Brookes University Laboratory Mortar test report (Heywood 2015)

Test reference	Maximum force (N)	Movement at max force (mm)	Shear stress (N/mm^2)[1]	Comment
C1/grey	-	-	-	Already failed
C2/grey	-	-	-	Failed during set-up
C3/grey	134	0.46	0.050	Broke in two pieces
R1/grey	103	0.18	0.039	Fibres still holding
R2/grey	97	0.17	0.036	Hairline crack only
R3/grey	141	0.38	0.056	Cracked but still one piece
S1/grey	110	0.44	0.043	Hairline crack only
S2/grey	114	0.31	0.044	Hairline crack only
S3/grey	100	0.30	0.037	Hairline crack only
T1/grey	103	0.25	0.039	Cracked but still one piece
T2/grey	138	0.61	0.053	Broke in two – few fibres
T3/grey	252	2.55	0.097	No reduction in force[2]
C1/red	-	-	-	Already failed
C2/red	176	0.50	0.064	Broke in two pieces
C3/red	-	-	-	Failed during set-up
R1/red	107	0.65	0.038	Cracked but still one piece
R2/red	221	0.72	0.073	Cracked but still one piece
R3/red	-	-	-	Already cracked through
S1/red	114	0.47	0.040	Hairline crack only
S2/red	100	0.79	0.036	Load increased to plateau
S3/red	114	0.32	0.041	Hairline crack only

T1/red	121	1.52	0.044	No reduction in force[3]
T2/red	-	-	-	Already cracked through
T3/red	-	-	-	Already cracked through

Notes:

1. The shear stress value was obtained by dividing the maximum force by the measured cross-sectional area.
2. Test T3/grey was loaded to the maximum applied force of 200N (plus bearing weight of 52N) with no reduction in load or sign of failure. It was only apparent that a crack had occurred on removal of the specimen from the testing machine.
3. Test T1/red was loaded to the maximum displacement of 1mm with no reduction in force. The test was then repeated towards a 2mm maximum displacement, but was stopped once a crack was spotted.

In most cases, initial failure resulted in a crack through the depth of the sample, leaving the two halves of the sample held together only by the fibres and hinged in the middle. In some cases, however, the crack was only hairline in nature and the test sample remained intact as a single unit. Test samples with sparse fibres were no better than the unreinforced controls and broke in two. With the exception of T3/grey and R2/red, there was little apparent improvement in shear strength due to the inclusion of the fibres. It was, however, apparent that the fibres provided a significant degree of resilience that allowed some samples to remain intact after initial failure.

In order to determine the ultimate failure resistance of those samples that had not already been broken in two, they were reinstalled in the testing apparatus and loaded to collapse. The results of these tests are presented in Table 2.

Table 2. Ultimate failure tests Source Oxford Brookes University Laboratory Mortar test report (Heywood 2015)

Test reference	Maximum force (N)	Movement at max force (mm)	Maximum movement (mm)	Comment
R1/grey	389	8.97	10.00[1]	Some fibres still attached
R2/grey	465	6.10	10.00[1]	Many fibres in centre
R3/grey	131	3.90	6.00	
S1/grey	148	4.63	7.30	
S2/grey	310	1.43	5.35	
S3/grey	234	3.87	7.62	
T1/grey	131	3.42	7.57	
T3/grey	296	3.57	10.00[1]	Many fibres in centre
R1/red	352	11.42	11.42[2]	Loaded twice but no failure
R2/red	445	7.07	10.00[1]	
R3/red	221	7.73	10.00[1]	
S1/red	186	2.89	6.67	
S2/red	141	1.49	5.08	
S3/red	183	2.80	9.10	
T1/red	269	9.36	10.00[1]	Many fibres in centre
T2/red	110	1.82	4.80	Some fibres still attached
T3/red	79	2.14	2.45	Some fibres still attached

Notes:

1. Test stopped automatically at 10mm without failure of the sample.
2. Test stopped automatically at 10mm without failure of the sample. Test then resumed and sample loaded to a maximum applied force of 300N (plus bearing weight of 52N) with no reduction in load or sign of ultimate failure.

In most of the cases, the load increased to a maximum before falling to zero, at which point the remaining fibres either failed in tension or pulled out of the mortar. In a few cases, however, the test was stopped automatically at a pre-set displacement limit of 10mm before the load had reached zero. In one case, the load continued to increase until the test was stopped automatically at an applied force of 300N (plus the bearing weight of 52N). It was noted that the key factor that determined whether the samples broke in two or remained intact to 10mm displacement was the density of fibres in the central region of the sample. Samples with sparse fibres or those with fibres only on the top or bottom surface of the sample did not perform as well as those with many fibres located at mid-depth. The type of reinforcement and even the type of mortar had little impact on the performance of the samples.

After completion of the test programme, the samples were photographed for a second time. The final condition of the test specimens is shown in Figure 3.

Figure 3. Photograph of final condition of test samples Source Oxford Brookes University Laboratory

III. CONCLUSIONS and RECOMMENDATIONS

A programme of shear tests has been undertaken on mortar samples with and without reinforcement. Significant variability in the initial resistance (when the samples first cracked) and ultimate failure was observed, but there was no clear distinction between the three types of reinforcement and two types of mortar. The performance of the samples between initial cracking and ultimate failure was largely dependent on the density of the reinforcement fibres and their location rather than the type of fibre or mortar. Samples with sparse fibres or those with fibres only on the top or bottom surface of the sample did not perform as well as those with many fibres located at mid-depth.

Although the reinforcement did little to improve the shear resistance of the mortar, the improved post-cracking behaviour is likely to have some benefit in terms of the earthquake resilience of the masonry, by improving the structural integrity of the building and reducing the risk of total collapse during a seismic event. Further research is needed to quantify the required reinforcement density and to provide guidance on achieving the required density and distribution in practice. Any further testing should be undertaken on cylinder samples and ultimately on small masonry panels subjected to dynamic loading.

The ultimate aim for this programme of testing is to promote safer construction to reduce risk of injury, death, and loss of property, using locally available and sustainable skills and materials. Field testing and expert advice as well as local knowledge and partnerships with non-governmental organisations working in the area would be invaluable in this respect. This paper therefore proposes further testing in the laboratory and in the field to fully investigate the behaviour of reinforced earth mortar samples including the efficacy of varying lengths and densities of fibre, It is also recommended laboratory testing be carries out on sample wall panels to determine the effect of reinforced earth mortar on the appropriate construction under seismic simulation.

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Random Rubble Masonry with Containment Reinforcement for Earthquake Resistant Houses in Hill Regions

Kaup S JAGADISH
Former professor of civil engineering,
Indian Institute of Science, Bengaluru,
ksjagadish@gmail.com

Rajendra DESAI
Structural Engineer, Honorary Joint Director, National Centre for People's-Action in Disaster Preparedness (NCPDP), rajrupal@gmail.com

Rupal DESAI
Architect, Honorary Joint Director, National Centre for People's-Action in Disaster Preparedness (NCPDP), rajrupal@gmail.com

ABSTRACT

In the earthquakes of the past few decades the buildings built with random rubble masonry in mud mortar have performed rather poorly, resulting in to the destruction of hundreds of thousand houses affecting the lives of millions of people. With timber becoming scarcer and increasingly expensive RC elements have been introduced in masonry to make it earthquake resistant. But past disasters in the mountainous areas of the Indian sub-continent have shown that this option is not viable in the remote mountainous regions on account of the excessively high cost of transportation of the materials used in it. In other words there is a dire need of a system that uses neither timber nor cement and steel bars. The Containment Reinforcement system is one such system that uses galvanized steel wires and weld wire mesh that is neither heavy nor unwieldy for long distance carting on human back or on mules. In past two decades the tests carried out on this system in different parts of India amply demonstrate its adequacy against high intensity earthquakes, while houses built after 2013 disaster in Uttarakhand demonstrate its viability. In short, this system has a potential for its large scale application for rebuilding in the earthquake hit Nepal.

Key Words – *Random Rubble Masonry, Out of Plane forces, Containment Reinforcement*

I. INTRODUCTION

In the past few decades with the engineering establishment in South Asia, or the Indian subcontinent, advocating predominantly reinforced concrete construction even for single and double story buildings, and the cement and steel manufacturers aggressively promoting their products through the visuals of neatly built engineered buildings, major shift has been observed from the traditional construction, especially masonry, to the RC frames. But this has resulted in to unprecedented escalation in the cost of construction, as also the fall in construction quality and building longevity. Another development is the introduction of RC elements in the masonry structures in the lieu of timber since the timber is in short supply. This has been legitimized through the inclusion of these changes in the building codes. But little or no attention has been given to the non-viability of these elements in the remote places, especially in the hill regions where motorable roads do not provide access to many places. The government's shelter rehabilitation program in the aftermath Himalayan Tsunami disaster of June 22, 2013 in Uttarakhand state in India brought this issue in forefront in a small way. It was observed that in a few affected villages that were far from the roads had extreme difficulties not only in ferrying cement and steel to their sites, but also in procuring the sand and aggregates. Now the Gorkha Earthquake has brought this issue in forefront in a big way since a few hundred thousand houses are to be rebuilt in such remote places and it is essential that they are "built back better" to be earthquake resistant. The quest for viable technology option has brought out a number of different suggestions from different people. One of them is the application of the "**Containment Reinforcement**".

II. CORE OF THE DELIBERATION

The concept of **containment reinforcement** was developed by the first author in the nineties after the Latur earthquake and taken up a number of studies[1, 2, 3, 4]. Second and third authors too had conducted a test after Kutchh Earthquake[5]. The reinforcement is intended for load bearing masonry buildings of one to three storeys. It is designed to meet the following requirements:

a) To prevent the out-of plane collapse of masonry walls which is frequently the cause of failure of masonry during earthquakes. Masonry is much stronger in the in-plane behavior. The problem of out of plane failure is accentuated in long walls pierced by many openings and in rubble stone masonry in mud mortar.

b) Masonry is inherently, exceedingly brittle, and there is a need to impart ductility through strategically placed reinforcement.

In this concept, the containment reinforcement is placed externally to the wall close to the surface. It could be of a material which has good tensile strength and elongation. It must be placed on opposite faces of the wall since the walls are invariably subjected to alternating out-of-plane loads. Since the wires/reinforcement have a tendency to delaminate/separate from the surface under alternating loads, the wires/reinforcement on opposite faces need to be held close to the wall by ties going through the walls. The containment reinforcement is essentially a vertical reinforcement placed at certain spacing along the wall. The spacing could vary between 0.5m to 1.0m depending on the design. Since the wall behaves like two dimensional plate under lateral loads, it also needs horizontal reinforcement in the form of bands placed in some of the bed joints. Such reinforcement should run continuously over the entire building except at door openings for sill bands. Fig.1 shows a schematic arrangement of a wall under construction with containment reinforcement and horizontal bends.

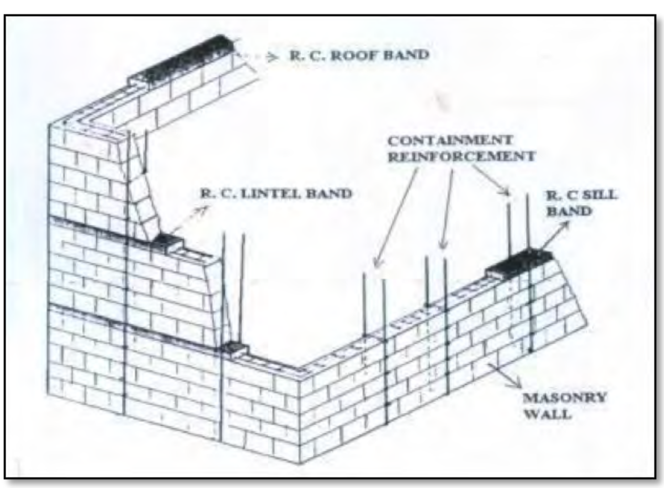

Figure 1 Schematic diagram of building model with seismic bands and vertical containment reinforcement

The extensive use of horizontal timber bands, sometimes coupled with vertical timber, in Uttarakhand (Figure 2), Himachal Pradesh and Turkey for earthquake resistance may be recalled at this juncture.

Figure 2 Vertical timber reinforcement in Uttarakhand houses

Recently National Centre for Peoples'-Action in Disaster Preparedness (NCPDP) under the guidance of the second and third authors implemented the containment reinforcement concept in the hill regions of Uttarakhand where stone masonry in mud mortar was the principal wall building material (Fig 3). Multiple strands of 14 guage (2mm) galvanized iron (GI) wires were used as the containment reinforcement in combination with the timber band at the top of the wall. There was no use of cement or steel bars for the structural purpose. These houses amply demonstrated the viability of the Containment Reinforcement in terms of not only the logistics of carting of the non-local materials but also the simplicity and adoptability of the technology.

Fig 3 – Houses with GI wire containment reinforcement and wooden wallplate in Uttarakhand state after 2013 Himalayan Tsunami

After the Nepal earthquake in 2015, a large number of rubble masonry in mud mortar buildings collapsed in rural Nepal. Hence, NCPDP decided to explore the potential of the containment reinforcement based on the Uttarakhand experience. Construction practices in Nepal, like in Uttarakhand, generally use $2^{1/2}$ half storied random rubble in mud mortar masonry buildings. Hence it was decided to test two models, first one of $1^{1/2}$ storey model and the second one of $2^{1/2}$ storey model of rubble stone in mud mortar. Both the models were to be of half scale and to be tested separately on the Nirma University shock table in city of Ahmedabad in Gujarat.

Shock Table test on 11/2 story model (Drawings in Annexure A)

This model was built with random rubble masonry in mud mortar with containment reinforcement consisting of 2-14 gauge GI wires at 450mm spacing plus weld wire mesh (WWM) bands placed in mud mortar at sill and lintle levels, and a timber band under the attic floor as shown in Figure 4, plus a timber wall plate to place the light tin roof on.

Several shocks were given to the table by raising different inclination and releasing the $1^{1/2}$ ton pendulum. The first two shocks were of low intensity using only 15^0 and 30^0 impacts. From the third impact the pendulum angle was raised to 45^0. The first two impacts caused no damage. From the 5^{th} impact onwards stones from the attic parapet started falling along with mud mortar falling from many areas. Some delamination occurred at the base on the South wall (pendulum side). The damage level increased with 9^{th} and 10^{th} shocks. During 11^{th} to 13^{th} shocks containment wires came off at top wall plate in the NW corner. A large number of stones got thrown out from the parapet portion during the 14^{th} shock. The post test conditions after the 14^{th} shock are shown in Fig 5.

Fig 4 – Construction sequence of $1^{1/2}$ story model

In the first 4 shocks the **table acceleration varied between 0.36g and 0.7g**. The **acceleration at the top of the walls varied between 0.4g to 0.9g**. This showed there was reasonably good amplification, and the building withstood these significant accelerations well.

Fig 5 –$1^{1/2}$ story model - post test condition

The fundamental frequency at the model varied between 4.6Hz to 5.0 Hz in these shocks. It must also be observed that since the model is of half scale it will be stiffer than the full scale building, and hence needs greater accelerations to cause damage. As the shocks progressed from 6 to 10 the table acceleration was between **1.5g to 1.98g**, while the wall top acceleration were varying between **0.56g to 1.26g**. The frequency of the model also came down to 1.5 to 2.46Hz. This is to be expected due to the weakening of the model. The damage which was predominantly in the form of delamination, although caused by the shocks, should also be attributed to poor quality masonry, as is most commonly observed.

Based on this experience it was felt that the $2^{1/2}$ storey model must, first and foremost, have better masonry interlocking with adequate through-stones, plus the diagonal-ties of GI wires on the faces of the model at top and bottom. It was also felt that a WWM band anchored securely at the top of the parapet walls with the containment wires could help hold together the wythes.

Shock Table test on $2^{1/2}$ storey model (Drawings in Annexure B)

This model had several features representing the improvements over the first model. At the attic parapet top level a WWM band was placed, and anchored down tightly with the containment reinforcement. Diagonal ties are provided in the attic wall tied between cross links on the wall face.

Fig 6 A - $2^{1/2}$ story test model under construction

Fig 6 B - 21/2 story test model complete with all wire and WWM reinforcement. Yellow indicates GI, red indicates Aluminum, and blue indicates WWM strap band

As in the first model a single timber wall-plate tied to the weld wire mesh band provided support to the roof. 2mm GI wires have also been installed around the corners on exterior face tied between the nearest cross links to keep small corner stones, if any, from separating from the wall. In the lower and upper storey WWM bands were provided at different sill levels extended to short slide (East and West) walls. In the South wall 4mm GI as well as 4 mm Aluminum wires are used to understand the performance of the weaker Aluminum. All the vertical wires were anchored to the wall masonry using 2-2mm GI crossties as before. Finally, the model had better quality masonry with adequate through-stones as well as better interlocking of the wythes (Fig 6A & B).

This model was subjected to 14 shocks from the $1^{1/2}$ Ton pendulum. The first two shocks were mild using 15^0 and 30^0 pendulum angles. From the third shock onwards up to 12^{th} shock, the pendulum angle was 45^0 degrees. For the 13^{th} shock the angle was 60^0 degrees, and it was 67^0 for the 14^{th} shock.

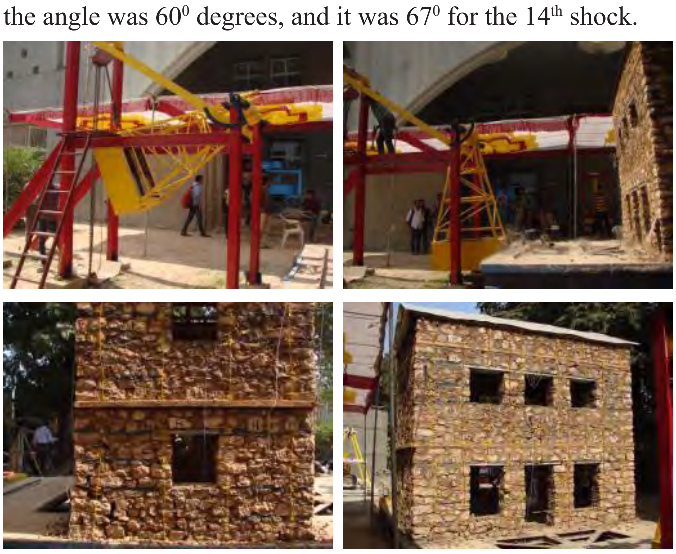

Fig 7 – Impact 14 - Pendulum at 67^0 inclination; Mortar and stones etc. falling at impact; Diagonal crack at NW base; Stone chips and mud mortar fallen out near S side base

From the 3^{rd} shock onwards mud mortar started falling from the masonry joints and fine horizontal cracks were developing. Between 7^{th} to 9^{th} shocks the NW corner at the base showed diagonal crack and the wall stated bulging. This continued up to 12^{th} shock. NE and NW corners at base showed cracking and stones jutting out. Unlike the first model there was no popping out of any stones, probably due to the better quality of masonry. As a whole this model behaved quite well.

The base acceleration of the table was around 0.5g in the first two shocks and increased to 1g in the 3^{rd} and 4^{th} shocks. It further increased to 1.6g up to 7^{th} shock and finally reached 2.0g in the 13^{th} and 14^{th} shocks. The acceleration of the model at the top storey varied between 0.4g to 0.7g indicating that there was no amplification. The natural frequency of the model was around 4.0Hz to 5.0 Hz initially, but settled down to 3.0 Hz as the shock intensity increased. Finally, it came down to 1.5 Hz at the 13^{th} and 14^{th} shocks. Fig.7 shows the photograph of the model after the 14^{th} shock.

III. Conclusions and Recommendations

Following are the principal observations:

- The use of better deployment of containment reinforcement and diagonal ties, plus the parapet wall top WWM band lead to better performance of the second model.
- Better quality random rubble masonry is a prerequisite for good performance.
- No difference in the performance was observed between GI and aluminum wires.
- No visible distress was observed in the cross-ties through the wall that hold the containment reinforcement.
- To sum it up, the containment reinforcement, WWM bands along with good quality random rubble masonry in mud mortar have a credible potential to bring safety against earthquakes in very high damage risk seismic areas of the subcontinent.

Acknowledgements

The authors express their gratitude to Nirma University, Prof. Paresh Patel HoD Civil and Prof. Sharad Purohit for their support and cooperation in the recent shock table tests conducted for Nepal model, as well as in recording the accelerometer data of these tests.

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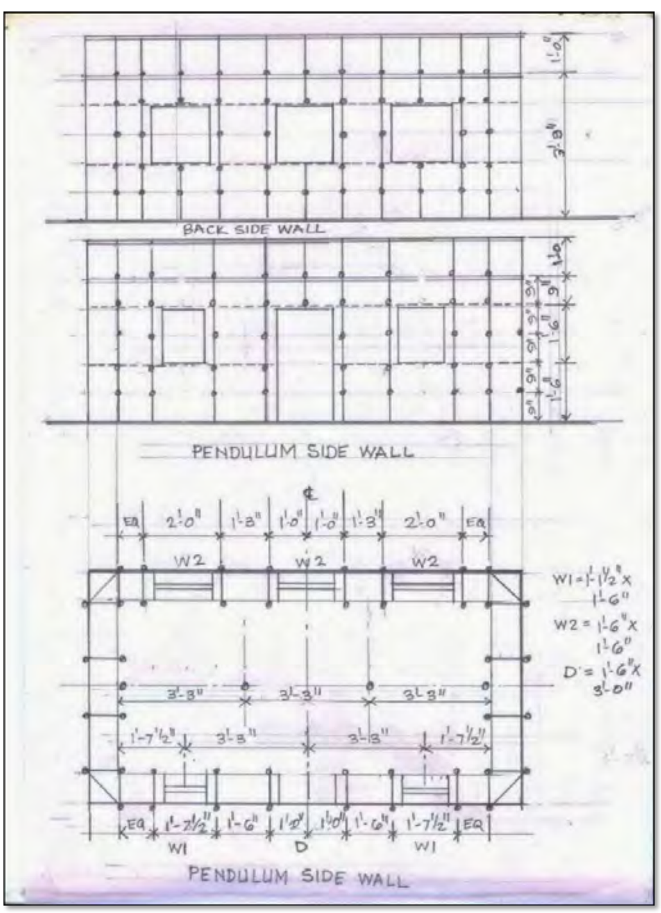
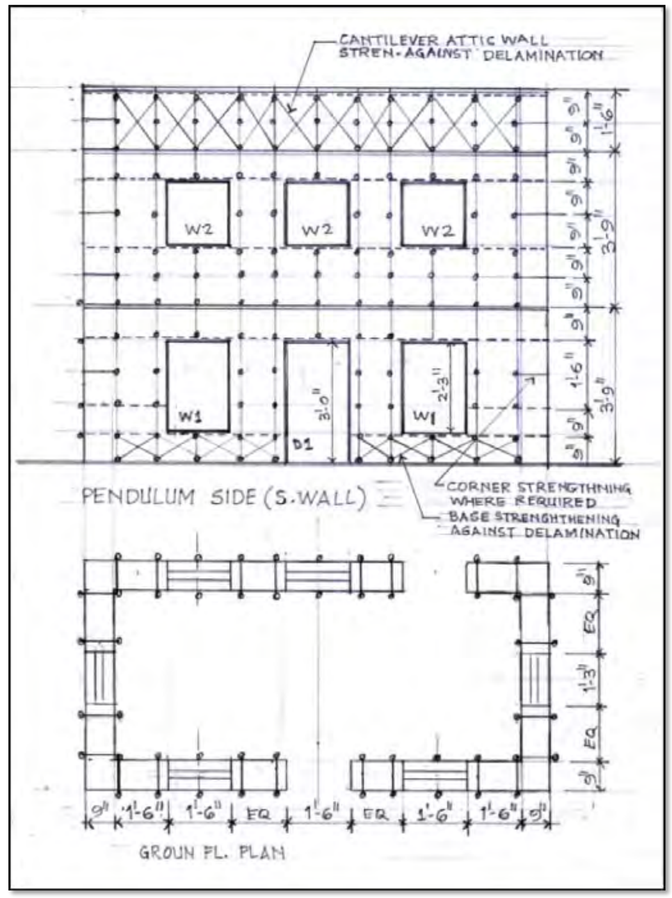
Annexure A

Shock Test 1 – 1 1/2 story model

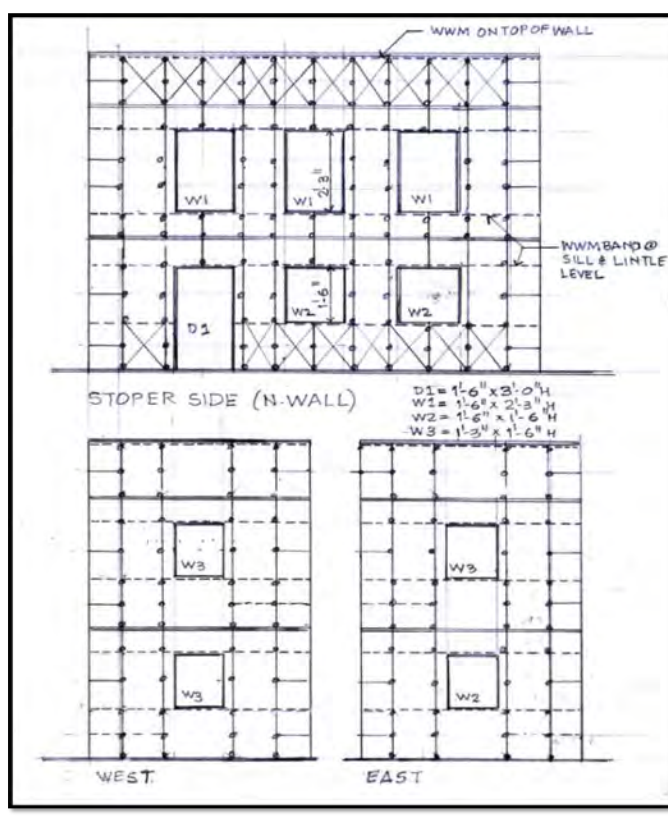


Annexure B

Shock Test 2 – 2 1/2 story model



Shock Test 2 – 2 1/2 story model



Annexure C

Brief Introduction of Authors

Former Professor K.S.Jagadish

K.S.Jagadish completed his graduation in Civil Engineering from Mysore University in 1961.He then obtained his Ph.D in Structural Engneering in 1969 from Indian Institute of Science (IISc), Bengaluru. He joined the faculty of Civil Engineering in 1967 in the Institute and also spent a year as Post Doctoral Fellow at Karlsruhe University in Germany. He eventually became Professor of Civil Engineering in Indian Institute of Science in 1984. He concurrently took up the Chairmanship of the Centre of ASTRA (Application of Science & Technology for Rural Areas) from 1983 to 1989. After retirement in 2002 he was Emeritus Scientist for 4 years at the institute. In 2006 he took up Visiting Professorship at R.V.College of Engineering and mentored their Master's Program in Structural Engineering till Jan 2015. He is currently associated with a voluntary agency GRAMAVIDYA and is involved in the dissemination of Alternative Sustainable Technologies. His interests include Structural Dynamics, Earthquake Engineering, Shell Theory, Masonry Structures, Alternative Buildings and Rural Energy Systems. He has authored several books.

Rajendra Desai

Hon. Joint Director, National Centre for Peoples' Action in Disaster Preparedness (NCPDP), Ahmedabad

Rajendra Desai obtained his BTech in Civil Engineering from IIT Bombay in 1970. Later, he got his MS in Structures from Rutgers University, USA (1972). After working in the USA for 12 years he returned to India in 1985 with his architect wife Rupal to focus on the softer technologies for shelter, water, sanitation and energy, which are more sustainable and viable for the Indian context. For next eight years he worked with different NGOs on these issues getting exposed to the grassroots conditions in these fields in the country.

A damage assessment assignment in the aftermath of 1993 Latur earthquake in Central India took him into the field of disaster risk mitigation, which soon became his principal field of work. He along with his wife devoted much effort on the performance improvement of the buildings made with vernacular building systems that predominantly depend upon the local materials and skills through the input of modern engineering and to some extent the modern materials. Recognizing the widespread vulnerability of buildings in the country and the potential of Seismic retrofitting in reducing that in a viable manner, he made it his mission to promote multi-hazard retrofitting in all hazard prone regions of the country.

In the past two and a half decades he has worked on disaster risk reduction of building, especially houses, in many parts of the country including Latur - Maharashtra, Jabalpur - MP, Kutchh – Gujarat, Kashmir, Uttarakhand, Himachal Pradesh, Delhi etc. As a part of these efforts he has been involved in the post disaster shelter rehabilitation programs, disaster risk reduction technology demonstration projects aimed at housing as well as public building, training of construction personnel including engineers, masons etc. as well as making of IEC materials such building technology manuals, brochure, etc.

His work has brought him a number of honors including the first A.S. Arya Award of IIT-Roorkee in 1998, Distinguished Alumnus Award from IIT-Mumbai in recognition of his work at the grassroots in 2010, and HUDCO Design Awards 2013, 2014 and 2015.

Rupal Desai

Hon. Joint Director, National Centre for Peoples' Action in Disaster Preparedness (NCPDP), Ahmedabad

Rupal Desai obtained her B. Arch. from Sir J.J.College of Architecture, Bombay in 1968. Later she got her and M. Arch. from Pratt Institute, Brooklyne, USA 1974. After work in the USA for 12 years she returned to Indian in 1985 with her engineer husband Rajendra to focus on the softer technologies for shelter, water, sanitation and energy, which are more sustainable and viable for the Indian context. For next eight years he worked with different NGOs on these issues getting exposed to the grassroots conditions in these fields in the country.

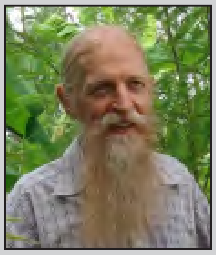
Beginning of 1994 saw a radical change in her work focus to post disaster shelter rehabilitation in the aftermath of the Latur Earthquake of 1993. She along with her husband devoted much effort on the performance improvement of the buildings made with vernacular building systems that predominantly depend upon the local materials and skills through the input of modern engineering and to some extent the modern materials. Recognizing the widespread vulnerability of buildings in the country and the potential of Seismic retrofitting in reducing that in a viable manner, she made it her mission to promote multi-hazard retrofitting in all hazard prone regions of the country.

In the past two and a half decades he has worked on disaster risk reduction of building, especially houses, in many parts of the country including Latur - Maharashtra, Jabalpur - MP, Kutchh – Gujarat, Kashmir, Uttarakhand, Himachal Pradesh, Delhi etc. As a part of these efforts he has been involved in the post disaster shelter rehabilitation programs, disaster risk reduction technology demonstration projects aimed at housing as well as public building, training of construction personnel including engineers, masons etc. Her most noteworthy contribution has been the IEC materials including instructional technical videos not only in the aftermath of various disasters, but several major publications for UNDP, UNESCO, Building Materials & Technology Promotion Council (GoI) consisting of technical manuals.

Her work jointly with her husband has brought her a number of honors including the HUDCO-Hari Om Ashram Award in 1995, the first A.S. Arya Award of IIT-Roorkee in 1998, and HUDCO Design Awards 2013, 2014 and 2015.

She is a life member of the Council of Architecture, India.

Earthbag Technology - Simple, Safe and Sustainable



Dr. Owen GEIGER
Director of the Geiger Research Institute of Sustainable Building, USA
naturalhouses@gmail.com

Kateryna ZEMSKOVA
Co-Founder and President of Good Earth Nepal, a New York based non-profit organization, USA
kateryna@goodearthnepal.org

ABSTRACT

Earthbag technology is an inexpensive, simple and sustainable method for *building* structures. Having evolved from military bunker construction and flood control methods, Earthbag buildings are notable for their ability to endure fire, flood, wind, *earthquake* and vermin, and are used in *disaster*-prone zones all over the world. In *Nepal*, all 55 Earthbag buildings survived a 7.8 magnitude earthquake with no structural damage. Because Earthbag technology makes minimal use of cement, concrete, steel and timber, and the fuel needed to transport them. This technique is easy on the *environmen*t, and doesn't deplete scarce natural resources. Earthbag technology also requires less expertise than other traditional building methods, and only the simplest of tools.

I. INTRODUCTION

Earthbag technology is a wall system with structures composed primarily of ordinary soil found at the construction site. The soil is stuffed inside polypropylene bags, which are then staggered like masonry and solidly tamped.
Barbed wire is used between the layers of bags and serves as mortar. For seismically active zones reinforcements like buttresses, vertical rebars and bond beams are recommended. The classical foundation used in Earthbag construction is a rubble trench foundation. The roof design can be of any preference as long as it is lightweight.

Figure 1: Earthbag Construction in Makwanpur, built by Good Earth Nepal **(Good Earth Nepal, 2015).**

Figure 2: Completed Earthbag House in Gorkha, built by Good Earth Nepal **(Good Earth Nepal, 2015).**

Earthbag construction minimizes the need for skilled labor, and does not require any special tools or machinery. An Earthbag building can easily be built by a group of unskilled workers under the supervision of a construction manager. Since Earthbag technology relies primarily on locally-sourced materials there is less need for transport, and lower fuel costs. This makes Earthbag building ideal for remote or isolated areas with bad roads and limited access.

Though Earthbag technology is relatively "new" its true origin dates back to thousands of years through the similar technique of rammed earth construction. Some call Earthbag technology "Rammed Earth in a Bag" or "Reinforced Rammed Earth". Structures utilizing similar tamped, solid earth techniques still stand from the Alhambra palace in Spain to the Great Wall of China.

Figure 3: Kagbeni Rammed Earth Monastery, Nepal, built 1429 **(Geiger, O., 2013).**

In recent centuries, the idea of building walls using bags filled with sand or earth and stacking them has been utilized by the military and industrial concerns.

Figure 4: 200-300 military history **(Hart, K. & Geiger O. , (n.d.)).**

Municipalities have used it for flood control, erosion control and retaining walls. Road builders have also deployed Earthbags, placing them under highways.

Figure 5: Earthbags are used under highways **(Geiger, O., 2013)**

Figure 6: Sandbags are used for flood control **(Geiger,O., 2013)**

In 1976, Gernot Minke of the Research Laboratory for Experimental Building in Kassel, Germany began to investigate how natural building materials like sand and gravel can be used to build a residential house.

Figure 7: Minke prototype house, Guatemala, 1978 **(Hart, K. & Geiger O., (n.d.))**

In 1980s, Iranian-born architect Nader Khalili popularized the notion of building permanent structures with bags filled with earthen materials.

Figure 8: Nader Khalili **(Bidoun, 2004)**

Figure 9: Earthbag house built by Nader Khalili **(Turner, I., 2016)**

At present, there are over 15,000 Earthbag buildings worldwide with recent Earthbag constructions gaining approval under strict US building codes.

An estimated 55 Earthbag structures built in Nepal survived the 2015 earthquake, in regions ranging from Solokumbu to Sindhupalchok to Kathmandu.[1]

Figure 10: Earthbag School in Basa, Nepal built by Small World next to a damaged stone building (Good Earth Nepal, 2015)

Figure11: Earthjbag School built by First Steps Himalayas survived the earthquake with no damage, Sindupalchok (**Geiger,O., 2013)**

[1] For the details of Earthbag structures built in Nepal before the earthquake, please visit Natural Building Blog (http://www.naturalbuildingblog.com/)

II. EARTHBAG TECHNOLOGY

Earthbag building offers many advantages over existing technologies:

- *Safety*- Earthbag structures built in Nepal before the earthquake had no structural damage. More traditional building techniques were tragically failed.
- *Ease of Construction*- Earthbag technology can be easily learned by rural villagers.
- *Reduced Use of Materials*- Earthbag structures require a minimal amount of cement, concrete, wood and steel.
- *Reduced Use of Fuel and Transportation*- Use of local materials and less quantity of materials mean less need for transport and lower fuel costs.
- *Less Pollution*- Building with soil means fewer factories and smoke stacks, fewer pollution-belching trucks for transporting the load, and less depletion of Nepal's forests and natural resources.
- *Cost-Effective*- Building with Earthbags is inexpensive. For example, a typical Earthbag house might cost 900 NPR per square foot, versus 2500 NPR for concrete block construction.

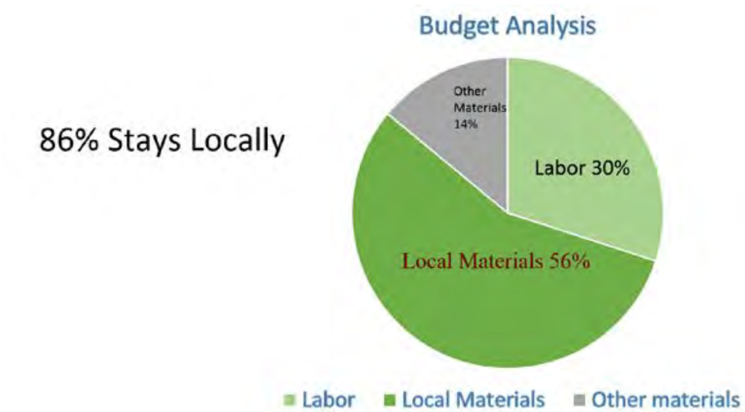


Figure 12: Earthbag Construction Cost Analysis **(Good Earth Nepal, 2015)**

MAIN BUILDING MATERIALS

Soil: The main material of an Earthbag structure is ordinary soil obtainable at the worksite. Most soils are adequate and precise ratio is not necessary, but there must be enough clay and moisture to bind the aggregate together. The soil can be easily tested without any equipment, using a drop test or a roll test. The most common mix is:
25%-30% Clay

70%-75% Sandy soil
10% Moisture (optimum moisture content)

Polypropylene bags or tubes: Earthbag construction is durable, and if the polypropylene bags are plastered properly the construction can last for hundreds of years. A study by the U.S. Federal Highway Administration found that the half-life of polypropylene fabrics in benign environments can be 500 years or more. The bags themselves have a tensile strength even higher than that of steel, and can resist circumferential forces generated from the weight above.

Figure 13: Rolls of polypropylene fabrics **(Hart, K. & Geiger O. (n.d.))**.

Barbed Wire helps to lock the bags together, and forms a matrix within the wall system. Barbed wire resists outward expansion of the wall caused by weight from above, and its tensile strength resists out-plane forces. Barbed wire should be 14 gauge, 4 pointed.

Figure 14: Barbed wire dispenser **(Geiger, O., (2015, September))**

Figure 15: Barbed wire layout **(Geiger, O. (2015, September))**

KEY COMPONENTS THAT MAKE EARTHBAG STRUCTURE EARTHQUAKE RESISTANT

Earthbag structures, despite being heavy, have high flexibility that makes them highly earthquake resistant.

1. **Rubble Trench Foundation**
   A rubble trench foundation was first popularized by Frank Lloyd Wright in 1922, and used for his Imperial Hotel design. This hotel survived the great Kanto earthquake, the most devastating in Japanese history[2].
   An Earthbag building uses its own weight to anchor itself to the rubble trench foundation. Since the superstructure is not attached to the foundation by bolts or rebars, the foundation and the superstructure are able to move independently minimizing the shock transfer to the walls. A rubble trench is also built of individual units rather than a continuous beam further absorbing the shock.

2. **Earthbags are resilient (helps to absorb the shock)**
   Earthbags are resilient. As per an experimental study on vibration reduction performed by three Chinese universities (Hohai University; Business School of Hohai University; Hefei University of Technology), Earthbags have a relatively high damping ratio with horizontal as well as well as vertical vibrations effectively reduced.

3. **Tensile strength of Barbed Wire**
   Barbed wire helps to lock the bags together and form a matrix within the wall system. It helps to resist any tendency of wall to expand outward due to the weight from above. During an earthquake tensile strength of the barbed wire helps resist the out-plane force.

4. **Thick walls (16''-19'')**
   Earthbag construction is flexible, and thick walls make the construction stable

5. **Concrete bond beam** provides integrity to the structure

   Figure 16: Concrete Bond Beam **(Good Earth Nepal, 2015)**

6. **Reinforcements. Vertical rebars** provide additional shear strength. **Buttresses** and **corner reinforcements** increase the in-plane stiffness of the wall.

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[2] Frank Lloyd Wright used rubble trench foundations successfully for more than 50 years in the first half of the 20th century, and there is a revival of this style of foundation with the increased interest in green building (Wikipedia, 2016)

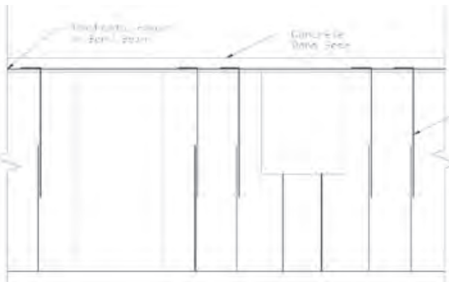

Figure 17: Vertical rebar reinforcement **(Good Earth Nepal, 2015)**

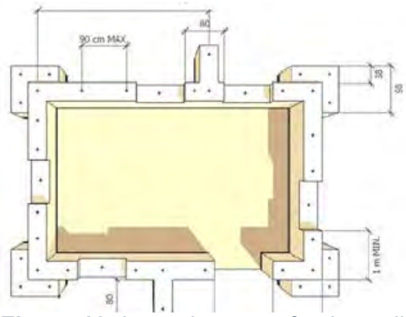

Figure 18: Lateral support for the wall **(Geiger, O. (2015, September))**

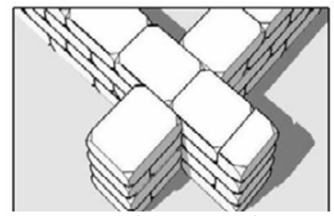

Figure 19: Corner reinforcement **(Geiger, O. (2015, September))**

7. **Thick plaster with galvanized or plastic mesh** provides additional strength to the wall to resist in-plane as well as out-place forces.

Figure 20: Plastic mesh and cement plaster **(Good Earth Nepal, 2015)**

8. **Rammed Earth in Polypropylene (PP) Bags**
   As PP bags have tensile strength even higher than that of steel, they can resist the circumferential force produced by the weight above before the earth in the bag has hardened. In between 2 to 3 months the earth in the bags hardens like a brick. Unless there is a movement the bag does not bear any forces.

All of these components make Earthbag structures extremely earthquake-resistant. Tests done in accordance with IBC standards have found that Earthbag construction far exceeds Zone 4 standards, devised to protect against the very highest level of seismic activity. Numerous Earthbag structures have been built in the United States. Earthbag structures are permitted by the California Building Code, the toughest in the United States due to high seismic activity.

EARTHBAG CONSTRUCTION DETAILS

1. **FOUNDATION**
   Earthbag structures generally employ a rubble trench foundation, though more traditional types of foundations can be used as well.
   Rubble trench foundation is suitable only for medium or hard soils. A 2’-3’ deep and 2’ wide trench is filled with rubble up to 6’’ below the ground level.

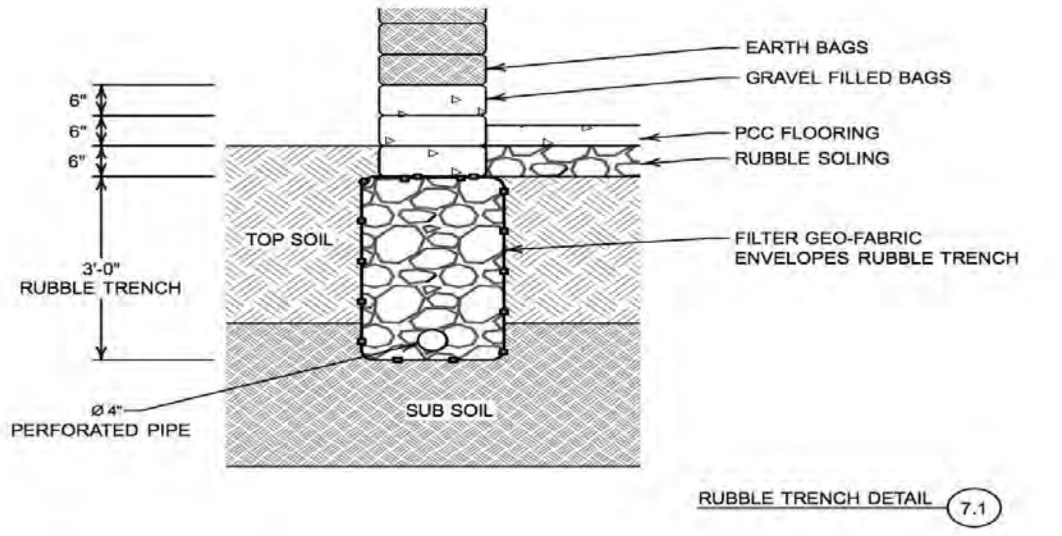

Figure 21: Rubble Trench Foundation **(Geiger. O., 2013)**

2. **GRAVEL-FILLED BAGS**

2-3 courses of bags filled with gravel are placed on top of the rubble trench foundation. The first course of gravel bags should be placed 6'' below the soil level. It is better to use double-bags for additional strength.
Gravel-filled bags starting below grade and extending well above grade in flood-prone areas reduce the risk of water damage.

3. **EARTHBAG WALL**

The Earthbag wall is composed of polypropylene bags or tubes filled with soil. At least two strands of 4-point barbed wire should be used in between each course of bags. Spacing of barbs should not be more than 100mm and minimum bending strength of barbs should be 200N.
Tamping is a very important step of Earthbag wall construction. It helps to maintain the level as well as solidify the soil inside the bags.
There are numbers of rules that need to be followed for the construction of an Earthbag wall:

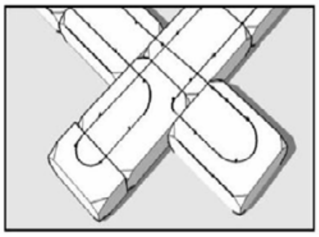

Figure 22: Earthbag wall with barbed wire **(Geiger, O. (2015, September))**

- The height to thickness ratio of a wall should not be more than 8.
- Any opening in the wall should be small in size and centrally located. Openings are to be located away from inside corners by a minimum 900 mm. There should a minimum of 900 mm spacing between the openings. Maximum opening allowed is 1500 mm.

- The maximum length of unsupported wall should not exceed 10 times its thickness. If the wall is longer than 10 times its thickness, buttresses should be placed at intervals not exceeding 10 times the wall thickness.

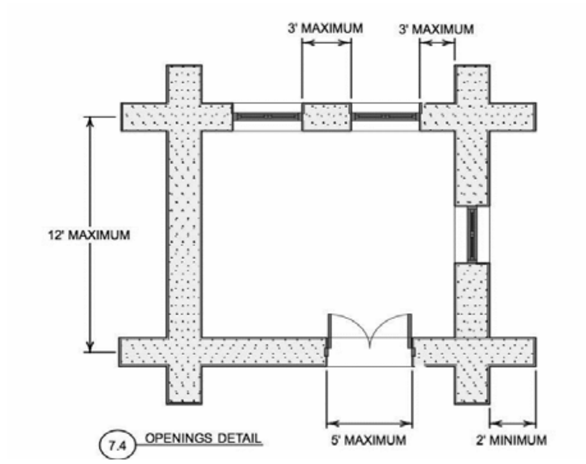

Figure 23: Specification of openings and unsupported wall length **(Geiger, O., Hart, K. & Stouter, P. (n.d.)).**

- Earthbag overlap: In order to achieve the full strength of Earthbag wall, the common bonds specified below should be followed.

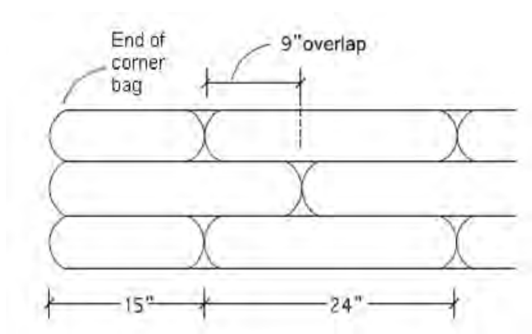

Figure 22: Good Overlap with 24” bags **(Geiger, O. (2015, September))**

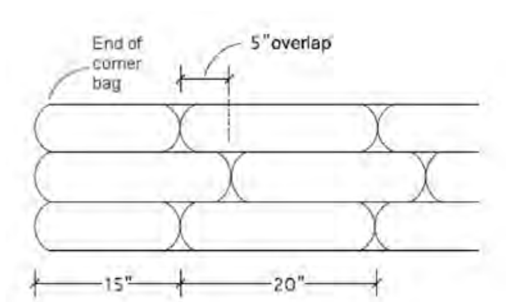

Figure 25: Poor Overlap with 20” bags **(Geiger, O. (2015, September))**

4. **CORNER REINFORCEMENT**

In earthquake-prone areas corner reinforcements are highly recommended. There are 5 types of corner reinforcements:

- **Corner Buttresses**

  Buttresses strengthen corners and stiffen straight walls. Straight walls need a buttress or a pier, an intersecting interior wall, or a minor corner every 3- 3.5 m (10'- 11'). They also make it easier to add on Earthbags to extend houses in the future. Buttresses can be straight, sloping, or stepped. Benches or wider wall bases will also strengthen straight walls if the bags are well woven into the wall. A vertical-edged buttress must stick out from the wall at least 60 cm (24"), and a sloping or stepped buttress 75 cm (30").

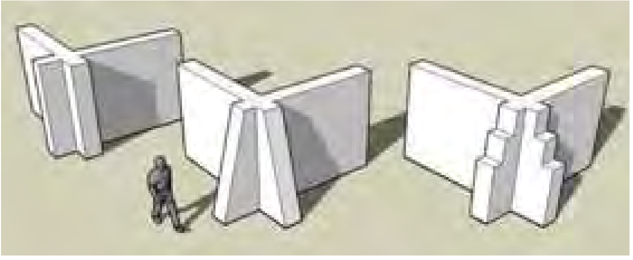

Figure 26: Corner Reinforcement Using Butresses **(Hart, K. & Geiger O. (n.d.)).**

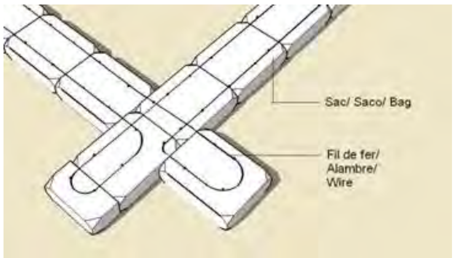

Figure 27: Corner Butresses with barbed wire **(Hart, K. & Geiger O. (n.d.)).**

- **Corner Rebars**

  This is the simplest way to strengthen corners of Earthbag buildings.
  When walls reach 1.5 m (60") height, a 1.5m (5') long piece of rebar is hammered through the corner bags. Bags are always alternated at corners and bag joints are staggered for strength.

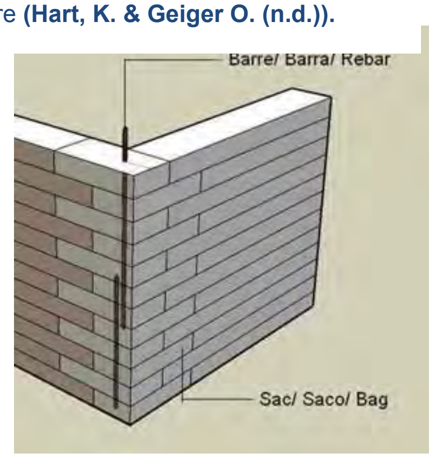

Figure 28: Corner Reinforcement Using Rebar **(Hart, K. & Geiger O. (n.d.)).**

- **Mesh Corner Reinforcement**

  A 1 m (39") wide strip of sturdy mesh is placed on the outside of the corner from top to bottom. The exterior mesh is fastened securely to the inside corner at every other course. Metal rod or bamboo rod or additional mesh should be placed on the interior corner. If bamboo is used it can be either well encased in earth or left uncovered for inspection. Earth-filled bags are alternated at corners.

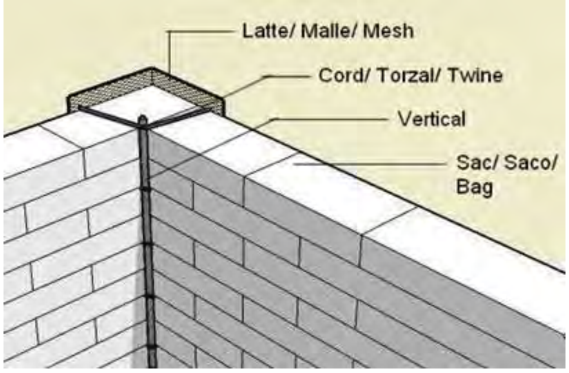

Figure 29: Corner Reinforcement Using Mesh **(Hart, K. & Geiger O. (n.d.)).**

- **Corrugated Metal Reinforcement**
  This corner reinforcement stiffens and strengthens as well as unites walls if repeated every 5 bag course. 60 cm (24") long rebar is driven through corrugated metal strip at the corner to tie the reinforcement to the wall below. Rebars repeat at ends of metal strip or every 60 cm (24"). Corrugated metal roofing is cut into strips 20- 30 cm (8- 12") wide and at least 75 cm (30") long. They are overlapped at the corners and nailed into the bag below.
  Strong cord or mesh is used with wire or strapping every 60 cm (24") to secure three layers of bags tightly around the metal.

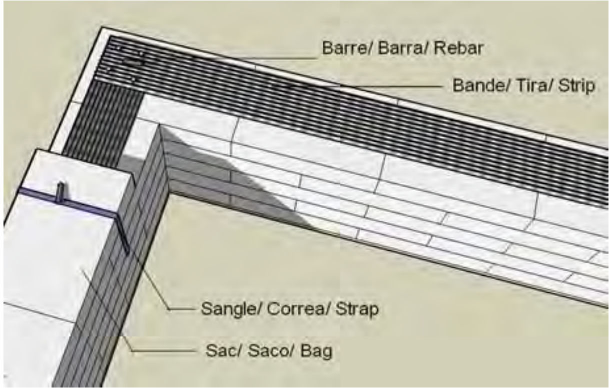

Figure 30: Corner Reinforcement Using Corrugated Metal Sheet **(Hart, K. & Geiger O. (n.d.)).**

- **Pier**
  A pier is usually a thickened wall section. It only projects out from the wall the width of a single bag.

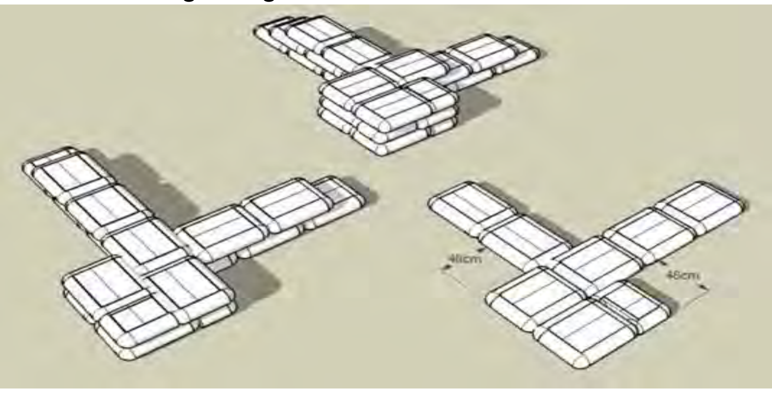

Figure 31: Corner Reinforcement Using Pier **(Hart, K. & Geiger O. (n.d.)).**

5. **VERTICAL REINFORCEMENT**
   Steel bars are installed at the critical sections (i.e. the corners of walls, junctions of walls, and jambs of doors) and every 1.2 m of the normal wall section.
   The vertical steel at the corners and junctions of walls must be taken into the roof band.

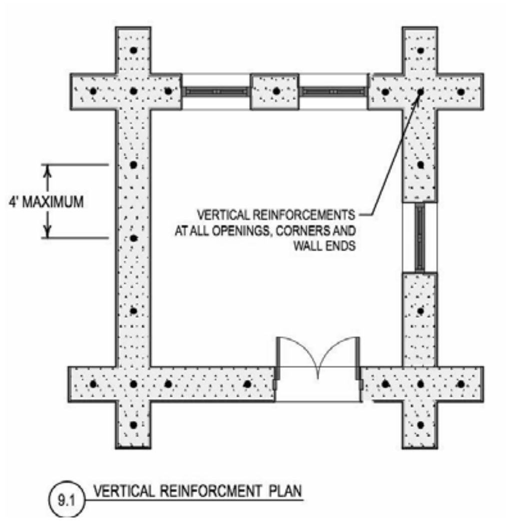

Figure 32: Vertical Reinforcement Plan **(Geiger O., (2015, September)).**

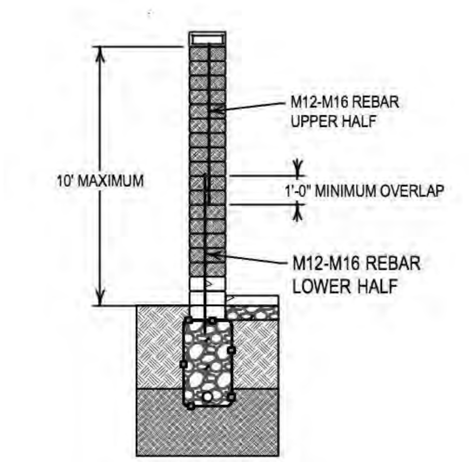

Figure 33: *Vertical Reinforcement (Sectional View)* **Geiger O., (2015, September)).**

6. **BOND BEAM**

   Bond beam provides integrity to the structure. It ties the walls together and provides a rigid connection for the roof structure.

   Concrete Bond Beam

   Concrete bond beam is the recommended option. The depth should be at least 6'' and the width 2/3 of the bag width. It is singly reinforced. The length of the part of the rebars embedded into the bond beam should be at least 60 times diameter of the rebar.

   Timber Roof Band

   Timber band can be used as an alternative to concrete band where there is ample supply of timber and poor road access.

   - Minimum 4”x6” structural wall plate member
   - Vertical reinforcement rebar carries through the wall plate
   - Cross bracing in all corners and junctions
   - Trussed roof only so there is no lateral load from the roof

7. **ROOF**

   As a general rule heavy roofs are more seismic hazard. Hence roofs as well as floors should be made as light as structurally and functionally possible. For Earthbag construction a trussed roof is recommended in all seismic zone.

CONCLUSION

Earthbag technology offers a safe, simple and sustainable building option. We encourage engineers and building professionals in Nepal and other countries to explore this exciting new technology, and its possible use in the communities most in need.

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An Approach of Build Back Better for Transportation Lifelines in Nepal After 25 April 2015 Earthquake

Jagat Kumar SHRESTHA, PhD
Assistant Dean, Institute of Engineering,
Tribhuvan University, Nepal.
email: jagatshrestha@ioe.edu.np

Abstract

Road networks provide a vital lifelines function to society, and their availability is critical for emergency response and recovery after major hazard events like 25 April 2015 Earthquake in Nepal. The resilience of the strategic road network in the country to a large magnitude earthquake has to be assessed and mapped. An approach of study is necessary to determine the vulnerability of the network, where the road network could be cut off by major landslides on road through rugged terrain, as well as liquefaction and lateral spreading at low lying areas and build back better. The approach based assessment will enable the government to manage the risks to the road network to provide the levels of service to fulfil the social and regulatory responsibilities. The study approach also enables decision makers to consider the whole inter-connected network in local and regional emergency response planning and developing initiatives to enhance resilience.

Keywords:*road networks, lifelines, earthquake hazards, vulnerability, resilience*

I. INTRODUCTION

The 25 April 2015 earthquake in Nepal caused damages to transportation infrastructures, which, mainly comprise roads. Tribhuvan International Airport, Nepal's only international airport, in Kathmandu, closed for few hours following both earthquakes and some of the larger aftershocks. However, the runway was closed to all large cargo flights as repairs were required to be carried out on the runway as the damage worsened in the immediate aftermath of the first earthquake due to the increased number of planes bringing aid and relief workers into the country. Nepal has two types of road networks. The first type of network is strategic road network (SRN), the core network of national highways and feeder roads connecting district headquarters, covers about 14,902 kilometers (km). Of this, 51% is paved, 13% is graveled, and 36% is earthen (DoR, 2015). The second type of network is local road network (LRN), which includes 50,944 km of local road networks in 75 districts of Nepal, of which about 3% is blacktopped, 29% is graveled, and 68% is earthen (DoLIDAR, 2012). Out of the 75 districts, 31 districts have been affected by the earthquake and 14 of them are severely affected. The 31 earthquake - affected districts have about 5,140 km of SRN and 29,443 km of LRN (PDNA, 2015).

The road and highway network across the districts were heavily impacted, with more than 2,000 kilometers (13 percent) of the network – damaged or destroyed. Some roads of the SRN have sunk or were completely destroyed by the earthquake of 25 April 2015 and the aftershock of 12 May 2015. In many mountain areas, landslides caused partial blockage of road traffic. Traffic and road access was disrupted in the 26 km Arniko highway section, adjacent to the border with China, totally interrupting trade flows between the two countries. Worst affected were the districts of Sindhupalchowk, Dolakha, and Nuwakot. The severe cracking and debris-covered roadways made it very challenging for relief and rescue works to the hardest-hit remote areas. Extensive road blockages and inaccessibility for a few weeks were seen in the LRN in the days after the earthquake. The obstructions were caused mainly by landslides, which washed out some road sections completely. Kathmandu, which is home to around 25 million people, did not have a significant reduction in transportation services and no destruction of public transportation vehicles reported. However, many narrow street all over Kathmandu were blocked by fallen debris of damaged buildings and compound walls.

II. IMPACT ON ROAD NETWORKS

According to PDNA, 2015, on SRN, very small percent of the network have suffered settlements or sinking or complete damage/washout due to the earthquake. In the hill/mountain roads, landslides have caused a partial blockage of road traffic for a few days in limited sections of the SRN only. As stated above, a major disruption of traffic has been experienced in the entire 26 km of Arniko highway sections adjacent to the Chinese border causing interruption of trade flows between the 2 countries. About 20 percent of strategic road networks affected.

The nature of the recovery works, however, is to provide a good opportunity for building more disaster resilient infrastructures. Out of 14,902 km of the SRN, the condition of SRN is 10% in good condition, 74% in fair condition and 16% requires urgent repair (PDNA, 2015). LRN comprises two types of road networks: District Road Core Network (DRCN) and Village Road Core Network (VRCN).

On LRN, extensive road blockages were reported in DRCN for a number of days, while VRCN, most of which were in the non-motorable conditions even before the 25 April earthquake, have suffered further blockages and inaccessibility. The obstructions were mainly due to landslides. Some road sections were completely destabilized and damaged. The districts have received reports of persisting instability in fragile areas with visible cracks and fissures noted above the road sections.

Figure 1: Road blockade

(Source:http://www.ekantipur.com/2015/05/19/national/worst-quake-hit-sindhu-in-shambles/405401.html)

It is obvious that, a significant portion of the VRCN and some DRCN roads require heavy investment to make them accessible, usable or indeed to have safe public transport on these roads. This has resulted in the limited accessibility or usability of LRN even before the current earthquake. The situation was worsened by the earthquake.

The total population of the most severely affected 14 districts is about 5.37 million. Most of them are in the remote rural communities and have faced the inaccessibility to social and economic services. The losses in economic flow due to this inaccessibility after the earthquake is very high however yet to be accounted. The damages to the LRN and the consequent interruption of traffic from the earthquake affected areas have severely impacted the more vulnerable members of the communities particularly women and children. LRN contribute to the economic activities to the rural communities inducing more agricultural production and technology transfers from urban areas. LRN contributes to the development of the social sector, such as health and education through the provision of easier access to social amenities (hospitals and schools) to the rural communities. In fact, loss due to the blockages of LRN arises more in agriculture, education and health sectors than in the transport sector itself. LRN is the lifeline for the rural people. The disruption in the system is due to improper development of LRN including lack of redundancy in the LRN network for alternative routing.

The recovery cost in the transport sector is estimated at NRs28,185 million ($282 million) (PDNA, 2015). This is because LRN are not designed to be resilient in disasters with considerations to the relatively low traffic volumes. There are urgent needs to recover the normal accessibility to the remote areas. About 68% of the estimated total needs are in the LRN, which are the lifeline infrastructure for the rural communities. The share of SRN is about 32%.

III. RECOVERY STRATEGY, AN APPROACH FOR BUILD BACK BETTER

The main objective of recovery efforts is to restore the various modes of transport to their pre-disaster functionality. It is also an opportunity for the government to design and build more disaster-resilient infrastructure facilities after the disaster. This involves immediate and transitional measures to resume the delivery of transportation services in the various subsectors until reconstruction and rehabilitation of permanent structures is completed.

The PDNA has indicated a build back better principle and proposed to be involved in rehabilitation, reconstruction, or upgrade of larger-scale transportation infrastructure and road networks. However, what the principle is and how the principle will work, yet to be finalized and implemented. An approach of build back better for transportation infrastructure in earthquake affected districts in Nepal is discussed as follows.

Performancestate of existing road network

A study can be carried out considering the risk to the road lifelines in the earthquake affected network where both the extent and duration of a loss of functionality can be obtained which are important for measuring the performance of the road network. The priority can be given to develop an approach to consider the functionality of the road network as well as provide meaningful parameters for risk mitigation and response planning. In this context, the concept of resilience of road transportation lifelines is dependent on their vulnerability to a loss of quality or serviceability, and the time taken to bring them back into original usage state after the reduction or loss of access. This concept is shown in Figure 2, where following an adverse event there is a loss of service that requires a period of recovery time to restore the network back to its previous level of quality. Thus, the smaller the shaded area, the more resilient is the lifeline. The greater the area, the poorer is the performance.

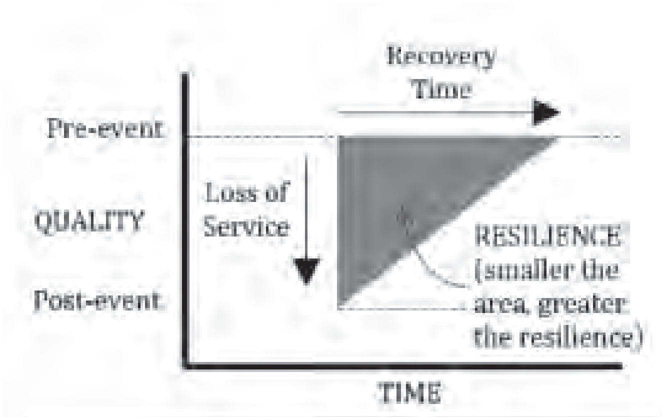

Figure 2: Resilience of road network (Mason and Brabhaharan, 2015)

Performance states or resilience states can represent the performance of the road network which can be determined considering the impact of various natural hazards on the road network on a similar basis (Brabhaharan et al., 2006).

Characterization of the road network

GIS-based spatial approach can be used to assess the resilience of the network through performance states, to characterize a road network (Brabhaharan et al. 2001). The road network can be characterized through drive over mapping of the road, and then characterization using a combination of the GIS and field information, geology and hazard maps and review with the aid of Google Earth Maps. The characterization can be captured into an ArcGIS spatial database.

All the resilience states can be developed for a large local earthquake affecting the whole of the affected districts. Regional studies of earthquake ground damage hazards such as slope failure and liquefaction can be carried out and can be presented in the form of maps. These maps can be used to assess the hazards to the roads.

Resilience state mapping

Performance or resilience states can be developed and used for mapping the vulnerability of the road network in the earthquake affected areas. This can be carried out by assigning resilience states (damage, availability and outage) for each road category used in the road network characterization as described above. The resilience states from the road characterization can be derived with the aid of ArcGIS.

The resilience states from the road characterization can be spatially mapped and displayed in GIS, so that the whole network can be visually seen together. A description of the resilience state levels can be obtained in tabular forms as well. The results of the resilience assessment for the road network can be presented on individual maps for each resilience state, as listed below.

- Damage state, illustrating the severity of damage to the roads after earthquakes.
- Availability state, indicating how much access can be expected to be available after the earthquakes event.
- Outage state, indicating how long the availability for access may be impaired.

Moreover, this can identify better measures of redundancy to provide higher robustness in terms of performance that in turn provides the ability to undergo faster and higher performance after disasters.

IV. CONCLUSIONS/RECOMMENDATIONS

Managing the risks to roads requires an understanding of the network resilience to hazard events. We can map the current resilience of critical lifeline routes across the affected districts and capture this onto a GIS platform. The resilience can be characterized as damage, availability and outage states based on the concept of resilience developed for lifeline networks. The key outcome of the resilience assessment is estimation of duration that will be closed following a large earthquake, due to major landslides on roads.

This approach can assess the vulnerability of the road network in the affected districts and provides useful information that includes prioritizing the road network links in terms of their importance, which provides the basis for prioritizing risk assessment, response, and recovery and risk management. Moreover, we can plan for backup alternatives for emergency situation when other links fail should be considered important. This approach can be replicated to other districts as well.

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An ICT Based National Disaster Management Information and Communication System (NDMICS) for Effective Reconstruction and Disaster Management

Er. Bikash POKHREL

([1]Principal/Senior Lecturer Aryan School of Engineering, Purbanchal University, [2]Executive Member, Association of Engineering Colleges of Nepal, [3]Executive Member, Nepal Engineers' Association, 29th Executive Council, [4]MSc. Information System Engineering, Himalayan Institute of Science & Technology, Kathmandu, [5]BE Electronics & Communication, Khwopa Engineering College, Bhaktapur, Email: bikaspokhrel@gmail.com

ABSTRACT

Nepal is vulnerable to disaster risks from a range of hazards, including earthquake, floods, avalanches, glacial lake, landslides and river erosion. The inability of the proper Nepalese Emergency Response System to deal with the large scale catastrophes is clearly exposed, particularly during the Gorkha Earthquake in April 2015. Not only this, we are facing disaster such as flood, landslide every year it is very much essential for us to understand the importance to build an active and a holistic Disaster Management based on Information and Communication Technologies (ICT) is essential to reduce losses for disasters in future. ICT and Crisis, Disaster, and Catastrophe Management contribute to the disaster management and the public administration and public policy implications of ICT in building a resilient society.

This paper deals with the aspect of the analysis, design, development, deployment, implementation, integration, operation, use, or evaluation of ICT for any phase of the comprehensive disaster management cycle. These findings can be utilized by the Government of Nepal in the area ICT based State of the Art National Disaster Management Information and Communication System (NDMICS) that could be a milestone and a paradigm shift for the Disaster Management in Nepal.

Keywords: *Disaster Management, ICT, National Disaster Management Information and Communication System, National Disaster Communication Network, Emergency operation Center, Geographic Information System*

INTRODUCTION

Background

The recent earthquake of 7.9 magnitudes and several aftershocks caused devastating effect with the loss of 8,891 lives, 22,302 injured and thousands of homeless [1], [6]. The Disaster Management approach guided by the nation resulted with an uneven and unmanaged rescue as well as relief operations. This indicates the need for the changes in disaster management approach from reactive and responsive centric to proactive and holistic disaster management plan for the rebuilding and sustainable development of the nation. Nepal has to learn lessons from the past and prepare for the future for many other disasters, including an earthquake, following the global trend, the paradigm shift in disaster management.

Aim and scope of the study

The aim of this study is to identify ICT enabled holistic disaster management framework with an attributes such as multidimensional, consistent and dependable, dedicated and responsive, digital, State of The Art information and communication support infrastructure for the effective reconstruction and for the future disaster management.

The scope of this study is to elucidate the role of ICT infrastructure as a useful and most integral tool to manage various phases of the disaster.

ICT and Disaster Management

In order to establish an efficient and capable emergency response system on a regular basis, advance technology is to be introduced and appropriate equipment's are required. The use of ICT network and tools during various phases of the disaster management process has been very effective and systematic.

The use of ICT during several Disaster Management phases [7]:

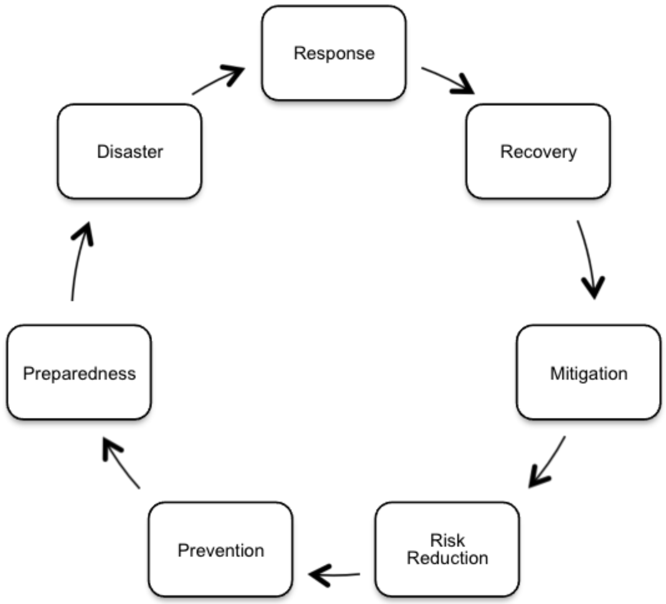

Figure 1: Various phases of Disaster Management [7]

Mitigation Phase

The activities performed before the disaster and so that the post-disaster effects can be minimized and these tasks are not critical with time. The collection of large volume of data (example use of Geographic Information System Database, GIS), populations details, open spaces, hospital trauma patients' capacities etc., disaster prone zones, fire and ambulance services etc. and the design of capable ICT network infrastructure capable of handling larger volumes of data and the connectivity among several groups of organizations, government line agencies etc. are to be performed during this phase.

Preparedness Phase

A dedicated network established for the dissemination of warning is the basic prerequisite. Accuracy is a very critical element in the information dissemination. The stage of disaster's early warning is improvising day by day, but still the earthquake disaster's early warning system requires wide distribution of information within few moments.

Response Phase

The response after the occurrence of any disaster event is quite time critical. Several logistical supports for the evacuation, relief materials, damage survey, equipment's and tools, human resources, fund requirements has to be easily approachable. Thus the basic communication between the response teams and the public is quite essential. The communication network has to be reliable, rapid, controlled and configurable for the effective disaster response operations in extreme conditions of infrastructure destruction, communication traffic peaks, mobile users, and sensitive data.

Recovery Phase

Even though the activities during this phase are less time critical, lot of work has to be done. On-site data related to the reconstruction, documentation of the lessons learnt for future reference. The outcomes will be used as feedback during the mitigation process where the previous records and databases are very crucial to avoid similar mistakes in the coming days. The proper use of the internet for the establishment of the communication link and for transfer of information is required.

Using the GIS platform during the entire phases of Disaster management

The use of advanced technologies based on satellite and Global Positioning Systems (GPS) embedded Geographic Information System (GIS) has the vital role to feed several stakeholders with the relevant information that is essential for formulating a holistic approach to perform data processing to produce crucial information during disaster management process. The real time information and the dynamic geographical map provided by the GIS system will be precious during the various stages of disaster management. GIS will be useful to plan the response team for identifying the appropriate routes for evacuation, unsafe infrastructure, and identify the fundamental supports required during the preparedness and response phases. Similarly the estimation of the amount of food supplies, tents, clothing, medicine and paramedical services, expected number of victims can be done with the use of information based on GIS [3], [2].

On the other hand the online observation and monitoring about the work progress, weather information can be viewed using GIS tools during the recovery phase. Infrastructure planning for the proper dissemination of disaster information is the proper combination of Information Communication technologies capable of responding adverse situations [3], [2].

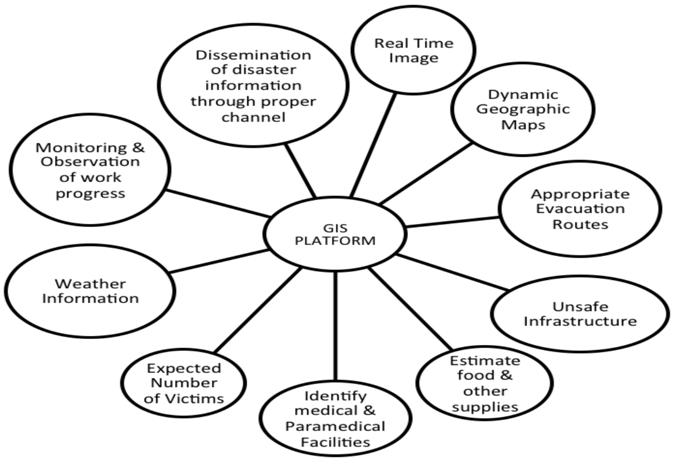

Figure 2 Use of GIS Platform on various phases of Disaster Management [2]

Existing Communication platform and the consequences

The telecommunication industry grew with the significance pace during the last decade. The service requires larger bandwidth. Several modes of communication such as radio, television, internet, fixed and mobile phones, SMS, are available for the information and warning. Government agencies like Radio Nepal, NTV also provided a reliable source of information during disaster.

Limitations of Existing Communication System for Disaster Management Work:

The most prominently used police networks operate in C band with the larger antenna requirements that are not compact and portable such as satellite terminals operating in Ku band. The police network is voice centric with a very limited data services. The nature of data during the disaster might be a video, voice or a data that requires maximum bandwidth.

Most prevalent Police Network operates in C-band, whereas, highly compact and portable satellite terminals are available in Ku-band. It is voice-centric with limited data handling capability, whereas, the needs for Disaster Management handling stakeholders are voice, video and data with adequate bandwidth. The number of communications and IT networks are operating at the National and regional level is in a standalone mode and there is need to connect them together for redundancy and interoperability. A government owned Government Integrated Data Center (GIDC) [5] is in operation, but an effective Data Fusion center is not established yet. So a data fusion at National Level along with GIS based applications is of overriding importance.

II. NATIONAL DISASTER MANAGEMENT INFORMATION AND COMMUNICATION SYSTEM (NDMICS)

The major difficulties faced during the Gorkha earthquake is due to the lack of early response, rescue and uneven distribution of relief materials, unplanned, uncontrolled and unmanaged international and national support, lack of proper tools and trained manpower for the rescue and relief, lack of appropriate reconstruction and disaster mitigation efforts, lack of proper databases based on vulnerability and risk assessment data. Same time due to lack of proper ICT based portal for the disaster management, the effect of the disaster was even more.

The Government of Nepal took several initiatives by prioritizing the disaster rescue operations and also the initiative for the formation of the **National Reconstruction Authority** as an apex body of the Government of Nepal. The time lag in the establishment of the authority delayed the overall reconstruction process. The Gorkha Earthquake 2015: Post Disaster Needs Assessment report, key findings has highlighted the several aspects associated with the reconstruction to highlight "Build Back Better (BBB)" slogan. The report indicates the importance of communications systems, recommending the priority for internet and telecommunication services in the disaster sites and to construct the towers in disaster effective districts that can be shared among various operators to ensure their services, need of public service broadcasting systems and disaster recovery integrated data center [6]. Promoting ICT or just raising the agenda of ICT use for the disaster management and recovery does not justify the ground base reality issues and it does not embrace with the global benchmarks. An ICT enabled framework to face all types of disaster cases and their holistic management with an attributes such as multidimensional, consistent and dependable, dedicated and responsive, digital, State of The Art information and communication support infrastructure as a National Disaster Management Information and Communication System (NDMICS) is a pressing need of Nepal. The scope of the NDMICS can be well defined as a useful and most integral tool to manage various phases of the disaster.

The various modules under the NDMICS are

National Disaster Management Information System (NDMIS)

The collection of large volume of data (example use of existing Geographic Information System Database, GIS), populations details, open spaces, hospital trauma patients capacities etc., disaster prone zones, fire and ambulance services, satellite images, hazard profile can be used to design of a capable MIS system capable of handling larger volumes of data and the communication among several groups of organizations, government line agencies. The data are archived with proper classification and with the connecting link to the data sources with the access mechanism developed as an appropriate search engines also the capability to access for different users. Data available from various sources such as government agencies, satellite images, GIS system, NGO/INGO's, social media etc. will be compiled to produce the information required for the Vulnerability Analysis and Risk Assessment (VA & RA) during the pre-disaster scenario and the Decision Support System (DSS) during the post disaster scenario [3], [2], [4].

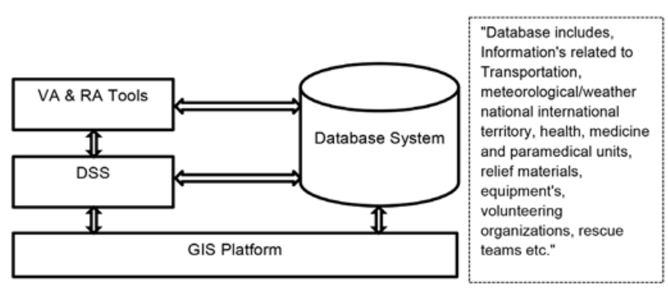

Fig 3: GIS Platform based NDMIS [4]

National Disaster Communication Network (NDCN):

The NDCN is used to disseminate the value added information among several stakeholders over the communication infrastructure and is capable of transmitting the imageries and several conventional formats of data such as voice, video and data. During the post disaster scenario, the terrestrial based communication network is most likely to be disrupted, which will hamper the response and relief works, NDCN will be equipped with the separate satellite based network that will help authorities to connect with the Emergency Operation Center's (EOC's) from all over the country using Very Small Aperture Terminal (VSAT) Network and Satellite phones assuring the reliability and guaranteed communication during the time of disaster. This ICT based communication infrastructure can be used to establish the proper communication to identify several logistical supports for the evacuation, relief materials, damage survey, equipment's and tools, human resources, fund requirements platform as quickly as possible during traumatic chaotic conditions everywhere with infrastructure destruction, communication channel traffic peak and excessive mobility of mobile users [4].

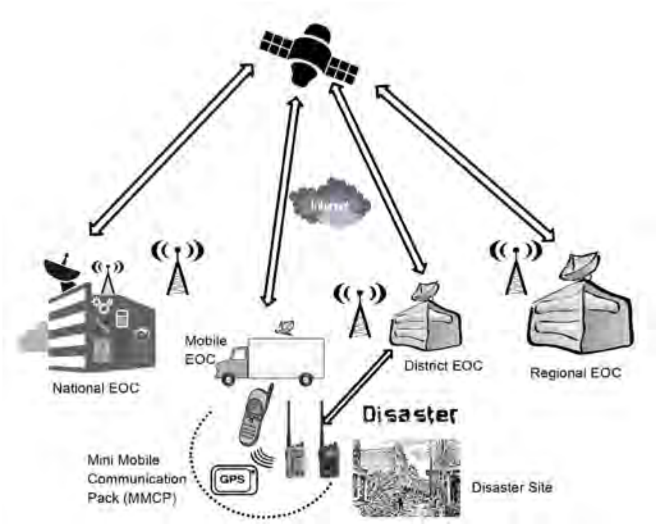

Figure 4: National Disaster Communication Network (NDCN) (Source: Authors analysis)

Emergency Operation Centers (EOCs):

The national disaster response force would require several static and dynamic EOC's operating in National, Regional and District level. EOC functions as a major unit for the control and command information required for the sound management of the disaster in all phases. Immediately after the function as the primary recipient of disaster information, EOC's would start operating full-fledged round the clock [4].

Connectivity: They will have the connectivity and there will also be the facility of accessing the required databases. Both the satellite and terrestrial Communication Channels will be utilized to establish the Communication among several EOC's and stakeholders.

Database & Applications Development: The pre, during and post-disaster related applications are required at EOC. The national EOC will have the platform for the application development and a Data Fusion Center, which will access all the relevant data for both the spatial and non-spatial platforms such as GIS and also the disaster sensitive and hazard specific data from various line agencies and ministries.

There are two major applications developed. For the pre disaster consequences, the major application developments are the Vulnerability Analysis and Risk Assessment System (VA & RA) and the Decision Support System (DSS) for the post disaster consequences that helps for relief and rescue management and assessing the damage and loss.

The database includes transportation facilities (airport, roadways), meteorological conditions, weather parameters (rainfall, snowfall, windstorm etc.) national international territory, health, medicine and paramedical units, relief materials, equipment's, volunteering organizations, rescue teams etc.

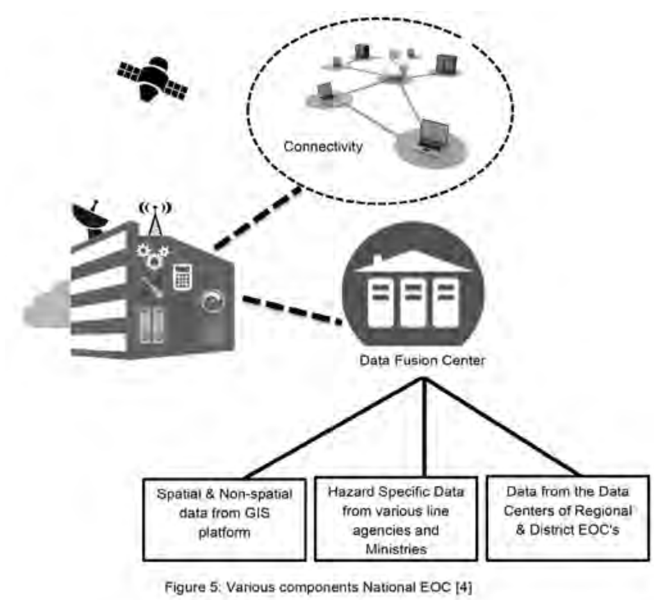

Figure 5: Various components National EOC [4]

Last Mile Connectivity

The incident area in case of disaster is the last mile connectivity. To establish the last mile connectivity the fundamental approach of NDCN is to guarantee the communication access to the incident area as quickly as possible. The communication will have to be established in various phases in the graded manner i.e. immediate after the disaster voice communication, after reaching the disaster site subsequent scaling up to video and data communication with a time frame[4].

1st Phase: A portable Mini Mobile Communication Pack (MMCP) consisting satellite phone sets, VHF radio receiver sets will have to be accessed by the district authorities. A portable MMCP will have to be sent to the disaster site by the district authority.

2nd Phase: National Disaster Response Teams with the satellite phone (INMARSAT/INSAT), VSAT terminals, VHF system, Cameras, Computing devices etc. in a Mobile Emergency Operation Center will have to be provided. MEOC will be transported to the site and establish the full-fledged communication with other EOC within three hours after reaching the site in a graded manner. MEOC will have to communicate with other EOCs, local authorities and the Disaster Response team with the help of satellite communication along with the photographs, audio, video and data communication.

3rd Phase: The restoration of the telecommunication services at the disaster site within a week. This may require the assistance of the telecommunication service provider.

Technological Challenges for Implementation of NDCN

To establish a National Disaster Communication Network (NDCN) involves following technological challenges:

Apart from the terrestrial links, immediate requirement of Satellite and VHF based Communication support with sufficient bandwidth. Graded manner Communication scheme, i.e. initially immediate after the disaster, basic voice communication, and after reaching the disaster site, immediate building the video and data communication in a resilient manner with the support for redundancy and diversity.

To establish a Disaster Management Information System, it is required to collect data and form a data fusion center at National level, and data centers at regional and district levels at the respective EOCs. It requires comprehensive ICT infrastructure for the effective management.

The technology is emerging, and the challenge is that the network infrastructure and hardware/software complexity at the NDCN may not be easily integrated with the satellite networks and local networks so the enough provision for the dynamic up-gradation of the system is required.

CONCLUSION

It is essential to develop guidelines for the NDMICS by the Government of Nepal. All stakeholders and various segments of administrations need to perform their irrespective duties to get the guidelines convert into the respective action plan. For the implementation it is essential to be well equipped with both the technical and financial resources. And the execution finally for the formation of an ICT based platform required for achieving National Reconstruction Authority (NRA's) goal for the holistic disaster management.

This paper focus on building disaster resilience society with the help of a State of the Art, Knowledge based, National Disaster Management Information and Communication System (NDMICS) that will use GIS based Information (imageries, video, audio and data) and develop a communication link among various stake holders ensuring the timely sharing of data and information required for the holistic disaster management. The findings also indicate the essence of establishing a National Disaster Communication Network (NDCN) using updated technology capable of transferring value added information and data among the various entities involved during the various phases of the disaster management.

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Cultural Continuity in Post Gorkha Earthquake Rehabilitation

Kai WEISE
President, ICOMOS Nepal
paharnepal@hotmail.com

ABSTRACT

A 7.8 magnitude earthquake struck Nepal on 25 April 2015 followed by numerous aftershocks devastating hundreds of villages and a large number of the historical monuments. The destruction was extensive and the response in respect to cultural heritage needs to be coordinated. This was done by the Department of Archaeology supported by UNESCO through the Earthquake Response Coordination Office (ERCO). This paper sets out to provide an overview of the response to the destruction to cultural heritage.

During the past years, the Kathmandu Valley World Heritage Site has been preparing for the earthquake that was to strike Nepal. Clearly, one is never fully prepared. After the initial response phase and with the onslaught of the monsoon rains, rehabilitation planning began. The challenge of agreeing to an appropriate approach to rehabilitation has been the major issue and the cause for the delay in getting the rehabilitation Conservation Guidelines 2072 finalized. One of the largest contentions has been in respect to the proposed use of modern technology and materials to strengthen monuments. This shows that there is a major lack of knowledge of traditional non-engineered structures in most of the local structural engineers.

Rehabilitation however cannot just focus on rebuilding the monuments. The tangible is an expression which has over the centuries always been damaged and rebuilt. It is therefore not necessarily the tangible aspects that are critical but whether the community has the motivation and skills to ensure rehabilitation. Sustainability is achieved when the community ensures continuity and the concept of continuity of culture become the basis for reconstruction.

Keywords: *cultural continuity, earthquake response, heritage conservation, seismic engineering, resilient communities*

I. INTRODUCTION

Throughout history it was through recurring tests of endurance and trial that communities learnt to improve their cultural expressions and create a resilient cultural environment. Similarly in the Kathmandu Valley during the early part of the second millennium CE the traditional buildings were first adapted to fire hazards by introducing a system of brick fire walls that stopped the spread of fires from one building to the next. These brick and timber buildings were then phase wise adapted to withstand earthquakes by inserting wooden ties and pegs to dampen the seismic forces. Innovative solutions were used to ensure structural stability against earthquakes for example by building square timber temples laced with wooden bands on high stepped plinths that functioned as base isolations.

One kept hearing of the great Nepal Bihar Earthquake of 1934 and regularly experienced smaller tremors. This raised concern that the next great earthquake would have devastating effect on the Kathmandu Valley which had over the decades developed with uncontrolled urbanization. During the past years the Kathmandu Valley World Heritage Site has been preparing for the earthquake that was to strike Nepal (Weise 2015). The rough assessment by the geologists, that there was a slip deficit along the section of the Himalayan arc in western Nepal and an earthquake was eminent, came to be true. On Saturday, 25 April 2015 a 7.8-magnitude earthquake did strike Nepal, with the epicentre about 40 kilometres northwest of Kathmandu. Even though there were several hundred aftershocks, with one of 7.3-magnitude on 12 May 2015, the geologists have warned that not sufficient energy has been released and there might be another big one right around the corner.

The rehabilitation of the cultural heritage will be a test for the motivation and resilience of the cultural communities (Gutschow 1982, Nepali 1965, Sharma and Shrestha 2007). Considering the fact that the last major earthquake was over eighty years ago, there has been a generational information gap. The lessons that would have been learnt from previous experiences seem to have been forgotten. The great difference in the response to this earthquake has been the introduction of engineered structures that have replaced the centuries of traditional knowledge. One of the greatest challenges in restoration of monuments has been the absolute lack of understanding and of confidence in the structural performance of traditional structural systems.

II. PREPARING FOR THE GREAT EARTHQUAKE

In anticipation of the next big earthquake preparations were undertaken. Several key government officials went to training courses on disaster risk management. Regular community meetings were held. International training courses on disaster risk management for urban heritage was carried out in Kathmandu. In November 2013 a week long symposium "Revisiting Kathmandu" was organized by ICOMOS Nepal, ICOMOS Scientific Committee for Risk Preparedness, UNESCO and Department of Archaeology with support from the local site managers. (Weise 2015) The eightieth anniversary of the 1934 Great Nepal Bihar Earthquake started the countdown to the next big earthquake. The international symposium was in preparation to the countdown, linking the discussions between authenticity, management and community with disaster risk reduction.

Collapsed structures of the Hanuman Dhoka Palace Museum, Kathmandu © Kai Weise

One is however never fully prepared for such a formidable display of natural forces. Even though the question of additional strengthening of monuments might be controversial for most conservation experts, the need for maintenance and restoration was clearly witnessed. The system and procedures for immediate response should also have been established.

III. THE EARTHQUAKE DID STRIKE – IMMEDIATE RESPONSE

On Saturday 25 April 2015 just before noon the 7.8 magnitude earthquake struck. It was an earthquake that seemed to specifically damage vernacular buildings and historical monuments. Villages in 39 districts were affected with about half a million houses collapsing and a further quarter million being severely damaged. The most badly affected were eleven districts within the area spanning between Gorkha and Dolakha. Listed monuments were affected in 20 districts with 190 being recorded as having collapsed and 663 partially damaged.

The immediate response after the earthquake struck was to look for survivors. There were locations where special events were being held on the Saturday and when the structures collapsed, large numbers were buried. The phenomenon we could observe in most heritage sites in the Kathmandu Valley was that people instinctively contributed to salvaging and safeguarding the components of the collapsed and damaged monuments.

Community and armed forces helping salvage from collapsed temple in Patan © Kai Weise

The first coordination meeting took place at the UNESCO Kathmandu Office just five days after the earthquake, together with the various authorities and stakeholders, as well as organizations involved in the cultural heritage sector. The following week the Earthquake Response Coordination Office (ERCO) was established at the DOA. To ensure that all stakeholders for the preservation of historical monuments were working together with a shared approach the first two months were declared a response phase. This meant that everything possible needed to be done to prepare the heritage sites for the onslaught of the Monsoon. The main building materials such as wood, brick, roofing tiles and stone along with the artefacts and ornaments which were lying in a pile of rubble needed to be salvaged and stored. Damaged structures needed shoring and protection from the rain. It was decided that a proactive approach would be applied to the World Heritage properties, the sites on the Tentative List and the monuments on the classified list of the Department of Archaeology (Government of Nepal 2007). The remaining monuments would need to be left to the communities and local authorities for them to restore, however providing them with support and expertise where required.

IV. STRATEGIC PLANNING – PHASING RECONSTRUCTION

The earthquake response in respect to cultural heritage has been strategically segregated into phases. The first phase of two months was exclusively reserved for earthquake response which involved preparing the affected cultural heritage for the oncoming rains. This was followed by the monsoon season when the rains don't allow much construction work to be carried out. The efforts of the response phase were being monitored especially in respect to the effects of the rains on damaged monuments. The next phase focused on planning and research is comprised of five approaches.

Archaeological investigations on the foundations of Kastamandap found them to be in perfect condition © Kai Weise

Legal Approach [1]: There was an immediate need for the preparation of policies and guidelines. The Post Earthquake Rehabilitation Policy for Cultural Heritage was formulated by a team from the Earthquake Response Coordination Office (ERCO) and was submitted to the ministry for adoption. The Conservation Guidelines for Post 2015 Earthquake Rehabilitation (Conservation Guidelines 2072) were formulated in line with the policy. The guidelines also look at sites, monuments and historical buildings over time and introduce provisions for maintenance and renewal. This will be augmented with a document defining rehabilitation processes and a related checklist.

Research Approach [2]: Extensive research is required to better understand the complexity of the sites in historical as well as technical terms. Detailed structural and material research of the damage on the monuments such as the Swayambhu Mahachaitya and Hanuman Dhoka palace will help retain most of the original structure. Geological research is required to study stability of slopes and soil conditions.

Urban archaeology has investigated the foundation of collapsed temples and cross-sections of Durbar Squares to better understand the damage in the substructure due to earthquake and also the chronology of these sites. Furthermore, the safeguarding and sorting of salvaged artefacts from the Hanumandhokha Durbar square is being carried out in systematic manner, detailed inventories need to be prepared in order to understand how much of the materials can be reused. Along with this, the conservation of mural paintings is also being carried out.

Planning Approach [3]: Several of the complex cultural heritage sites and historic settlements will require specific "Rehabilitation Master Plans". These will be prepared for Hanuman Dhoka, Swayambhu, Changu Narayan as well as Sankhu, Nuwakot and Gorkha. The Rehabilitation Master Plan will help clarify the multitude of involved donors, managers, supervisors and the communities. It will also define how and over what time period the reconstruction will realistically be carried out. This will require procedures for supporting the restoration of settlements and traditional dwellings.

Practical Approach [4]: The rehabilitation and reconstruction of the monuments will only be possible if we have knowledgeable and skilled artisans. The master crafts-persons must be identified and acknowledged. They must be seen as "living national treasures" as the Japanese do for "keepers of important intangible cultural properties". The system of apprenticeship must immediately be expanded to ensure that sufficient artisans are trained to allow for the restoration of the tangible heritage. This would have to be coordinated with the procurement of appropriate materials. The government must also change the system of tendering and giving

sorting of salvaged wooden elements from Hanuman Dhoka Durbar Square © Kai Weise such delicate work to the lowest bidder. A system of prequalification, inclusion of skilled artisans and quality control must be introduced.

Information Approach [5]: The damage assessment is linked to the collection of a lots of informations which will be closely linked to the preparation for post-earthquake rehabilitation. This will require a systematic database and easy access to information. For this it was decided to establish a database system using ARCHES as the information platform. The process of establishing the database, working on the adaptation of the software as per local requirements and the establishment of inventories has been challenging.

V. CONSIDERATIONS FOR REHABILITATION AND CULTURAL CONTINUITY

The rehabilitation of the communities and the cultural heritage will take many years. An initial six year plan is being prepared so that certain targets are met by July 2021. Though there will be a formal system of carrying out the rehabilitation of many of the heritage sites, it will be the informal interventions by the community that will be most critical. The response in most areas has been controlled and communities have been obstinate not to give in to the dire circumstances. It is this spirit of the communities that will be vital to ensure that recovery will take place rapidly.

The clash between modern engineering interpretations and traditional non-engineered knowledge seems to have come to a head. Reconstruction is being proposed using modern engineering parameters without even properly assessing the performance of the traditional structure or understanding the reason for the damage or collapse. Why did the central timber mast of the Bauddhanath stupa get damaged? Was it because the base of the harmika had been casted using cement concrete? Was the brick masonry in mud mortar in the plinth of Pratappur temple shattered by the recent reconstruction of the superstructure in the more rigid lime-surkhi mortar? Did the upper part of the nine-storey palace at Hanuman Dhoka collapse because of the fracturing of a reinforced cement concrete tie-beam introduced in the 1970s restoration? There were several tiered temples that collapsed that had concrete tie beams. What was the cause of the collapse? Even the collapse of Kastamandap raises questions concerning earlier interventions rather than design faults after the archaeological investigation.

The lack of understanding of the traditional structures is alarming. In the rush to reconstruct certain monuments, simplified procedures are used. It is important to understand that the restoration project of Kastamandap in the 1970s covered up the fact that one of the main four central posts was not resting on a saddle stone. At the base of many of the posts the tendons were missing and the holes in the saddle stone filled. The structure probably collapsed because it was not locked to the plinth and was standing on only three out of four main posts (Coningham 2015). We also know that the structure did not collapse immediately and many would have survived if they would have moved away. Further research and documentation is required to fully understand what happened.

Great expectations are placed on intangible heritage as the vehicle for cultural continuity. The rehabilitation of the cultural sites will depend more on the strength of the intangible than that of the tangible heritage. We talk of strengthening the monuments to withstand the impact of earthquakes. There are misconceived ideas floating around promoting the use of modern technology and materials to ensure resilient structures. Over time it is not the structures that will persist. Cultural continuity can only be ensured through the knowledge and skills of the community being passed on from generation to generation.

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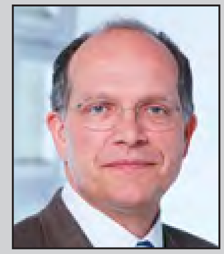
Vulnerability of The Nepalese Building Stock During the 2015 Gorkha Earthquake



Max DIDIER
PhD Candidate, Swiss Federal Institute of Technology (ETH) Zurich, Dept. of Civil, Environmental and Geomatic Engineering, 8093 Zurich, Switzerland, didierm@ethz.ch

Siddhartha GHOSH
Professor, Indian Institute of Technology Bombay, Dept. of Civil Engineering, Mumbai 400076, India, sghosh@civil.iitb.ac.in

Bozidar STOJADINOVIC
Professor, Swiss Federal Institute of Technology (ETH) Zurich, Dept. of Civil, Environmental and Geomatic Engineering, 8093 Zurich, Switzerland, stojadinovic@ibk.baug.ethz.ch

ABSTRACT

Updated building fragility functions can be used to quantify the seismic vulnerability of the Nepalese building stock. In a first step the building stock is categorized into different building typologies. Fragility functions for the different building types are derived using damage probability matrices for Nepal. The Rapid Visual Damage Assessment (RVDA) database from the Nepal Engineers' Association (NEA) is then used to update the fragility functions with damage data from the April 25, 2015 M_w 7.8 Gorkha earthquake. The obtained updated fragility functions can be used to quantify the risk of the building stock towards potential future seismic events and to analyze possible risk mitigation measures. The following study presents first preliminary results based on the processing of the RVDA database.

Keywords: *vulnerability, fragility functions, Gorkha earthquake*

I. INTRODUCTION

A devastating M_w 7.8 earthquake hit Nepal on April 25, 2015. The mainshock with an epicenter located approximately 80km north-west of Kathmandu in the Gorkha district, was followed by a series of aftershocks, including the M_w 7.3 aftershock on May 12 with an epicenter east of Kathmandu, close to the border to Tibet. About 9000 people lost their lives, 22'000 were injured and more than 750,000 buildings were damaged or destroyed [1]. Many people lost their homes and were forced to move to emergency shelters. The earthquakes caused damage to the building stock, as well as to the different civil infrastructure systems. Residential buildings, schools, heritage structures (e.g. temples) and hospitals suffered from severe damage and many were irreparably damaged. Civil infrastructure systems like the electric power supply system, the water distribution network or the cellular network were affected and their service supply capacity was markedly reduced [2,3]. The recovery of some parts of these networks could not be achieved to date; blackouts and shortage of service have ongoing negative impacts on the communities in Nepal. The amount of damage caused by the 2015 Nepal earthquake series is not only due to the magnitude of the mainshock and several of the aftershocks, but as well to the limited application of seismic construction standards in Nepal. The Nepalese building stock includes a large amount of buildings built using fragile materials and weak construction techniques, like the traditional mud mortar rubble stone houses found mainly in the more rural regions. The absence of basic seismic design or detailing, in combination with the poor adherence to the building code, adds to the vulnerability of the built environment in Nepal.

Despite these shortcomings, updated building fragility functions can be used to quantify the seismic vulnerability of the Nepalese building stock. In a first step the building stock is categorized into different building typologies. Fragility functions for the different building types are derived using damage probability matrices for Nepal. The Rapid Visual Damage Assessment (RVDA) database from the Nepal Engineers' Association (NEA) is then used to update the fragility functions with damage data from the April 25, 2015 M_w 7.8 Gorkha earthquake using the Bayesian probability updating method. The obtained updated fragility functions can be used to quantify the risk of the building stock towards potential future seismic events and to analyze possible risk mitigation measures. The following gives an introduction to the composition and the main types of buildings of the Nepalese building stock and shows the preliminary results of the analysis of the 2015 RVDA damage database.

II. VULNERABILITY OF THE NEPALESE BUILDING STOCK

The Nepalese building stock can be divided into different occupancy and building construction types [3]:

- Residential: including adobe, brick in mud, brick in cement, timber and reinforced concrete residential buildings
- Industrial: including small industries and medium/large industries

- Commercial buildings
- Critical buildings (non-commercial): including hospitals and schools

The distinction of the different occupancy and building types is necessary in order to account for the magnitude of the impact of damage to different building functions and for the varying robustness of the structures. Reinforced concrete buildings are, for example, often expected to be more robust than traditional mud mortar rubble stone houses; the consequences of damage to a hospital are usually larger than those to a single family house. The seismic behavior and fragility of the different building types can be described by fragility functions.

Fragility functions express the probability of a given building type to reach a certain (or higher) damage state, as function of a given intensity measure. They can be derived in four different ways [4]: expert judgment-based, analytical, empirical, and from hybrid methods. In the following, lognormal fragility functions, expressing the damage state probability, depending on the peak ground acceleration (PGA) at the building site are computed for the different building types listed above using a hybrid method.

Using the damage matrices for the predominant building types in the Kathmandu Valley, provided by [5,6], and the maximum likelihood method as proposed by [7,8], lognormal fragility functions for partial damage (DS2) and complete damage (DS3) can be computed.

The parameters of the obtained fragility functions for Nepalese adobe, mud bonded, cement bonded and RC frame buildings are given in Table 1. They can be employed for the different occupancy types, taking into account information given by [9,10]. Due to the lack of available data for Nepalese timber houses, the fragility functions given, for example, by [11] for "wood, light frame (W1)" can be used to represent the damage probability of the timber houses.

Table 1 – Lognormal fragility function parameters conditioned on PGA, with median λ and log standard deviation ζ, adapted from [3,12], for building types in Nepal.

Building type		Typology	Fragility function DS2		Fragility function DS3	
			ζ	λ	ζ	
Residential	Adobe	AH	-1.183	1.094	-1.187	1.095
	Brick in mud	BM	-0.970	0.950	-0.830	0.967
	Brick in cement	BC	-1.026	0.947	-0.284	0.827
	Reinforced concrete	RC3	-0.582	0.932	0.078	1.114
	Timber [11]	TH	-1.079	0.640	-0.051	0.640
Industrial	Small industries	BC	-1.026	0.947	-0.284	0.827
	Medium/large industries	RC4	-0.808	0.810	-0.197	0.989
Commercial		RC3	-0.582	0.932	0.078	1.114
Critical	Hospitals	BC	-1.026	0.947	-0.284	0.827
	Schools	BM	-0.350	1.467	-0.883	0.861

The accuracy of the risk quantification of the building stock of a given community, using fragility functions, is however limited by several factors. First of all, it is often not possible to obtain exact exposure data. The exact exposure data, in terms of the number of buildings, and their exact location. Secondly, the fragility functions, provided in literature, are often mean fragility functions, obtained for several realizations of the evaluation of the seismic robustness for a given building type. Additionally, the fragility functions proposed above are representative for aggregated classes of buildings. The single buildings may have a more or less pronounced variance in their seismic performances, depending, for example, on the quality of workmanship or the quality of the used building materials. Often, fragility functions that take into account all regional characteristics of a certain building type are not available and, therefore, they need to be substituted in a risk assessment studies by fragility functions for building types with similar characteristics, coming from other regions. The fragility functions used from [11] for timber houses might, for example, slightly overestimate the seismic performance of timber houses in Nepal, as they are derived for low seismic code timber houses in the United States. In order to reduce the epistemic uncertainty to a minimal degree, an individual fragility function for every single building would be needed. However, this is very difficult to do in practice.

III. UPDATING THE FRAGILITY FUNCTIONS USING THE NEA RVDA DATABASE

Updating (prior) fragility functions from literature with data obtained from earthquake damage surveys (e.g. through RVDA), can help to better take local vulnerability parameters into account, and thus better represent the seismic behavior of the buildings in a certain region, district or city. To update the fragility functions presented in Table 1, the RVDA database, assembled by the NEA after the April 25, 2015 Gorkha earthquake, is used. The RVDA, done by the NEA immediately after the earthquake, provided the trained evaluators with a paper form, permitting to determine the safety of buildings in the areas affected by the disaster. The form is divided into 5 different parts (Figure 2). The first part provides general information on the inspection, including information about the inspector, the inspection date and time, as well as the type of the assessment (exterior only, or interior and exterior). The second part contains a description of the building, including its location, type of construction, type of floor and roof and occupancy type. However, no data about the building height (number of floors) and the date of construction are collected. The third part comprises the evaluation of the damage of the building. The damage was first rated using 6 criteria on a 3-level damage scale (minor/none, moderate, severe damage). The conditions include, for example, the degree of collapse of the building or damage to primary structural members. The overall building damage is then, finally, judged on a scale from 0-100%, corresponding to no damage (0%) and complete damage (100%), respectively. The building is labelled according to the damage evaluation (green / yellow / red placard), signaling if the use of the building is safe and unrestricted, or unsafe and, therefore, (partially or completely) restricted. Recommendations for future actions can be included in the assessment.

Figure 1: Districts of Nepal with RVDA data (yellow shaded) (underlying map from [14])

Rapid Evaluation Safety Assessment Form

Inspection

Building Description

Type of Construction

Type of Floor

Primary Occupancy

Type of Roof

Evaluation

Posting

INSPECTED (Green placard) RESTRICTED USE (Yellow placard) UNSAFE (Red placard)

Further Actions

Figure 2: Rapid evaluation safety assessment form [13]

More than 40’000 buildings have been evaluated after the earthquake, using the presented form. The paper forms were manually digitalized by the NEA. In order to use the damage data in the database, a pre-processing was necessary [12]. Accuracy of spelling and data entry needed to be verified and corrected, where necessary. Incomplete datasets were discarded (e.g. missing information about the construction type). In a first step, the correct spelling and assignment of the districts needs to be verified, in order to assign the correct PGA values to the different building locations. The database contains damage data from the following districts and municipalities (yellow shaded in Figure 1): Banepa (municipality in Kavrepalanchok), Bhaktapur, Chitwan, Dhading, Dolakha, Gorkha, Kathmandu, Kavre, Kavrepalanchok, Lalitpur, Lamjung, Makwanpur, Nuwakot, Sindhupalchok, Tanahun and Tokha (municipality in Kathmandu).

In a subsequent step, the buildings are categorized into the different building typologies, as used in Table 1. This step is necessary in order to compare the derived fragility functions to the observed damage data from the earthquake. In total five building types are used for the updating: adobe, brick/stone in mud, brick in cement, RC frame and wood frame.

The assessment of damage, as proposed by the form, leads to some difficulties in the processing of the data: the qualitative evaluation of the damage, using the 6 proposed criteria, is not always coherent with the quantification of the total building damage (i.e. buildings that had no damage using the proposed criteria were classified overall as severely damaged and vice-versa). For many buildings, no total damage evaluation was assigned by the evaluator. To assign a damage state, the following procedure was used [12]: 1) if an estimated building damage was assigned by the evaluator, it is used to assign the damage state to the building. 2) If no estimated building damage is provided, a damage state was assigned to the building, according to the evaluation of the 6 damage criteria, as shown in Table 2.

Table 2: Assignment of DS according to the RVDA, adapted from [12]

Damage according to 6 damage criteria	**Estimated building damage**	**Assigned DS**
5 or more criteria rated as minor	None / 0-1%	DS1 (no/minor damage)
2-4 criteria rated as moderate	1-10%	DS2 (partial/ moderate damage)
	10-30%	
	30-60%	
3 or more criteria rated as severe	60-100%	DS3 (severe/ complete damage)
	100%	

A peak ground acceleration (PGA) value, according to the USGS Shake Map [15], was assigned to the different buildings. The PGA was assigned on a district level. This is done, due to two reasons: the building location data out of the assessment forms is very rough and the low quality of cartographic information of Nepal does not allow a more exact localization without a tremendous effort. The second reason is the limited resolution of the available Shake Map for the April 25, 2015 Gorkha Earthquake (Figure 3).

In the study presented hereafter, a preliminary version of the database was used, as the complete database containing all evaluated buildings was not yet available. This version of the database contains originally a total of 37'416 buildings. 2'389 datasets could, however, not be categorized into one of the 5 building types. For 9'265 buildings, no intensity measure and no damage was assigned, due to incomplete data. Nevertheless, 73.5% of the initial database was usable for the subsequent updating of the fragility functions [12]. Figure 4 shows the distribution of the damage states for the different PGA levels for RC frame buildings, obtained from the RVDA database. Similar plots can be generated for the other building types.

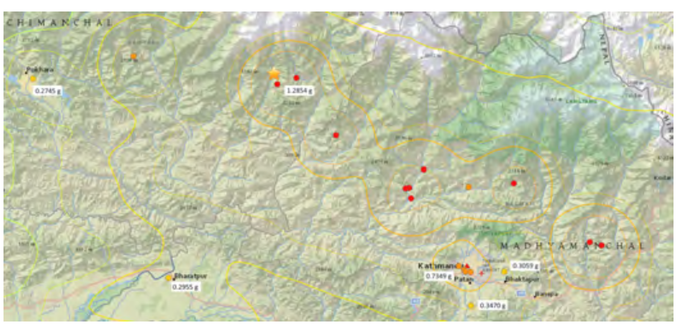


Figure 3: Shake Map of the April 25, 2015 April Gorkha earthquake [15]

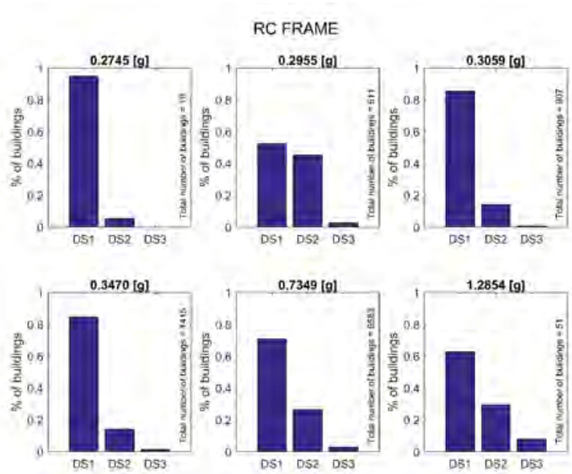


Figure 4: Distribution of the assigned DS, using the NEA RVDA database [12]

The obtained database is used to update the prior fragility functions (Table 1) employing Bayesian updating. The likelihood of the empirical data is multiplied with the prior distribution to obtain the target distribution. To find the parameters of this curve, a lognormal distribution is assumed and shifted to fit the target distribution [16]. The used data points, the prior, and the updated fragility curves are shown for RC frame buildings (Figure 5) and brick in mud buildings (Figure 6). The blue line corresponds to the prior fragility curve as presented in Table 1, the crosses to the data points and the orange line to the updated fragility function. The parameters of the updated fragility functions are given in Table 3.

Table 3 –Updated lognormal fragility functions conditioned on PGA, with median λ and log standard deviation ζ, adapted from [12]

Building type		Typology	Fragility function DS2		Fragility function DS3	
		λ	ζ	λ	ζ	
Resid-ential	Adobe	AH	-2.246	1.370	0.030	1.926
	Brick in mud	BM	-2.148	1.993	0.558	1.994
	Brick in cement	BC	-0.128	1.976	0.687	0.799
	Reinforced concrete	RC3	0.655	1.973	0.687	0.580
	Timber [11]	TH	-2.035	1.765	0.143	1.706

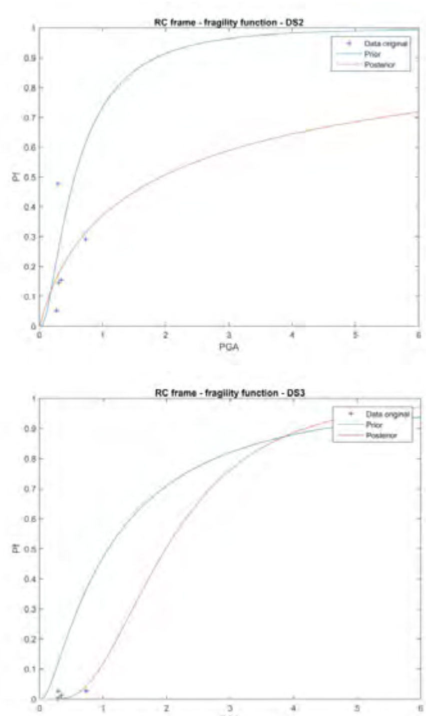


Figure 5: [illegible] pdated) and damage data from the RVDA database for RC frame buildings [12]

Figure 6: DS2 and DS3 fragility functions (prior and updated) and damage data from the RVDA database for brick in mud buildings [12]

IV. CONCLUSIONS

First preliminary results of the analysis of the RVDA database collected by the NEA and preliminary updated fragility functions are presented. Bayesian updating of prior fragility curves, obtained from literature using the data from the 2015 RVDA database, generally results in updated fragility curves that indicate lower damage probabilities than the prior fragility curves (e.g. DS2 fragility function for RC frame buildings in Figure 5 and DS3 fragility function for brick in mud buildings in Figure 6). The overall damage of the building stock might therefore be lower than would be initially estimated using the existing damage probability matrices.

To fully utilize the updated fragility functions, it is essential to enhance the collected data by reducing ambiguity, or by collecting additional data, such as the building height or the local soil type, or by obtaining more precise location data. This would allow for a more coherent classification of the buildings into building types and could improve the overall vulnerability assessment. The proposed updated fragility functions can be employed to more accurately quantify the risk of the Nepalese built environment in future earthquakes and help plan to engineer an increase of seismic resilience of communities in Nepal.

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Liquefaction of Soil in Kathmandu Valley From the 2015 Gorkha, Nepal, Earthquake

Mandip SUBEDI
PhD student, Department of Civil Engineering, Institute of Engineering, Pulchowk Campus, Nepal, mandip.subedi@gmail.com

Indra Prasad ACHARYA
Assistant Professor, Department of Civil Engineering, Institute of Engineering, Pulchowk Campus, Nepal, indragb@gmail.com

Keshab SHARMA
PhD student, Department of Civil and Environmental Engineering, University of Alberta, Edmonton, Canada, keshab.sharma@gmail.com

Kalpana ADHIKARI
Engineer, Department of Roads, Ministry of Physical Infrastructure and Transport, Nepal, erkadhikari@gmail.com

ABSTRACT

The Gorkha Nepal earthquake of moment magnitude M_w 7.8 occurred at 06:11 UTC on April 25, 2015, with the epicenter about 77 km northwest of Kathmandu at a focal depth of approximately 13 km. This motion induced several geotechnical effects such as landslides, liquefaction, lateral spreading and local amplification. Immediately following the earthquake, a field investigation was carried out in Kathmandu Valley. This paper provides first-hand observations of liquefaction and the associated effects. As the Kathmandu valley deposits are composed mainly of sand, silt and clay layers with a shallow ground water table, liquefaction is highly anticipated. Extensive liquefaction was also observed in Tundhikhel area during the 1934 Nepal-Bihar earthquake. However, unlike in previous major earthquakes, the liquefaction triggered by the M_w 7.8 Gorkha earthquake appears to be fairly limited and localized. This may be attributed to low amplitude ground motion and low ground water table at the time of earthquake event. Liquefaction-induced damage to structures in these areas was not found except buildings on some places tilted slightly. The liquefaction susceptibility maps prepared for the Kathmandu Valley were compared and verified with real liquefaction events. Tectonic and geologic setting of Nepal is briefly described; recorded ground motions from the event are featured; local site effect on the occurrence of liquefaction is also described briefly. Observed liquefaction with case studies is presented.

Keywords: *Gorkha Nepal earthquake; Earthquake damage survey; Liquefaction; Liquefactoin potential; Ground motion*

I. INTRODUCTION

The Gorkha Nepal earthquake of moment magnitude M_w 7.8 occurred at 06:11 UTC on April 25, 2015, with the epicenter about 77 km northwest of Kathmandu at a focal depth of approximately 15 km (USGS, 2015). Tremor was felt in Nepal, India, Bhutan, Bangladesh, and China. Two aftershocks of M_w 6.7 and 6.3 struck Nepal within 25 hours of the main shock. Another big aftershock of M_w 7.3 shook the region on May 12, having the epicentral location in the northeast of Kathmandu, and caused additional damage to rural towns and villages in the northern part of central Nepal. The spatial distribution of aftershocks, which extended 150 km to the east of the epicenter, suggests that the rupture propagated from west to east, thus producing severe destruction in Kathmandu, at approximately 80 km southeast of the epicenter. These seismic events in the central Himalaya were the strongest after the 1934 earthquake that was located northeast of Kathmandu.

Liquefaction can occur in moderate to major earthquakes resulting in severe damages to infrastructure. Loss of shear stress of granular material due to increased pore pressure due to earthquake shaking behaving as a liquid is generally called liquefaction. When this happens, the sand grains loose its effective shear strength and will behave more like a fluid.

Liquefaction potential is commonly associated with saturated sand and silty soil having low plasticity and density. However, Liquefaction potential is attributed to characteristics of earthquake event (e.g. duration of earthquake, amplitude and frequency of shaking, distance from epicenter), particle size distribution of soil, cohesion and permeability of soil, location of water table, and relative density etc (Subedi, 2012).

The deposition in the Kathmandu Valley is lacustrine and fluvial in origin with thickness up to 500 m (Sakai, 2001). The deposited sediments are made up of clay, silt, sand and gravel. Areas with loose sand deposits have a greater chance of liquefaction after the earthquake. After the devastating M_w 8.0 Bihar-Nepal earthquake in 1934, occurrence of liquefaction at many places in Kathmandu Valley was reported (Rana, 1935). Studies of previous major earthquake and characteristics of soil in Kathmandu Valley give evidence that significant damage to buildings and infrastructures occurred in Kathmandu valley as a result of widespread liquefaction (Piya, 2004 and Subedi et al., 2012). Liquefaction susceptibility was judged "high" and "medium" in a large area along the major rivers.

Field reconnaissance was carried out in the Kathmandu by the authors immediately after the main shock. Several geotechnical dynamic phenomena were triggered by the earthquake and this is the reason the authors considered that this could be a good case study with plenty of information available for the study of liquefaction, whose occurrence was forecasted in many areas of Kathmandu Valley.

II. TECTONICS ANG GEOLOGIC SETTING

The Himalaya was formed by the collision of Indian plate and Eurasian plate starting from 40 million years ago. The Himalayan arc, which marks an active boundary between Indian and Eurasian plates, has caused numerous major earthquakes of moment magnitude 7.5 or greater in past centuries (Bilham et al., 2001 and Ambraseys and Douglas, 2004).

The presence of numerous active faults clearly highlights the seismic hazard in this region (Fig. 1). Nepal is affected by major tectonic zones, namely Terai zone, Sub-Himalayan (Siwalik) zone, Lesser Himalayan zone, Higher Himalayan zone and Tibetan-Tethys Himalayan zone (Dahal, 2006 and Sapkota et al., 2013) as shown in Fig. 1. These tectonic zones are separated from each other by the active tectonic faults such as Main Central Thrust (MCT), Main Boundary Thrust (MBT), Main Frontal Thrust (MFT), and South Tibetan Detachment System (STDS) as shown in Fig. 1. The faults are produced by the collision of the Indian plate into the Eurasian plate (Decelles et al., 2001).

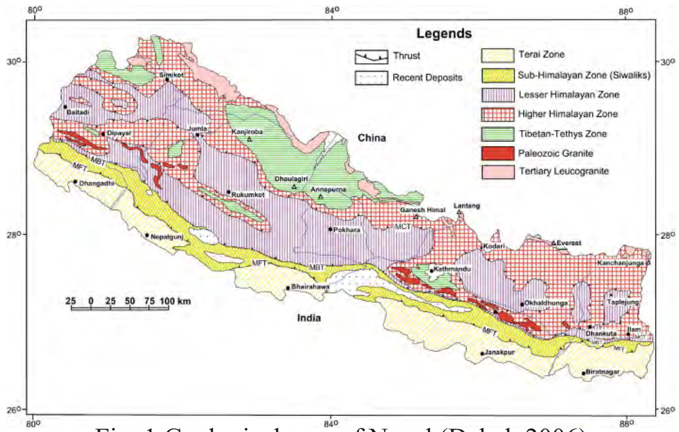

Fig. 1 Geological map of Nepal (Dahal, 2006)

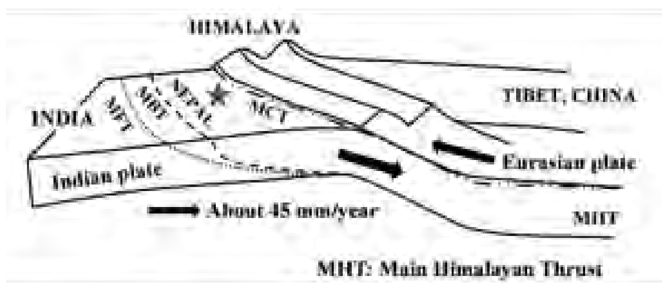

Fig. 2 Schematic illustration showing the relative motion and main features associated with the type of plate boundaries in Nepal (after Chiaro et al., 2015)

The 2015 Gorkha earthquake and aftershocks were the result of thrust faulting between the subducting India plate and the Eurasia plate to the north (Fig. 2), where the Indian plate converges with the Eurasian plate at a rate of approximately 45 mm/year towards the north-northeast, driving the uplift of the Himalayas and the Tibetan Plateau (Copeland, 1997). The Gorkha 2015 earthquake released large amount of energy accumulated along the main fault under Kathmandu Valley. The fault segment to the east is the one that ruptured in 1934, and the probability of another great earthquake there is therefore relatively low. The segment to the west and close to the 2015 rupture is the gap that has not ruptured since 1505, and the recent quake should increase the probability of the next great earthquake rupture there (Bollinger et al., 2016).

III. SEISMOLOGY AND GROUND MOTIONS

Ground motions data of 2015 Gorkha earthquakes were collected from United State Geological Survey (USGS) seismological station, located at Kanti Path, Kathmandu (N: 27.704323°, E: 85.313612°). Critical parameters for April 25 main shock and May 12 aftershock are summarized in Table 1. The recorded accelerograms for EW, NS and UD components of both M_w 7.8 main shock and M_w 7.3 aftershock are shown in Fig. 3(a) and (b). It is seen from Fig. 3 that the acceleration records contain the long-period components. It is believed that this shaking was primarily due to amplification of the local soil, lacustrine sediments several hundred meters thick in Kathmandu Valley.

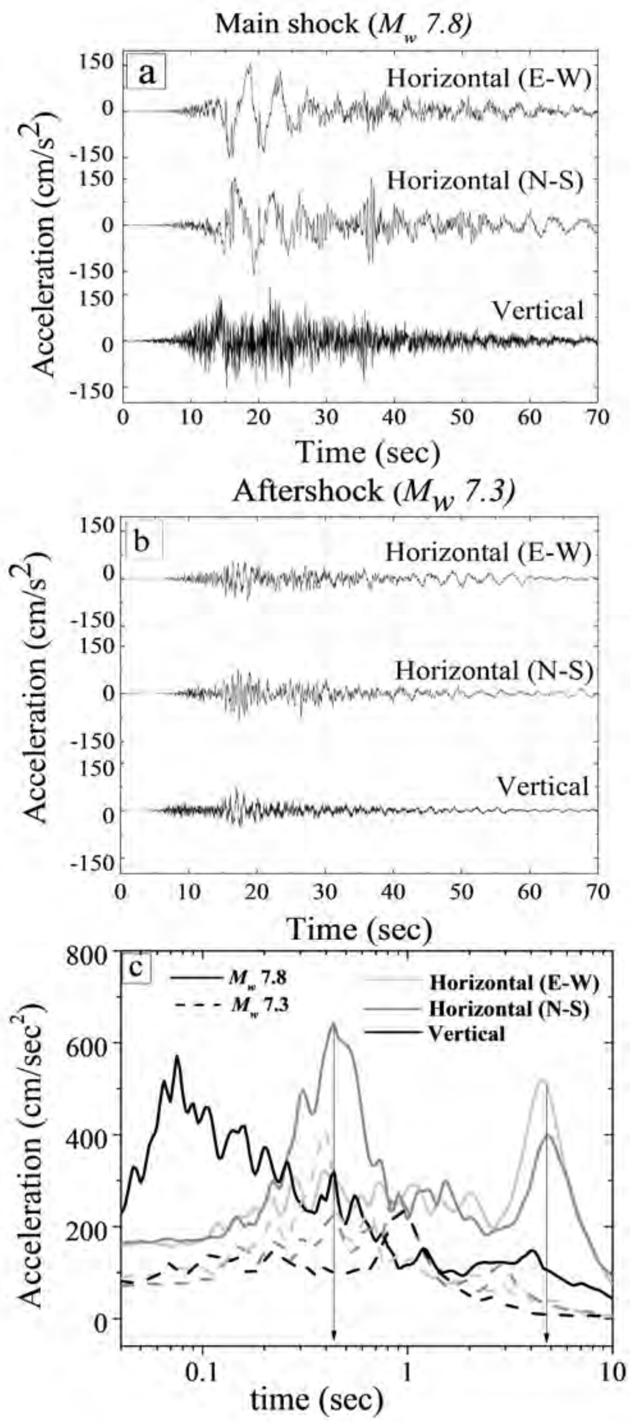

Fig. 3 Recorded accelerograms for the (a) Mw 7.8 main-shock, (b) the Mw 7.3 aftershock, and (c) 5%-damped spectral accelerations (SA) of the recorded accelerograms

However, due to lack of rock outcrop recordings and limited ground motion data, the soft sedimentary basin effects on the ground motion are poorly understood.

Peak ground acceleration (PGA) of the recorded ground motions is about 168 cm/s^2 and 80 cm/s^2 for the M_w 7.8 main shock and the M_w 7.3 aftershock, respectively. Peak ground accelerations recorded at KATNP station did not exceed the PGA estimates with 10% probability of exceedance in 50 years from the recent regional seismic hazard studies (JICA, 2002 and Ram and Wang, 2011). The USGS preliminary estimation of the PGA in the epicentral area was about 0.35g (USGS, 2015).

The 5%-damped response spectra of the recorded accelerograms for the M_w 7.8 main shock and the M_w 7.3 aftershock are calculated and compared in Fig. 3c. The horizontal components of the main shock and aftershock show the similar characteristics when the period is less than 1 s. Moreover, it is seen from the Fig. 3c that the NS component of the main shock has two prominent peaks, one at about 0.47 s and another one at about 5 s. Interestingly, both EW and NS components of main shock have peaks at about 5 s whereas the aftershock components do not show the peaks at about 5 s. The peaks at about 5 s of the main shock components may be attributed to main shock sources, and not due to local site effects. Details of the characteristics of ground motions recorded during the Gorkha earthquake can be found in Parajuli and Kiyono (2015), Martin et al. (2015) and Galetzka et al. (2015).

Table 1. Key characteristics of M_w 7.8 main shock and M_w 7.3 aftershock

Station	Date	Time (Local)	Dep-th (km)	NS (cm/s^2)	EW (cm/s^2)	Vert-ical (cm/s^2)	Latit-ude (N°)	Long-itude (E°)
KATNP	4/25/15	11:56	15	164	158	184	28.15	84.71
KATNP	5/12/15	12:50	15	87	72	75	27.84	86.08

IV. GEOLOGICAL ASPECT OF KATHMANDU VALLEY

The surface of Kathmandu valley is generally broad and almost flat except towards the boundaries of the valley, where rivers are deeply incised. Well-developed terraces, formed by erosion from rivers, are common in the valley. The main rivers in valley are Bishnumati, Dhobi Khola, Manohara, Hanumate, Nakhu Khola and Bagmati. All tributaries drain towards the center of the basin in the Bagmati River, which cuts the Mahabharat Lekh (hill range) in the south and drains the river water to the Gangetic plain through the Chovar gorge as the main drainage channel of the basin.

The Basement rocks of the Kathmandu valley is covered by thick semi-consolidated fluvio-lacustrine sediments of Pliocene to Pleistocene age (Fig. 4). The basin is filled by thick semi-consolidated fluvio- lacustrine sediments. These thick sediments are mainly derived from the surrounding hills by the ancient drainage channel system. The sediment consists of; arenaceous sediments composed of fine to coarse-grained sand with a small quantity of rock fragments, which are believed to have been supplied from the northern gneiss rocks. Argillaceous sediments composed of clay and silt resulting from the erosion of limestone and phyllite, which are exposed in the eastern, southern and western mountainous areas. Lignite and diatomite produced from the lake sediments. Agglomerate of boulders and gravel with a clayey and silty matrix in the southern basin de- rived as debris flow from the southern hills (Sakai, 2001 and Piya, 2004). The shear wave (V_{30}) velocity of the soft sedimentary deposits in Kathmandu Valley ranges from 160 m/s to 300 m/s and ground amplification may range from 2.0 to 8.0 (Chamlagain and Gautam, 2015).

V. GEOTECHNICAL ASPECT OF DAMAGES

Kathmandu valley was highly affected with instrumental intensity of VII and VIII and some parts even lay on IX and X (Fig. 5). Higher intensity was observed in Kathmandu due to non-engineered construction and local site effect due to thick soft sedimentary deposits in Kathmandu Valley. Towns such as Gorkha, Abukharehani, Mugling, Malekhu, Gajuri and Naubise located near the western extent of the fault plane showed much less destruction to man-made environment as opposed to the Kathmandu Valley area (Sharma, 2016).

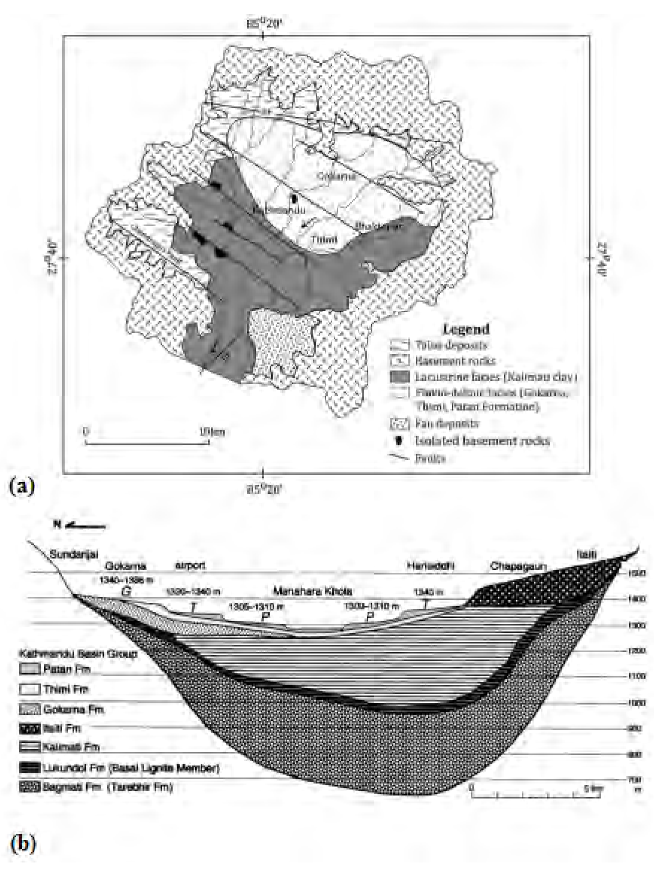

Fig. 4 (a) Geological map of Kathmandu Valley (b) North–south cross-section of the valley (Sakai, 2001)

The earthquake led to several ground failures such as subsidence of walkways, fissures, slope failures and liquefaction. However, the geotechnical failures in Kathmandu Valley, triggered by the *2015* Gorkha

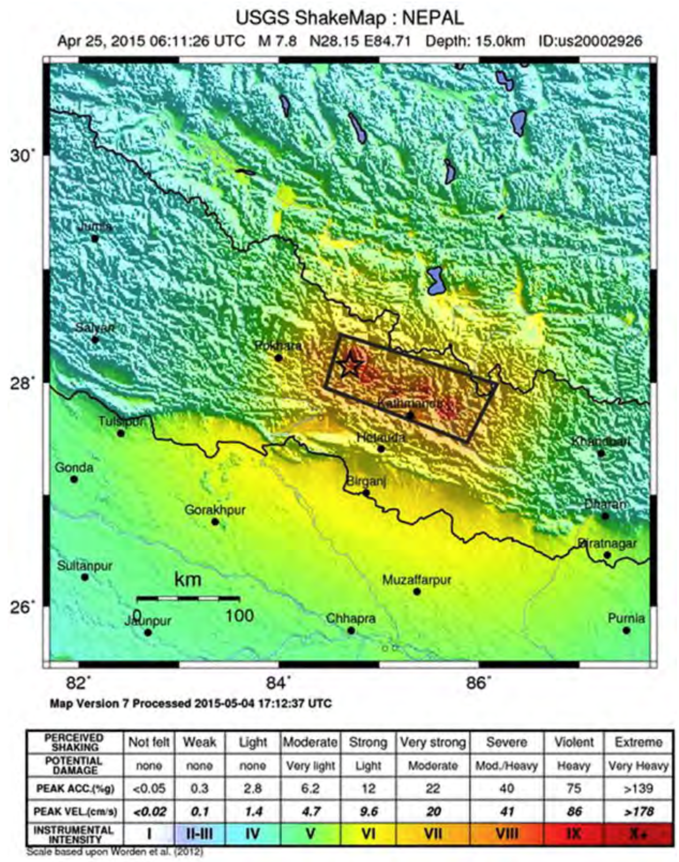

Fig. 5 Estimated MMI intensity in central Nepal from the M_w 7.8 main shock

earthquake seem to be limited and localized. There was only nominal damage due to main shock to roads, bridges, retaining wall, and life line services. All lifeline services were revived within couple of days after April 25 earthquake. Particularly, damage to roads was minimal and was localized to the eastern edge of the Kathmandu Valley. Larger fissures and settlements were found in Lokanthali (along 6 lanes highway) and Sinamangal, near international airport. Though most of bridges in Kathmandu are old and have been suffering from lack of maintenance, no single bridge was found suffered severe damage. Sand blows caused by liquefaction were noted within the Kathmandu basin. In contrast to geotechnical failure in Kathmandu, structural failure was severe. Thousands of buildings were collapsed or suffered from severe damages (Sharma et al., 2016).

However, the shallow depth of the earthquake has contributed to numerous landslides, subsidence, ground fissures and the high losses in village of the affected district. Both geotechnical and structural failure in the Kathmandu Valley were caused not only by the poor quality of construction and/or poor soil condition but also by local site effects induced by soft alluvial soil deposits, basin effect and basin edge effect (Sharma and Deng, 2016).

VI. FORECASTED LIQUEFACTION SCENARIO

Many studies have been done on liquefaction potential analysis in the Kathmandu Valley. A liquefaction hazard map of Kathmandu Valley (UNDP/MOHPP 1994) was prepared using semi-empirical approach proposed by Juang and Elton (1991) with 123 boreholes which indicates most part of the Valley seems to be in high liquefaction potential zone (Fig. 6). Liquefaction susceptibility was judged “high” and “medium” in a large area along the major rivers and channel. Similarly, JICA carried out a study on earthquake disaster mitigation in Kathmandu Valley in 2002 and prepared a liquefaction hazard map. In contrast to UNDO/MOHPP, JICA report (JICA, 2002) shows that most part of Kathmandu Valley is not vulnerable to risk of liquefaction (Fig. 7).

Piya (2004) studied the liquefaction potential in Kathmandu Valley and forecasted the liquefaction in many areas of Kathmandu Valley as shown in Fig. 8. Piya (2004) found that general characteristics of soils along with shallow ground water table in Kathmandu Valley are susceptible to liquefaction. His results pretty coincide with UNDP/MOHPP (1994) studies. Subedi et al. (2012) analysis more than 300 SPT and bore hole data to prepare the liquefaction hazard map of Kathmandu Valley. On the basis of his findings and past literatures he concluded that some parts of Kathmandu Valley pose a risk of liquefaction as shown in Fig.9.

Shrestha et al. (1999) prepared the geo-environmental map for the sustainable development of the Kathmandu Valley. In this study, liquefaction was also considered. Shrestha et al. (1999) also indicated high risk of liquefaction in Kathmandu Valley even due to moderate shaking. As that the airport area largely consists of sandy deposits, and the ground water table in the airport is at shallow depth, Dixit et al. (2013) concluded that there are greater possibilities of soil liquefaction in Tribhuvan international airport (TA) during an earthquake. Dixit et al (2013) emphasised potential damages on bridge in Kathmandu Valley due to liquefaction. However, no single bridge has been reported damaged due to liquefaction. Similarly, Gautam and Chamlagain (2015a) highlighted Runway and taxiways are particularly vulnerability of Runway and taxiways due to liquefaction during earthquake. They recommended for ground improvement to decrease the liquefaction potential in airport during earthquakes. Mugnier et al. (2011) reported many heritage/monuments (e.g. Pashupatinath temple, Hanuman Dhoka, Patan and Bhaktapur and the stupa of Boudanath) are located at the sites have moderate to high susceptibility to liquefaction.

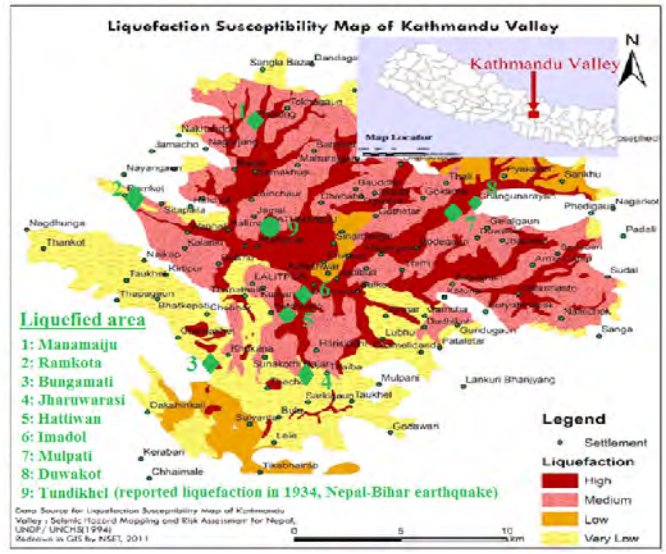

Fig. 6 Liquefaction susceptibility map of Kathmandu Valley (UNDP/MOHPP, 1994) together with reported liquefied locations during 2015 Gorkha earthquake

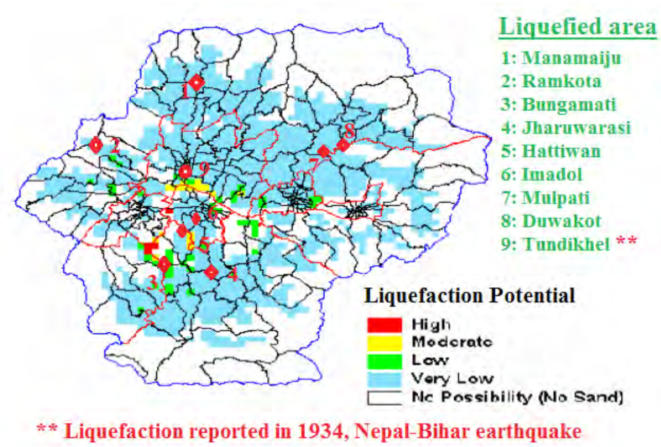

Fig. 7 Liquefaction potential map of Kathmandu Valley together (JICA, 2002) with reported liquefied locations during 2015 Gorkha earthquake

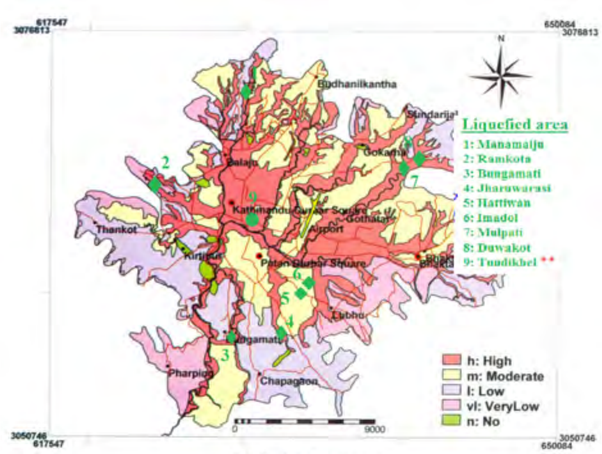

Fig. 8 Liquefaction potential map of Kathmandu Valley together (Piya, 2004) with reported liquefied locations during 2015 Gorkha earthquake

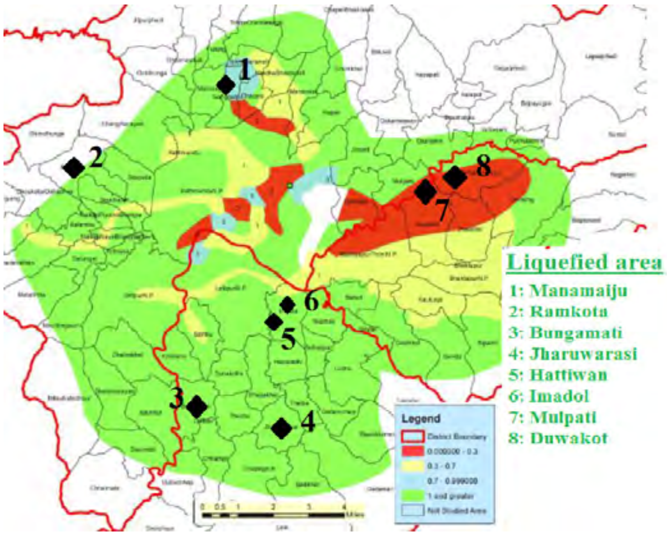

Fig. 9 Liquefaction potential map of Kathmandu Valley together (Subedi et al., 2012) with reported liquefied locations during 2015 Gorkha earthquake

VII. LIQUEFACTION DURING 2015 GORKHA EARTHQUAKE

The deposition in the Kathmandu Valley is lacustrine and fluvial in origin with thickness up to 500 m (Sakai, 2001). The deposited sediments are made up of clay, silt, sand and gravel. Areas with loose sand deposits have a greater chance of liquefaction after the earthquake. UNDP/MOHPP (1994), Piya (2004), Shrestha et al (1999), Dixit (2015), Mugnier et al (2011) and many other researchers concluded that a large area in Kathmandu Valley is susceptible to liquefaction in this region. However, unlike in previous major earthquakes, the liquefaction triggered by the 2015 Gorkha earthquake appears to be fairly limited and localized. The localised areas where liquefaction was observed are Manamaiju, Ramkota, Bungmati, Jharuwarasi, Hattiwan, Imadol, Mulpani and Duwakot (Figs. 6-9). Typically, these are sand boils formed by freshly ejected sand forced out of over-pressurized sub-strata. At most site, sand was ejected to agricultural fields forming deposits that varied from thin veneers to sheets, a few centimeters thick. Liquefaction-induced damage to structures in these areas was not found except buildings on some places tilted slightly.

All these eight locations were indicated as the moderate to high liquefaction susceptibility zone by both UNDP/MOHPP (1994) and Piya (2004) with the exception of Jharuwarasi area as shown in Figs. 6 and 8. In contrast to UNDP/MOHPP (1994) and Piya (2004), JICA (2002) and Subedi et al (2012) identified most of these locations as a non-liquefiable area even though the estimated ground motions in their research is higher than the observed ground motion in Kathmandu Valley during the 2015 Gorkha earthquake. JICA (2002) and Subedi et al (2012) considered magnitude of earthquake as 8.0 and the corresponding peak ground acceleration as 0.3g as. While the observed peak ground acceleration in Kathmandu Valley during 2015 Gorkha earthquake was about 0.18 g. Brief descriptions of reported liquefied locations are presented here. Some studies have shown that Kathmandu bears non homogenous soil strata even within small area (e.g. Neupane and Suzuki, 2001) which suggest that liquefaction potential of small area may differ. Past studies have limited information on site specific liquefaction potential. Most of these researches were based on SPT value and bore hole data. Extensive dynamic and/or cyclic field test and/or laboratory test are very important to predict the liquefaction potential accurately. Site specific liquefaction potential of even small area of the Kathmandu Valley is necessary in order to implement the reconstruction planning properly and also to prevent and minimize the probable damage that might occur during large earthquake in the valley.

The Manamaiju area is located on the north-west edge of Kathmandu Valley (Fig. 6-9). Some sand boils and traces were found in paddy fields on the right bank of the Bishnumati River (Fig. 10a). Other geotechnical problems were observed in the surrounding area but were not associated with liquefaction. Structural damage was significant in Manamaiju but not associated with liquefaction. Ramkot is located on the western edge of the Kathmandu Valley (Fig. 6-9). Liquefaction of very fine sand was observed on slope (Fig. 10a and b). Both Geotechnical and structural failures were observed in Ramkot but not associated with liquefaction.

Extensive liquefaction was found in the flood area of the Bagmati River in Bungamati, located on the south edge of Kathmandu Valley (Fig. 6-9). The ground water table was at shallow depth (□1.5 m) as the flood plain is 200 m far from Bagmati River. Sand boiling and fissures were seen on a flat plain along the river channel. Jharuwarashi is located on the southeast edge of Kathmandu Valley (Fig. 6-9). Ground fissuring of about 100 m long and 10 cm wide, parallel to the Karmanasa River, was found (Okamura et al., 2015). Sand boils were ejected through the fissures.

Numerous sand boils were found in Hattiban, Lalitpur (Fig. 11a). Unlike other liquefaction sites that this site is not located on the river bank and not a borrow pit for sand. Some sand boils and traces were also found in the field of Nepal Agriculture Research Council, Khumaltar. Small scale liquefaction is found in Imadol area, southern part of Kathmandu (Fig. 6-9) as shown in Fig. 11b. The white spots in the field are the trace of sand boiling. There is no any significant damage in this surrounding area except tilting of a house.

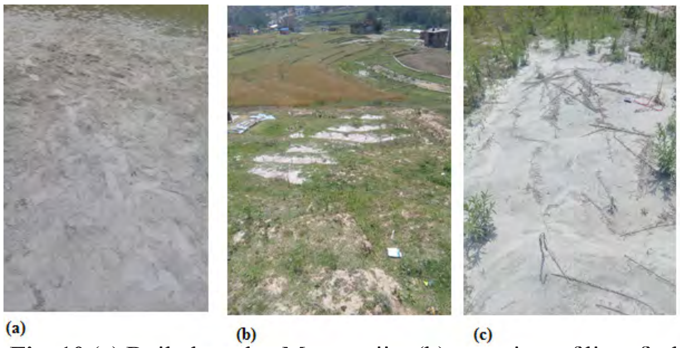

Fig. 10 (a) Boiled sand at Manamaiju, (b) overview of liquefied area at Ramkot, and (c) boiled sand at Ramkot

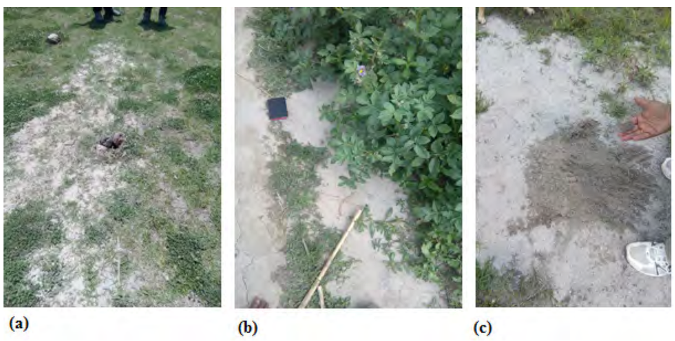

Fig. 11 (a) Boiled sand at Hattiwan, (b) boiled sand at Imadole, and (c) boiled sand at Duwakot

Duwakot is located on the east edge of Kathmandu Valley (Fig. 6-9). This is another site with clear liquefaction as indicated by sand boils. It is also famous for sand mining. Extensive liquefaction was found in Duwakot near Nepal Engineering College on the left bank of Manohara River (11c). Sand boiling and fissures were seen on a flat plain along the river channel. As liquefied site is about few hundreds meter from Manohar river, the water table at shallow depth may lead to liquefaction during the earthquake. No significant damage due to liquefaction was found. However college building was reported subsided slightly (Okamura et al., 2015). Small scale liquefaction was also reported in Mulpani which is located on the same flood plain as Duwakot (Moss et al., 2015). Case studies of liquefaction have also been reported at other sites by Chiaro et al. (2015), Goda et al. (2015); Okamura et al. (2016) and Sharma and Deng (2016).

Wide spread liquefaction was reported in the Kathmandu Valley during the 1934 Nepal-Bihar earthquake. Severe fissures and boiled sand were found in Tundhikhel area (Rana, 1935) as shown in Fig. 12. Though both 1934 and 2015 earthquake occurred in the dry season, most of open fields and roads in Tundhikhel area were flooded by boiled sand during 1934 Nepal-Bihar earthquake. No evidence of liquefaction caused by the 2015 Gorkha earthquake was detected in this area. This might be attributed to the vibration pattern of the Kathmandu valley deposits seems to have been very different from that during the 1934-scenario earthquake. It is surprising to most researchers to note that the liquefaction has been incomparably less than forecasted.

Fig. 12 Liquefaction in Tundhikhel area, Kathmandu by the 1934 earthquake (after Rana, 1935)

The low liquefaction occurrence at the valley may be attributed to low amplitude of high-frequency shaking of the main shock. Peak ground motion observed in Kathmandu Valley (about 0.17g) was lower than the estimated peak ground motion (about 0.3g) (UNDP/MOHPP, 1994 and Piya, 2004) for these studies. Additionally, water levels were likely at their lowest levels because of dry season at the time of earthquake and/or rapidly sinking water table as a result of uncontrolled ground water withdrawal in Kathmandu Valley may be decreased the liquefaction potential. The lacustrine sediment might be insusceptible to liquefaction because of the fine grain-size distribution (Moss et al., 2015).

Very interestingly, most of liquefied sites are either within or on the edges (Figs. 6-9) of the Kathmandu Valley, regions likely to be more vulnerable to liquefaction due to basin edge effect. The basin edge effect can be defined as the amplification of seismic energy at the margins of sedimentary deposits. When such sediments are laterally confined by a more rigid basement rock as in the Kathmandu Valley, the seismic behaviour becomes multi-dimensional and lead to severe damages (Iyisan and Khanbabazade, 2013).

VIII. CONCLUSION

The paper aims to investigate liquefaction phenomena in Kathmandu Valley induced by the 2015 Gorkha earthquake. The field reconnaissance was carried out immediately after the earthquake. This paper briefly explained the tectonic and geologic setting of Nepal and recorded ground motions from the earthquake events, reviewed the past studies on liquefaction potential in Kathmandu valley, presented the case studies of liquefaction by 2015 Gorkha earthquake and compared with past studies. The following conclusions could be reached.

Liquefaction triggered by the M_w 7.8 Gorkha earthquake seems to be fairly limited and localized. This may be attributed to low amplitude motions observed in Kathmandu Valley and low ground water table at time of earthquake.

At most site, sand was erupted to agricultural fields forming deposits up to few centimeters thick. Liquefaction-induced damage to structures nearby liquefied area was not found except buildings on some places tilted slightly.

Most of identified liquefied locations were indicated as the moderate to high liquefaction susceptibility zone by some previous studies.

Most of liquefied sites are either within or on the edges of the Valley, regions likely to be more vulnerable to liquefaction and other damages.

Since the damage to buildings and other infrastructure in the Kathmandu Valley is linked with local ground conditions. Comprehensive geotechnical investigations should be carefully planned and executed in order to accurately characterize seismic response of soft sedimentary deposits and liquefiable soil deposits (seismic microzonation), and take it into consideration in the reconstruction works and development plan.

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Pakistan's Experience with Post-Earthquake Reconstruction and Rehabilitation

Muhammad Masood RAFI
Professor, Department of Earthquake Engineering, NED University of Engineering and Technology, Karachi-75270, Pakistan. Email: rafi-m@neduet.edu.pk

Sohail BASHIR
Vice Chairman (Civil), Institution of Engineers, Pakistan, Karachi Centre, Karachi, Pakistan

Sarosh Hashmat LODI
Dean, Faculty of Civil Engineering and Architecture, NED University of Engineering and Technology, Karachi-75270, Pakistan

Aziz JAMALI
Former Project Director, Housing Reconstruction in Awaran (HRA) Project, Awaran, Balochistan

ABSTRACT

Pakistan lies in a seismically active region of the world. Recent earthquakes caused damages in different parts of the country. This paper presents the details of post-earthquake rehabilitation and reconstruction strategies and challenges in areas affected by the 2005 Kashmir and 2013 Awaran earthquakes in Pakistan. In both these cases, owner driven reconstruction approach was followed for private housing which became successful in terms of safe and seismic resistant construction and timely completion. Capacity building trainings of large number of people were carried out to increase the resilience of communities against natural hazards. Participation of multiple agencies in the rehabilitation of infrastructure facilities caused some delays due to the shortcomings in the decision making process and larger coordination demands.

Keywords: *earthquake, rehabilitation, reconstruction, damage, cob material; confined masonry.*

I. INTRODUCTION

Pakistan is blessed with four seasons, diverse topographical features and varied climate in different parts. These elements, in some cases, are also responsible for different natural hazards the Country faces from time to time. Further, similar to other developing countries, Pakistan experiences the problems related to varied population density, unplanned development in disaster prone areas, vulnerability of population segments and poverty. These factors have compounded with the lack of disaster awareness and emergency planning to increase the risk of property and life loss across Pakistan. As a result, many natural hazards turned into disasters in the recent past and created unprecedented impacts on human settlements in different parts of the Country.

Human response to natural hazards has been a subject of intense investigation and study. These hazards have proved to be the most difficult enemy of mankind as they are able to cause destruction on a large scale close to human settlements. The events of natural disasters may be identified by excessive magnitude, frequency or duration (Arey and Bauman 1971, Bolt et al. 1975). The study of human history indicates that the ability of natural hazards to cause destruction is partly due to lack of awareness of human beings to mitigate the effects of these hazards. Different natural hazards include hurricanes, floods, tornados, typhoons, famine, fires, landslides and earthquakes. Of all these hazards, earthquake is considered as the most disastrous natural hazard with ability to cause devastation in terms of high number of human loss, and wide spread building and infrastructure failures and sufferings.

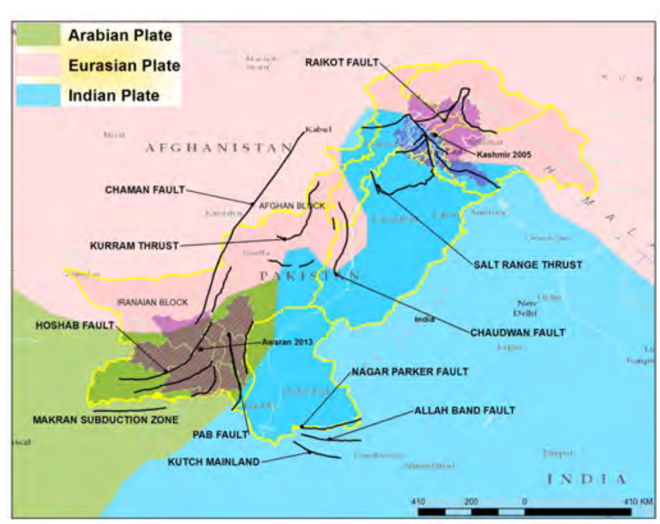

Fig. 1 Seismo-tectonic settings of Pakistan

Pakistan lies in a seismically active region. The north and western sections of Pakistan (which are located along the boundary of the Indian plate, and the Iranian and Afghan micro-plates) have been the centre of earthquake activities (Fig. 1). From Kalat (in the northern Makran range), Chaman Fault runs all along Pakistan's western frontier with Afghanistan; it passes by Quetta and enters Kabul in Afghanistan (DRIP 2014). An active fault also runs along the Makran coast where an active subduction zone exists off the coast (DRIP 2014). This is no surprise that the coast of Pakistan has been affected by tsunamis in the past.

The seismic threat for different parts of Pakistan has been demonstrated by a number of recent earthquakes. An earthquake, being a natural hazard, cannot be controlled; however, the risk associated with the building damages during an earthquake can be mitigated. Disaster preparedness and mitigation prevent a hazard from turning into disaster which can reduce the efforts for reconstruction and rehabilitation (R&R) of the affectees after an earthquake. The need of R&R also indicates the level of resilience of a society; lesser the need higher the resilience and vice versa. This paper presents two case studies related to R&R in the aftermath of earthquakes in Pakistan. The lessons learnt from these experiences are based both on the available literature and the authors' personal observations during their involvement with the R&R activities on the sites of both seismic events.

II. CASE STUDIES

2005 Kashmir Earthquake

This earthquake occurred on 8 October 2005 in the northern parts of Pakistan. The magnitude of the earthquake was recorded as 7.6 on Richter scale. The districts which were affected by this earthquake are highlighted in Fig. 1. Based on the destructions caused, this can be termed as the most devastating earthquake in the recent history of Pakistan. More than 73,000 people were killed and, at least 69,000 people were injured (ERRA 2006) due to the collapse of buildings during this earthquake. In addition, about 2.8 million people became homeless owing to the damage of nearly 450,000 buildings (Rossetto and Peiris 2009). Table 1 summarises the damages in the affected districts.

Table 1. Statistics of damages and reconstruction of houses in affected districts (ERRA 2006)

District	Destroyed		Moderate Damage		Slight Damage		Reconstruction	
	Number	Percent	Number	Percent	Number	Percent	Planned	Completed
Muzaffarabad	121,995	89	12,499	9	2,891	2	131,932	123,062
Bagh	79,514	96	2,716	3	627	1	84,509	76,838
Poonch	39,190	83	7,209	15	1,084	2	39,326	36,689
Shangla	141,141	54	8,514	33	3,277	13	15,151	14,398
Mansehra	106,523	70	32,702	22	11,933	8	106,653	65,323
Kohistan	6,323	46	4,850	35	2,646	19	11,976	10,088
Abbottabad	19,704	31	17,982	28	22,585	35	21,073	19,673
Battagram	49,345	85	7,035	12	1,777	3	52,783	39,396
Total	436,735		93,507		46,820		463,403	415,467

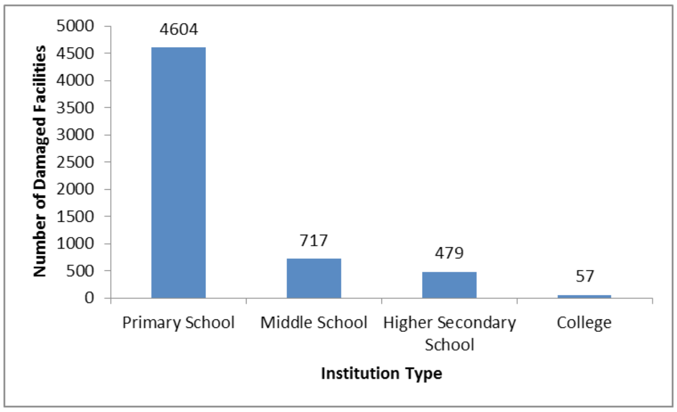

Fig. 2 Damaged educational facilities in earthquake affected areas (ERRA 2006)

Apart from houses, the earthquake also made significant damages to the buildings of educational centres. A total of 6298 centres were destroyed or damaged which comes out to be 67 percent of the total institutional facilities in these regions (ERRA 2006). The collapse of educational buildings caused deaths of nearly 18,000 students. Fig. 2 illustrates a distribution of damaged educational facilities. The total estimated cost of damages of these facilities came out to be US$ 54525 million (ADB/WB 2005). Heavy damages were also caused to infrastructure facilities such as water and sanitation, power supply infrastructure, roads and communication infrastructure. Considering the scale of destruction and devastation, the Kashmir earthquake was termed as Qa'yamat (Halvorson and Hamilton 2010) which is an Arabic phrase for the 'Day of Judgment'.

Rehabilitation and reconstruction activities

This earthquake happened at a time when there was little (if any) awareness about the consequences of earthquakes in Pakistani. As a result, the communities were completely unprepared and the capacity to deal with the aftermath of such tragedy did not exist. In view of the aforementioned scale of devastations, R&R was a paramount challenge. According to the initial estimates, housing reconstruction required 44 percent of the total reconstruction cost (ADB/WB 2005). After mobilizing in the affected areas, relief agencies of Government of Pakistan, and national and international organisations started their efforts for establishing temporary shelters, providing care for victims, rebuilding damaged structures and infrastructure, and resettlement of affectees (Halvorson and Hamilton, 2010).

Earthquake Reconstruction and Rehabilitation Authority (ERRA) was established by Government of Pakistan in March 2006. It was given the mandate of planning and coordination of R&R activities in the affected areas. The local governments of these areas were made responsible for micro level planning and implementation of R&R activities in coordination with ERRA. The policy of owner driven construction with the slogan of 'build back better' was adopted by ERRA. The building design proposed in the earthquake affected areas was based on light-weight construction using wood and corrugated galvanized iron sheets. A number of international agencies and organizations also provided architectural styles and seismic resistant design for houses (Halvorson and Hamilton 2010). ERRA mobilised nearly 600 teams in the field to provide training and technical advice in seismic resistant construction to the people and to monitor progress of reconstruction work. These teams provided training to 84,000 home owners (ERRA 2006). Financial grants were provided only to those affectees who followed safe construction guidelines issued by ERRA for the construction of houses. ERRA arranged master plans to rebuild the affected towns scientifically. A summary of the planned and reconstructed houses until October 2008 is given in Table 1. It is noted in Table 1 that nearly 90 percent of the planned houses were completed.

The damaged educational facilities were reconstructed with the help of local governments, sponsors and donors. Since a significant number of people were made disabled by the earthquake (Rafi et al. 2010), the rehabilitation programme included such designs of hospitals which should provide better access for the disabled. The medical staff members were provided special trainings related to the type of injuries inflicted by the earthquake.

Issues and challenges

Apart from private housing, the rehabilitation programme also included drinking water supply systems, sanitation systems, restoration of power sector, reconstruction of roads and bridges, and provision of modernised telecommunication infrastructure. Unfortunately, many of these were not completed on time due to multiple reasons. For example, reconstruction of public buildings especially education and health facilities took longer time and efforts than planned initially. The ERRA's approach of futuristic and grand building construction (both in quality and size) may be partly blamed for this delay which made it difficult to achieve the targets within the available resources. Nevertheless, rehabilitation of water supply schemes succeeded next after housing owing to its close connection with livelihoods.

Halvorson and Hamilton (2010) identified that the issues related to housing reconstruction included higher cost of lightweight construction material such as wood and CGI sheets, and difficulty faced by people in accessing the information about specific construction methods or available construction material.

2013 Awaran Earthquake

The Awaran, Balochistan earthquake occurred on 24 September 2013 (Fig. 1). According to United Sates Geological Survey, the magnitude of earthquake was 7.7 on the Richter scale whereas its focal depth was 20 km. The epicentre of earthquake was 66 km north-northeast of Awaran. The districts Awaran and Kech were badly damaged by this earthquake where significant life and property losses were reported. The most affected areas in these districts included tehsils Gashkor, Mashkai and Awaran of District Awaran and tehsil Dandar of District Kech. A summary of the damages and losses due to this earthquake is given in Table 2. In addition, 43 water supply schemes and 25 water channels were also damaged by the earthquake in the affected districts. Fig. 3 illustrates view of building damages in Awaran.

Rehabilitation and reconstruction activities

In the aftermath of this earthquake, the provincial government of Balochistan along with its

Table 2. Damage and losses due to Awaran earthquake (NDMA 2013)

Area	Casualties		Affected Villages	Damaged Houses		Damaged Schools	Damaged Health Facilities
	Death	Injury		Partial	Comp-lete		
Mushkay	179	158	45	12961	6151	33	6
Mangoli			20	411	95	14	1
Awaran	174	255	90	20129	12422	53	13
Kech	46	186	83	10236	6844	47	1
Total	399	599	267	32638	14118	147	21

Fig. 3 Damaged buildings in Gajjar

line departments started an R&R programme for the victims. It was termed as Housing Reconstruction in Awaran (HRA) Project. A project director was appointed for HRA to execute and monitor the reconstruction activities. In view of the present bitter experience of damages and losses by the deficient construction, the Government of Balochistan decided to promote seismic resistant construction to avoid similar incidents of damages and losses due to the future earthquakes. The authors were engaged by the Government of Balochistan to assist in the designing of earthquake resistant houses for the people in Awaran.

Similar to several other parts of Balochistan, earthen materials such as adobe and cob were the predominant material types for private housing in the affected region. These materials offer several advantages such as low construction cost without modern technology and skilled manpower, and better thermal insulation. Although these buildings are considered weak in their resistance to earthquake forces worldwide research activities have indicated that their seismic resistance can be increased keeping the fabric of the buildings intact.

Fig. 4 Model of earthen construction with earthquake resistant features

Therefore, the design of a seismic resistant earthen building was carried out by the authors which was tested using a shaking table (Fig. 4). This type of testing is one of the highly reliable and most sophisticated methods for the evaluation of seismic performance of different structures. Based on the satisfactory performance of the model during the shaking table test, the design was implemented in the construction of houses in the affected areas.

HRA Project followed owner driven reconstruction approach whereby people were enabled to build their houses themselves by providing them the necessary training and funds. The reconstruction activities started in July 2014. A number of model houses were constructed by HRA in different places throughout the affected areas to educate home owners (Fig. 5). Brochures and visuals were also used to disseminate the necessary information on the construction of safe houses. The authors presented lectures and workshops on safe housing construction. Close monitoring of construction activities was carried out by HRA with the help of field support staff members to ensure that the guidelines are followed during construction. As a result of these efforts, construction of 6000 houses was completed in December 2015 (18 months from the start of HRA project). In addition, Communication and Works (C&W) Department of the Government of Balochistan started construction of 65 schools and 7 health facilities in different areas in January 2014 and was able to complete construction of 25 schools and 5 health units by December 2015.

In addition to earthen buildings, design of confined masonry (CM) houses comprising of one and two rooms was also carried out by the authors on the request of HRA. For the execution of this design, capacity building of masons was carried out in NED University by providing them on-site training of different construction activities. Fig. 5 illustrates a typical CM model house constructed by HRA in Awaran.

Fig. 5 Views of seismic resistant model houses in Awaran: (a) cob house; (b) CM house

Issues and challenges

The issues faced by the HRA project management were related to the demands of larger room sizes by the people due to cultural reasons compared to those proposed in the seismic resistant design of houses. Similarly, resistance from some sections of people was experienced towards the owner driven construction policy and the capacity building training sessions. Nevertheless, it was made clear by the project management that any deviation from the guidelines would result in stoppage of financial assistance. This proved to be the most effective deterrence to stop violations of the instructions and guidelines issued by the HRA project management.

As a result of courtesy mobilization efforts and masonry trainings, reconstruction of housing units surpassed the pace and quality of construction manifested by contractors/builders of education and health facilities. While in one year only 25 school buildings were constructed by C&W Department of the Government of Balochistan using contractors, HRA was able to complete the construction of 5000 houses. Based on this experience, community construction model has been suggested to the Government of Balochistan for rebuilding damaged school buildings.

III. CONCLUSIONS

This paper presents an overview of rehabilitation and reconstruction experience in Pakistan from two recent earthquakes. Important conclusions drawn from the study are listed as under

1. Owner driven construction provides an effective model for private housing reconstruction in disaster stricken areas. It can be linked with monitoring, disbursement of funds, and training and capacity building of home owners. This approach ensures transparency in the disbursement and utilisation of funds and facilitates timely completion of work.
2. The engagement of higher educational institutions is beneficial in providing efficient seismic resistant design and capacity building of builders and craft persons. The required level of knowledge and skills can be transferred to both the home owners and the builders.
3. Delays in the reconstruction of public infrastructure facilities could be resulted due to coordination issues between different agencies. Community based reconstruction of these facilities can provide an alternative, similar to the approach for private housing reconstruction.

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The Challenges of Housing Reconstruction after the April 2015 Gorkha, Nepal Earthquake

Jitendra K. BOTHARA

Technical Director-Seismic Engineering, Miyamoto International New Zealand Ltd., Christchurch, New Zealand

Rajesh P DHAKAL

Professor of Earthquake Engineering, University of Canterbury, Christchurch, New Zealand

Jason M. INGHAM

Professior, Civil & Environmental Engineering

University of Auckland, Auckland

Dimytro DIZHOR

Lecturer, Civil & Environmental Engineering

University of Auckland, Auckland

ABSTRACT

On 25th April 2015, mid-western Nepal was hit by a devastating earthquake measuring Mw 7.8 on magnitude scale with the epicentre located 76 km north-west of Kathmandu. The earthquake was followed by a series of aftershocks, with the most significant occurring on the 12th of May, 2015 with Mw 7.3 with the epicentre located north-east of Kathmandu. The earthquake and the associated aftershocks resulted in destruction of half a million buildings in addition to another quarter million buildings that were damaged; thereby leaving millions of people homeless and causing a loss of more than Rs350 billion (≈US$3.5billion). Approximately 9,000 people were killed and over 23,000 people were injured. Thirty-one among the country's 75 districts were affected, of which 14 districts were declared "crisis-hit". The hard-hit area was spread over more than 30,000 square kilometres of hills and mountains. A large part of the earthquake-affected area is difficult to access and highly snow-prone, with rugged terrain and scattered settlements. The earthquake also triggered numerous landslides, causing the destruction or evacuation of several settlements. It posed unique challenges to an under-resourced country like Nepal, and the post-earthquake recovery and reconstruction is bound to require efforts and resources on a massive-scale. For residential buildings, the Government of Nepal adopted a house-owner driven approach. The recently established post-earthquake reconstruction policy states that the government would provide equal technical assistance and subsidy to each family, without differentiating between who lost what. This paper presents the authors' first-hand experience in the recovery and reconstruction efforts, unique challenges faced and the opportunities perceived.

Keywords: *Gorkha earthquake, reconstruction, recovery, resources, capacity building*

I. Introduction

An earthquake of magnitude Mw 7.8 on the Richter scale struck mid-western Nepal (Figure 1) on 25th April 2015. The earthquake was followed by hundreds of aftershocks, the most significant one (M_w 7.3 with the epicentre located north-east of Kathmandu) occurring on early afternoon of 12th May 2015. The earthquake affected an area of approximately 30,000 square kilometres across fourteen districts. It killed around 9,000 people and injured another 23,000. The earthquake destroyed around half a million houses and damaged another quarter million houses. Around 7000 schools were either completely or significantly damaged (NPC, 2015). It was lucky that the 25th April earthquake struck on afternoon of Saturday, Nepali weekend and the 12th May aftershock struck during school holidays after the earthquake. The total direct economic loss due to the earthquake is estimated to be US$7billion. By far the private housing sector suffered the most devastating blow, followed by Tourism and Environment and Forestry sectors (NPC, 2015). Out of total loss of Rs700 billion, housing and human settlement sector alone lost Rs350 billion. The fatalities, injuries and extensive damage and devastation made it by far the most destructive earthquake ever to have occurred in Nepal.

Poor performance of the buildings is to take a major share of the blame for the catastrophe. Buildings performed poorly because they were made of very weak construction materials such as dry stone, stone in mud mortar; or were constructed very poorly despite being of good materials such as fired brick or stone in cement mortar, confined masonry, and reinforced concrete (RC). A study conducted by Manadhar (2015) showed that in the Kathmandu valley around 55% of the new house owners did not consult engineers for construction of their buildings. Nonetheless, considering rural nature of the majority of the earthquake-affected areas, it is expected that probably more than 95% of buildings in the area were non-engineered, and almost all buildings were without any aseismic features. The authors believe that the major problem was the lack of preparedness despite knowing that the region is earthquake-prone. (Nepal has experienced a number of moderate to large earthquakes in the past), and despite recent studies also showing the potential of major earthquakes in the region (Bilham, 2001). It led to a lack of implementation of building standards at a larger scale, knowledge dissemination on earthquake-resistant construction and awareness for the need for it. Some improvements have been made in last two decades; however mostly in the urban areas.

The recovery and rehabilitation of earthquake affected areas in Nepal have posed unique challenges. Limited affordability and accessibility to modern construction materials, information, skills and technology are some of the impending factors. Providing technical support for Build Back Better (BBB) to the earthquake victims, many of whom live in remote areas, poses severe logistical challenges. Scarcity of trained manpower for reconstruction is another major issue. Other challenges include the high reliance on agriculture and low cash flow in the area, and a large proportion of male population who have migrated overseas for jobs, resulting in families that are led by women who are already overburdened by daily chores and agricultural work.

Figure 1: The earthquake-affected areas in Nepal (as of 21 May 2015) (Source: Ministry of Home Affairs, Government of Nepal)

II. Building MateriALs and Typology

The common construction materials in the earthquake affected areas are stone, fired or unfired brick, mud, steel and concrete, and timber. Overall, stone is the most common building material because of its local availability and affordability. The most common binding material for masonry construction is mud, which is revealed by the unproportionate number of damaged or destroyed low strength masonry buildings (see Table 1) due to the earthquakes. The selection of building materials is largely controlled by local availability, affordability, the local economy, and local climatic conditions. Many of the earthquake-affected areas are prone to heavy snowfalls that require thick walls and heavy insulation to keep occupants protected from the extreme weather.

Table 1: Breakdown of damaged or destroyed houses (private houses only)

Details	House typology	Number of houses
Collapsed/ destroyed	Low strength masonry (stone/ fired brick in mud, adobe, mud construction, etc)	474000
	Cement based masonry (stone/ fired brick/ cement block in cement mortar)	18,000
	RC frame (with masonry infill)	6600
Damaged	Low strength masonry (stone/ fired brick in mud, adobe, mud construction, etc)	174000
	Cement based masonry (stone/ fired brick/ cement block in cement mortar)	66,000
	RC frame (with masonry infill)	17,000

(Adopted from NPC, 2015. All the figures are rounded)

Construction materials and skilled masons are extremely scarce in the area, particularly with respect to the modern materials. Other than large scale construction, hand mixing is the most common method of concrete preparation, and concrete vibrators are rarely used for compaction - resulting in low-grade, honeycombed concrete. Curing of concrete is also not very well practised as an integral part of the concreting process.

Further to the above, Nepal in general going through rapid change in building types particularly because of economic liberalisation, cash inflow in the country from remittance from Nepalese working overseas, increased accessibility to construction materials and technology due to increased road network. The motive to go vertical expansion of the building due to increased land cost, rapid urbanisation, etc has further helped the process. The relatively better performance of RC frame buildings compared to low strength masonry buildings during the earthquakes, despite these being structurally deficient, have further reinforced the notion that RC frame buildings are the only building types which are safer for earthquake and somehow owning a RC frame building has become part of social respect. All these factors have made other building types obsolete where the potential building-owners can access and afford RC frame buildings.

2.1 Stone masonry buildings

Stone masonry (and its variations) is the most common building type in the earthquake-affected area, particularly in rural areas which is shown by the unproportionate number of this type of buildings suffering damage and destruction. The walls of these buildings are constructed of boulders, rubble stone, or dressed/ semi-dressed stones in mud mortar or dry mortar (no mortar). Floors of these buildings are mostly constructed of timber overlain by mud or RC slab and the roof is usually made of light corrugated iron sheet or tiles or slate on a timber frame (Figure 2) or even RC slabs. However, these classifications blur with time, space, affordability and accessibility. Commonly, these buildings are up to two storey plus an attic. In higher Himalayas, cladding the walls from inside with timber is also common. These buildings suffer from deficient strength, a lack of integrity between orthogonal walls, and between walls and floor/roof of the building.

If cement mortar is used for stone masonry wall construction, the floors and roof are constructed of RC slabs (Figure 2). These buildings are more common for government buildings or in urban areas or along transport corridors for private buildings.

2.2 Unreinforced mud or adobe buildings

Unreinforced mud or adobe buildings for wall construction (Figure 3) are more common in the Kathmandu valley and in valleys where stone are not available in abundance. It is a common practice in the valley to clad adobe buildings with burnt clay bricks to protect them from weather and aesthetic reasons. However, the cladding veneers are not tied to the main structural walls. These buildings are usually two to four storeys high. The foundations of these buildings are mostly constructed of stone masonry or fired clay bricks, and the floors are usually constructed of timber overlain by mud and roof of jhingati (small sized clay tiles on mud). These buildings are mostly old and severely lack maintenance. Similar to the stone buildings, these buildings too suffer from deficient strength, a lack of integrity between walls, and also between walls and floor/ roof of the building,

2.3 Unreinforced brick masonry buildings

a) A Stone masonry building in cement mortar with RC floor and roof slab

b) A stone house under remediation in the higher Himalayan region (source: Subash)

Figure 2: Typical stone masonry buildings.

a) An adobe building with timber floor

b) An adobe building under partial demolition

Figure 3: Typical unreinforced masonry buildings in adobe.

Bricks are mostly common in the Kathmandu valley or along the transport corridors, and in many other places these have to be transported from the distant plains or the Kathmandu valley. The walls of these buildings are constructed of brick in mud or cement-sand mortar (Figure 4). The walls are usually 230 to 460 mm thick. However, the palaces, monumental buildings have thicker walls. The roofs and floors are usually constructed of timber or cast-in-situ RC slabs. These buildings are also mostly two to four storeys high. Many of these buildings, particularly constructed of mud mortar are old and severely lack maintenance. These buildings suffer from deficient strength, a lack of integrity between walls, and between walls and floor/roof of the building,

2.4 RC frame construction

a) A brick in mud mortar with timber floor and timber roof structure, Kathmandu valley

b) A brick in mud mortar with RC slab, Kathmandu valley

Figure 4: Typical brick masonry buildings.

Residential buildings constructed of RC frame vary widely in terms of height; most commonly between 2 and 6 storeys (figure 5). However, there are several RC frame apartment buildings up to 20 storeys high (a few even higher). The roofs and floors of these buildings are typically cast-in-situ RC slabs. Invariably, the cladding and partition walls are constructed of concrete block, brick or stone masonry, although brick in cement mortar is the most common mode of construction. The partition walls are usually a half brick (110 mm) thick, and are neither tied into the frame nor reinforced. These frames suffer from poor configuration, materials and workmanship, and a lack of ductile detailing.

2.5 Mixed building

A typical RC frame building on steep terrain

A typical multi-storey RC frame building with masonry infill

Figure 5: A typical RC frame building with concrete block infill wall.

Mixing of construction materials and structural systems (Figure 6) is not uncommon in the earthquake-affected areas. In many buildings, lower storeys were constructed in mud mortar and upper storeys were made of cement mortar.

a) Mixed material and structural systems (RC frame, timber and stone in bottom storey and timber in upper storey (source: Subash)

b) Stone and fired brick walls

c) Mixed walling materials (adobe and fired brick)

d) Adobe building with fired brick cladding. Note roof constructed of timber and concrete slab

Figure 6: Buildings with mixed material and structural systems.

III. Diagnosis of disaster

3.1 Performance of buildings in the earthquake

All types of buildings discussed above, irrespective of construction materials or systems, performed very poorly, although performance of modern RC frame buildings was relatively better than unreinforced masonry building. Engineered RC buildings performed much better than their non-engineered counterparts because of better materials, better detailing and the some engineering input. However, many of these performed at a far less than satisfactory level. Stone and adobe buildings suffered delamination of wall wythes, toppling, shear failure of walls and disintegration of floor and roof (Refer Figure 7). Most of these buildings just turned into a heap of materials because of a lack of integrity between structural components.

Similarly, brick and block masonry buildings mostly suffered out-of-plane failures and shear failure of walls. Many nonengineered RC frame buildings suffered soft-storey failure, and a few suffered overturning failure, shear failure of columns and beams, lapping failure of reinforcement, anchorage failure of beam bars, opening of stirrups, cold joints and crushing of concrete leading the building to partial or total destruction. A few of these damage patterns are presented in Figures 2, 4, 5, 6 and 7.

a) Delamination of stone masonry infill wall in a RC frame building.

b) Brick in mud mortar, note a lack of positive connections between return walls.

c) Destroyed masonry buildings

d) Pancaked RC framed building

e) A damaged RC frame building (note lack of stirrups in the joint)

f) A destroyed RC frame building, note anchor failure of beam bars.

g) Disintegration of upper story.

h) Diagonal tension failure and toppling of partition walls in a multi-storey RC frame building.

Figure 7: Typical failure modes of RC frame buildings.

Observations of damage showed that many buildings suffered catastrophic destruction or damage because of very minor deficiencies which could have been easily avoided. The authors believe that, had there been even the smallest awareness of proper construction methods, a minor input in building construction practices, the magnitude of the disaster would have been far less than what was observed.

3.2 Building production mechanism

The building production mechanism in the area is highly informal and incremental in nature, and engineers have very little involvement in the design of private buildings other than mostly larger buildings in urban areas. Professional advice is rarely sought for most of the residential buildings (even in urban areas) and, if sought for residential houses, is mostly limited to the building permit process. More than 95% buildings in the earthquake affected area are estimated to be non-engineered, and even the engineered ones are not much better than the non-engineered ones in many cases (Bothara & Sharpe, 2003).

Design and construction of residential houses in the area is mostly managed by the owners themselves, employing a local skilled artisan to provide overall technical and organizational support. Note that most artisans don't have a formal training, although some efforts have been made in the last couple of decades to train artisans. The traditional artisans play a pivotal role in the overall construction activity, and the owner heavily relies on them for all types of advice. Furthermore, construction of these buildings is largely dictated by the local availability of construction materials and skills (Bothara et al, 2003). The owners procure the materials themselves, according to the quantities advised by the chief mason, and are therefore responsible for material quality selection.

Governmental buildings or large scale construction projects are developed and constructed more formally. However, many of these also clearly showed conceptual deficiencies and lacked quality control in their construction.

3.3 Nepal Building Standards

Until 1994, Nepal had not had any formal regulations or documents of its own setting out either requirements or good practice for achieving satisfactory seismic performance from buildings. Indian standards were commonly used for building design. The 1988 Udaypur earthquake in Nepal and the resulting death and damage to both housing and schools drew attention to the need for change and improvement in the building practices which resulted in development of building code for Nepal under UNCHS' funding support (NBCDP, 1994). The project also developed a Management plan to implement building standards and drafted the Engineering Council Act and Building Act for effective enforcement of the building code. However, due to various reasons the building code and the acts could not be promulgated until 2004, and even when promulgated, both the acts which were supposed to monitor and control building safety came in much softer form, and were unable to enforce any measurable change. Further to the above, the focus of the Building Code was urban areas; and that too was left to the discretion of the Territorial Authorities resulting in a lack of effective enforcement and compliance. There were efforts, supported by the Government of Nepal and international and national organizations, aimed at strengthening capacity on the supply side of earthquake building safety. However, still there is not enough demand from the potential homeowners that their houses should be earthquake resilient (Manadhar, 2015).

3.4 Site for building construction

The buildings along the ridge lines and on sloping grounds suffered significant damage during the Nepal earthquake and aftershocks. It could be because of three reasons: i) extremely high accelerations at sites located on topographic ridges, ii) foundation failure of the buildings located on the sloping ground, and iii) structural failure of the building structure due to the geometrical irregularity in the building caused by unequal heights by columns on the slope. However, in Nepal most of the market centres in the hills are located along the ridge lines. Usually, the houses are constructed on the land which is not suitable for agriculture, which is essential for food security.

Figure 8: Destruction of buildings on steeply sloping ground.

3.5 Major factors that contributed to the disaster

The building damage and/or destruction was the major contributor to human and property loss. The buildings in the earthquake affected areas were grossly inappropriate for a seismic region because of poor materials and the deficient construction processes as discussed above. It appears that majority of the human casualty from the earthquake were caused by the buildings themselves.

A few factors that contributed to this disaster are: lack of appreciation of seismic hazard, the building production mechanism, the lack of a techno-legal regime to address the safety issues, access to information, weak local economy, affordability and accessibility to better materials and technology; knowledge gap on earthquake resistant construction at all levels; lack of local availability of better materials or their improved use, quality control issues, poor maintenance, loss of traditional skills and ignorance or negligence on quality control in construction (Bothara, 2007).

IV. Reconstruction Challenges

- Inaccessibility: The earthquake area is spread over 30,000 square kilometres and is mostly in very rugged, high altitude, in some parts snow-falling terrain that is remote and with low accessibility. It is so spread out that, in some cases, it takes days to travel from one end of the affected district to the other. To reach many of the villages at high altitude, it takes many days from the nearest motorable road. Several areas are totally cut off during winter months by snow. The terrain makes transporting construction materials impossible or unaffordable in most areas. Although urban areas also suffered damage and destruction, rural areas were affected most substantially by this earthquake and urban areas luckily escaped the major blow (Figure 9). Although, exact numbers are not available, it is expected that over 90% of the buildings destroyed and damaged are rural houses which are, in many cases, very sparsely distributed (Figure 10). This makes it extremely difficult for any outside help to be delivered in many areas.

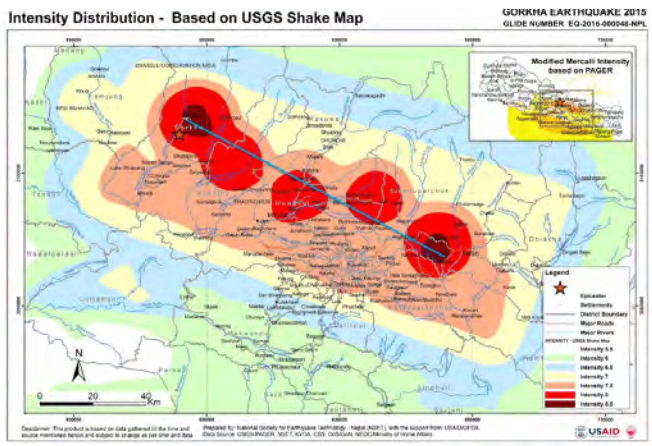

Figure 9: Intensity Distribution.

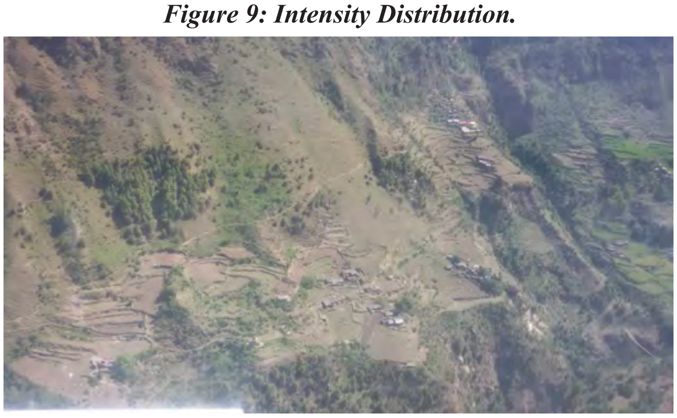

Figure 10: Typical sparsely populated and isolated area in the affected region.

- Infrastructure for reconstruction: Although, the impending earthquake risk in the area was well known, no effective infrastructure was developed to cope with the current level of disaster. Further to it, the earthquake caused wholesale and widespread impacts the existing local level governmental and non-governmental machinery. However, it was fortunate that the Kathmandu, capital of Nepal survived practically unscathed.. Due to the absence of democratically elected local government representatives (local elections have not been held in Nepal since 2002), government officials serve as the in-charge of local government offices. This adhoc solution has helped continuation of governance at local and district level. But it has affected the quality and availability of services at local level as well as accountability and responsiveness of government institutions to a significant degree (Bhattarai, 2012).

- Affordability: The preliminary assessment of the incident and impact suggests that the earthquakes have disproportionately affected the poorer, rural locations relative to the urban and less poor areas. Even in Kathmandu valley, where the per capita income is disproportionately higher than the rest of the country, households who were either poor or vulnerable have been particularly exposed (NPC, 2015). The Government of Nepal has offered to provide Rs200k to the owners of collapsed (or severely and irreparably damaged) houses. The question is how the gap between the funds required and that offered by the government will be bridged? A rough estimate has indicated that construction of even a 300sqft (which is much smaller than the average house size in Nepal) house constructed of stone in mud where stones are locally available would require at least Rs400k (including cost of earthquake resistant features). Remittance will certainly be helpful to bridge the gap, if a family member is working abroad and sending money home; but the question is how well the funds are spent to create a safer abode. A study conducted by Manadhar (2015) has shown that despite significant level of remittance money being spent on construction of buildings, these were of not any better quality if compared with the other buildings constructed of similar material.

- Loss of faith: After the April earthquake, people are still scared. Aftershocks have continued on a regular basis and are still striking. Some people moved away from their land due to landslides, destroyed buildings and other reasons. Most would not sleep in any of the remaining buildings, preferring outdoor tents even in the winter. Many people removed upper storeys of their houses. Their greatest fears were from stones and heavy roofs. As the earthquake had also destroyed many RC frame buildings, many people felt none of the building techniques they had seen before is safe. It was extremely difficult to convince them that it was not the earthquake, but the poorly constructed buildings that caused the disaster and that buildings can be made earthquake-resilient. The authors saw evidence of demolition of many damaged buildings because the house owner had lost faith in the building's strength, even though the building could have been easily repaired and strengthened. Crack-phobia was common after the earthquake, even among engineers and they were observed recommending deconstruction of buildings that had suffered nominal damage.

- Building materials and construction practices: The building construction mechanism is mostly vernacular, non-formal, incremental in nature, and dictated by the affordability of the owners and local availability of construction materials. The most common construction materials in the earthquake-hit areas are non-engineered and traditional such as stone, mud, brick, timber which are likely to continue in years to come. The engineering community does not have much knowledge about these construction types and methods. It is envisioned that the government's financial assistance package comes with a precondition that the houses will incorporate earthquake resistant features. Structural grade timber is very scarce in the majority of the area and transporting cement and reinforcement from outside is not possible, although Nepal produces enough cement and reinforcing steel within the country. Scarcity of quality sand for constructing RC structures is a major hindrance. Further to the above, rapid strength gain of cement is misinterpreted such that cement based construction does not need any quality assurance. It is commonly accepted that once a building is made of RC frame, any height is possible, although building may not have been designed for earthquake resilience. Within the affected area, building practices vary rapidly with changes in local availability of materials, culture, and economic status of the building owner. Thus, it is neither possible to strictly standardise everything, nor would it be appropriate to do so.

- Manpower shortage: It was predicted that the reconstruction effort would require some 20,000 skilled manpower that includes engineers, foremen, masons, and carpenters (NPC, 2015) – excluding semi-skilled/unskilled workers. Other issues is: when even the engineers do not have much knowledge in the traditional materials and construction techniques, how they will be able to disseminate appropriate knowledge to lower part of the pyramid and create a cascading effect for transfer of knowledge. Developing manpower on this scale is in itself a mammoth task. Note that Nepal's currents needs of skilled workers are met largely by Indian workforce, which is not enough is cope with the sudden increase in demand in the short term. Further to this, outward migration of youth population for long term employment has taken a phenomenal shape, which has resulted in a severe scarcity of working male population in various parts of the country (WB, 2011). "Everyone is leaving – Who will sow our land?", a study undertaken by Adhikari and Hobley in 2011 in Khotang district, a district classified as "hit with heavy losses", shows the plight of shortage of working male population and its impacts on daily life. The condition has worsened further with ever increasing rate of outward migration. The challenge is, what percent of the 16% of the migrants who have engaged in the construction industry in other countries (WB, 2011) could be brought back to Nepal to contribute in the reconstruction.

- Knowledge gap: The construction workforce in Nepal lacks knowledge on earthquake-resistant technology, in general, and traditional technologies and processes in particular. It is because earthquake resistant technology has never been considered an integral part of general engineering education in Nepal. Moreover, construction artisans in the area are not formally trained. Their skills are passed down from generation to generation, or learnt from other masters. Of course, a small number of craftsmen who have worked abroad or in cities of Nepal have got better experience in steel fixing, concrete, etc. Without formal training, newer techniques using modern materials cannot be introduced to craftsman and technicians at massive scale. Generally, technology dissemination is considered complete once it reaches the professional engineering community and mid-level technicians. Furthermore, the academic institutions appear to be apathetic towards both non-engineered materials and traditional technologies. These are depicted as obsolete and unsafe options, despite unavailability of other options in the given circumstances. The lack of recognition of the non-engineered materials and process by the engineering community has led to a gap in use of the technology for safer construction despite these being documented in Nepal Building Code. The issue is how to disseminate this knowledge widely in digestible language to general masses and craftsman (Bothara, 2003).

- Limited options for rural reconstruction: Due to both the lack of knowledge and the attitude of the formal construction industry towards traditional materials, technologies and process has resulted in very limited research work in this sector. Which has resulted in availability of a very few options for earthquake-resilient rural reconstruction where local materials and processes dominate, and affordability and accessibility is low. The engineering sector pushed cement and steel-based construction which was not viable from economic or accessibility points of view in most of the affected areas.

- Rural vs urban reconstruction: Needs of urban and rural reconstruction are significantly different. Although the system has theoretically been equitable for urban and rural housing reconstruction, the reconstruction efforts are far more complex in urban areas, particularly because of scarcity of land, complexities involved with land registration, multi-ownership of land, dispute between kins resulting from inheritance of property, larger size of buildings, higher reconstruction costs and a need of larger cash flow for reconstruction. Particularly in old urban centres of the Kathmandu Valley, the situation is further complicated by very densely populated areas, narrow alleyways which, in general, are not motorable. There is no mechanism of soft loans.

- Lack of awareness: Although the area is highly seismically active, and there is history of large earthquakes in the area, there is a severe lack of awareness at all levels of the society. Most of the area affected by the earthquake is remote from modern information systems, and people there lack information on better construction even though they desire to know.

- Techno-legal regime: There does not exist an effective building monitoring and site enforcement mechanism for building design and construction - even in the urban areas, let alone in the rural ones. The current Building Act (GoN, 1998) envisions implementation of building code in municipal areas only, having no provision for promotion of safer buildings in rural areas. Large majority of the municipalities in Nepal lack skill, capability and infrastructure to implement the building code.
- Socio-cultural issues: The community is highly fatalistic, and earthquakes are considered to be God's way of punishing sinners. This did not let people think about the causes of disaster and possible mitigation options. Furthermore, earthquake safety is not only a technical problem, but is more a socio-cultural and economic issue that has a lot to do with one's faith, religion, fatalism, interpretation and understanding of natural phenomenon, education and affordability. Without understanding these issues, earthquake safety essentially remains illusive in Nepal. However, the formal sector largely lacks this understanding, and assumes that once any solutions are proposed people will adopt these.
- Vulnerable population: many of the districts hit by the earthquake have skewed female population due to male out-migration, which means women, who are already overburdened by their current responsibilities towards family, agriculture and livestock, will also have to take on the new and unexpected responsibility for rebuilding. Similarly, earthquake affected senior citizens, female-headed households and people living with disabilities have limited or no means for reconstruction. Other challenges are how the Dalits (i.e. backward communities), other marginalised groups and people living in the remote geographical regions who are deprived to social services could be facilitated for safer reconstruction. In addition to above, if the government support is tied with land ownership, women, Dalits and some ethnic groups who have limited ownership of land will fall out of the net.
- Availability of land for reconstruction: The earthquake caused thousands of landslides destroying or threatening many houses, settlements and livelihood. These may require reclustering. Generally, it is recommended to construct an earthquake-safer building on a flat land; however, the flat lands in hills and mountains are scarce and are used for agricultural purposes to guarantee food security. The proposals put forward after the earthquake for integrated settlement developments proposed conventional reclustering of population for ease and economy of providing services. However, it requires development of settlement on flat or near flat land – basically agricultural land - jeopardising food security of the area. Usually, people like to stay close to their land so the animals could be tendered on the land. Hence, a very balanced and sensitive approach is essential when planning any uprooting and reclustering of population. It must be resorted to only when no other in-situ solutions are available.
- Seismic assessment and strengthening of buildings: Nepal lacks both policy environment and technical capability for assessment and seismic strengthening of existing buildings on a large scale. Despite performance of some strengthened masonry buildings was excellent (Figure 11), seismic assessment and strengthening is not part of earthquake recovery.

Figure 11: A strengthened brick masonry building in mud mortar (Kathmandu valley), the building survived without any damage.

V. Reconstruction Strategy and Institutional framework

Seeing the mammoth task of reconstruction and challenges posed, a comprehensive policy for reconstruction and rehabilitation is essential which should address the challenges discussed above with a focus towards "build back better". It should recognise that the interventions in rural areas would not necessarily be suitable for urban areas because of the diverse socio-economic regimes in rural areas, their restricted accessibility to materials, technology and information, infrastructure to enforce compliance with building codes. The strategy should envision a community-based approach that shares the responsibility for reconstruction amongst as many qualified partners as possible. The salient features of the reconstruction policy are discussed below:

- House-owner driven reconstruction strategy: The policy should put special focus on house-owner driven reconstruction with prior training, information dissemination, and education and communication campaigns. It advises rebuilding with familiar methods and easily accessible materials (ensuring sustainability and cultural preferences in design); providing technical assistance during construction; use of self-labour and salvaged material; additional resources such as hired trained craftsmen; and ensuring building materials supply chains.

- Ensuring seismic safety: The policy should put special attention on incorporation of seismic safety in building construction through a review and approval mechanism for construction guidelines, structural designs, and training curricula, and through the development of reference minimum requirements that meet internationally-accepted requirements for low-cost earthquake-resistant housing. To achieve this, a well facilitated and inspected reconstruction regime is essential.
- Assisted and Inspected Reconstruction: To assist the reconstruction, implementation of seismic safety in construction, and proper quality control, it requires mobilisation of a large number of assistance and inspection (AI) teams for house-to-house advice, and subsequent inspection to certify for disbursement of cash grants in tranches through the banks. In the majority of the affected areas, the AI teams should consist of trained technical personnel and designated local community representatives.
- Uniform assistance packages: As recommended by the PDNA (NPC, 2015), the Government of Nepal (GoN) has decided to set up a uniform policy for financial grants and technical support throughout the earthquake-affected areas. It offers a uniform financial assistance package for rebuilding to all those affected. However, the government has not set-up any financial package for repair and strengthening of damaged buildings. Unless this is set-up, there is danger that the buildings which can be easily repaired and strengthened (Figure 12) against future earthquakes would potentially be demolished leading to wastage of scarce resources.

A stone masonry building with nominal damage.

A brick masonry building with toppled wall.

Figure 12: Examples of unreinforced masonry buildings which can be repaired and strengthened cost-effectively

- Partnering Organisations: After the 2005 Kashmir earthquake in Pakistan, the Earthquake Reconstruction and Rehabilitation Authority (ERRA) devised a policy of uniform technical assistance, which was based on a model of one partner organisation operating in (and responsible for) each Union Council (approximately 25,000 people). The partner organisation had technical and social mobilisation staff in multiple mobile teams delivering information, advice and assistance at village level. The teams included artisans supported by engineers. Partner organisations were supported at district level by Housing Reconstruction Centres (HRC) operated by UN-HABITAT. The HRC were focal points for training, technical support, information, advice, research and development. UN-HABITAT also acted as a partner organisation. There were no donor-built houses for individual beneficiaries, and this greatly helped avoid any difficulties in expectations, dependency, inappropriate imported models, and arbitrary or unaccountable selection of beneficiaries. There were no variations between the official technical or financial assistance each beneficiary household was eligible to receive (Mumtaz et al, 2009). Considering the success of the above structure in Pakistan, similar arrangements could be made in Nepal.

To implement the above strategy, the following requirements are to be satisfied:

- The reconstruction policy, plans and programmes need to adopt to changing expectations, requirements continuously,
- Continuous technical support/ facilitation for quality construction is essential
- For smooth functioning of the system, extreme transparency is required
- Decentralised, multi-stake holder approach, hence better planning is required
- Timely support/ inspection is a must
- Concept of incremental safety: “something is better than nothing”

VI. LESSIONS LEARNT

The following sections list a few of the learnt lessons extracted from Mumtaz (2008) after the 2005 Kashmir earthquake. Considering similar terrain, socio-cultural issues in the earthquake affected areas of Pakistan and Nepal, the following lessons could be worth considering while developing the framework for reconstruction:

6.1 Capacity-building in aseismic construction and the cascade effect

Capacity building of local engineers, technicians, craftsman, and contractors in aseismic construction is one of most important activity for long term success of the programme. The programme could be accompanied with construction of demonstration buildings; and undertaking of awareness-raising programmes for aseismic construction. The programme should be able to develop a cascade effect for dissemination of knowledge. Figure 13 presents the model used by ERRA (2006).

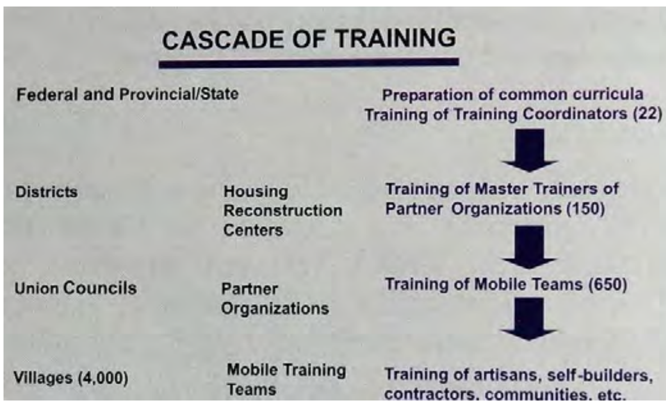

Figure 13: Cascade of Training.

The training in Pakistan covered basic technical skills for assessment and inspection, community mobilisation, and aseismic construction. These professionals are further providing training to communities and are involved in construction assessment and inspection processes.

7.1 Replication

It was clear that most of the people do not have any idea of reconstruction, and wait for each other so that the first initiative is replicated by others. It has been observed that some NGOs built semi-permanent shelter, and people replicated them. For example, in some areas, a few people used timber posts in a block masonry house, and most of the people of that area then did the same. Some people used angle-iron posts in block masonry, and most of the people of that area followed that example. Similarly, in the area where someone first used seismic bands, most of the people in that area subsequently did the same. A large number of people invested both salvaged materials and financial resources in shelters, sometimes copying shelters built by NGOs. These shelters met immediate needs for the first winter, but are not appropriate for long-term occupation because of the climate. Moreover, once there was an investment in the shelter, many did not wish to start again with a new building, preferring to incrementally improve the shelter. The lesson is that, after an earthquake, it is best to build some model houses at prominent locations with appropriate techniques before anyone else starts with wrong practices that will be replicated by the community.

7.2 Owner and mason both need training/orientation simultaneously

It has been experienced that, in order to synergise the efforts, the training for mason and owner should be mobilised simultaneously. If the mason does not know the safe techniques wanted by the house owner, the result is annoyance and frustration. On the other hand, if artisans know the techniques and the owner is not sensitized, the artisan training is wasted. Where both have been made aware, a synergy has been seen between craftsman and the house owner, resulting in better quality building.

7.3 Policy delay

The reconstruction policy was announced six months after the earthquake because of lack of preparedness for this scale of disaster in Pakistan. The delay resulted in a lot of gossips/rumours and caused confusion, frustration, and annoyance among the potential beneficiaries, particularly because of the approaching winter. Furthermore, there was delay in compliance standards being issued for the construction of houses. This further added to the confusion, although the policy was clear and consistent. In the absence of ERRA Standards, standards including half-height walls were disseminated, and some reconstruction players advised people to construct RC columns to get a roof up before the next winter, and to deal with the infill walls later. By then, many others had started to construct, especially at high altitude, in order not to miss the 2006 building season. They used their past practice of dry stone walls, but to half height, with lighter construction above. Others used the past practice of wooden posts in masonry, but changed the masonry from stone to hollow concrete blocks which they considered safer, and which were relatively cheap and quick to construct. None of these buildings incorporated any earthquake-resistant measures, were considered non-complying with ERRA requirements and thus not eligible for financial support.

7.4 Emerging building types and compliance guidelines

The initial ERRA standard was only for steel-reinforced masonry with sand-cement mortar and it put restrictions on the construction of stone masonry, though it was not viable in the first instance. Later on, other building types were added and restrictions on stone masonry construction were relaxed. The expansion of the menu of construction options was incremental, responding to demands and practices in the field. However, the process of gradual additions was perceived by some in the community as an irritation or shifting of the goal posts. In future reconstruction scenarios, a wider range of options and a locally derived menu should be prepared from the outset, with an agreed process and schedule for review and revision.

7.5 New materials. New technologies. Poor results.

It was observed that many agencies, companies and institutions tried to promote their products in the name of safe, fast construction – even though these might not stand the test. The regulatory system should be aware of this and should be able to regulate the market.

In 2006, most households managed only to construct a plinth. By early 2007, a new phenomenon became apparent - the proliferation of new, small-scale, hollow concrete blocks to meet the massive demand for blocks. This was not a consistent trend. It was most common in the areas of southern PAK. In the Khyber Pakhtunkhwa (formerly NWFP) there was also an uptake of blocks but the preference was for solid blocks. The main reason may be the increased carriage costs for solid blocks due to their weight and the increased labour costs for block laying. Hollow blocks were considered a cheap and easy way to construct a wall. However, the quality of blocks severely deteriorated because of a lack of experience among the block manufacturers and no compliance with any quality control system. More importantly, neither they nor their customers understood curing and freshly-made blocks were sold and moved the same day due to the high demand. This has led to poor quality construction.

In addition, those living in new, hollow concrete, block houses are realising the longer-term costs with respect to poor environmental performance, particularly because of high heating bills. However at the time they were constructing they were not aware of this factor. These buildings needed to be insulated, which caused an additional financial burden on the building owner.

7.6 Engineering advice and local circumstances

When formulating policy, guidelines or advising, it is extremely important that the person-in-charge understands its implications, its acceptance from an affordability point of view and cultural acceptability. Under ERRA policy, a restriction was put on the use of stone masonry and insisted on the use of cement-based mortar and steel reinforcement for building construction. These were not feasible options because of the high cost of transport of cement, sand and steel. In many areas there was not any alternative to stone. When advising site selection the guidelines advised that the distance between house and background should not be less than the height of the house, if the house were to be constructed on a slope. Compliance with this advice was not possible in many areas because of a lack of space.

7.7 Quality control issues

With increasing pressures on the construction market, deterioration in material quality and construction skills was observed. It became obvious that it is not possible for owners to achieve good results and the desired earthquake resistance if they are not protected by regulatory standards with regard to key materials. The role and responsibilities of owners, technical support and government developed or improved in response to problems or needs during this process, but could have been better anticipated and planned. For future disasters, at least the role of governments with regard to policy, standards and regulatory frameworks needs to be prepared.

During the process, UN-HABITAT advocated better regulation of the material supply and the fabrication sector and assisted through training and information both block makers and house owners. As a consequence block testing was introduced by ERRA and block standards improved significantly in most of the areas. The lesson is: if committed regulatory systems are in place, they work.

7.8 Remedial measures

Due to the delay in the confirmation of standards, many who started early construction found themselves non-compliant with basic safety standards and thereby failed to be certified or approved for financial assistance. UN-HABITAT and its partners put considerable effort into identifying the common and serious defects and providing solutions for house owners to carry out additional remedial works to improve the safety of those houses. These remedial measures were intended for new construction, not damaged construction, but could be used for retrofitting any substandard buildings. The measures were all tested on site to check and improve their practical application, to train the engineers and artisans and to generate information materials. Site testing also helped to ascertain the time, material, skill level and costs involved, all of which are key criteria in the process of convincing owners to carry out the additional work.

The significant focus on remedial measures meant an increased knowledge and skill level among not only the engineers, but also the artisans. It has also meant that an active diagnosis and response capacity, and the ongoing development and testing of solutions is required. It also needed to consider engineering principles as well as cost benefit analysis. This means not only equipping the technical support team with a curriculum and training skills, but also with investigative skills and improved judgement. Basically this means not only equipping them with a few answers, but with the right questions. This is an important consideration in the development of technical support capacity for a fluid and fast moving reconstruction programme which is indeed driven by owners and the field and not by predetermined assumptions of the engineering community.

7.9 Dynamism in curricula

It is important not to assume that it is adequate to rely on a fixed curriculum for technical training, without reviewing the field practices of those who have participated in training and the very large number of other artisans who have not received training. They may have specific patterns of malpractice, such as no mortar in vertical joints, levelling mortar beds flat in stone work and letting it set before laying the next course, thereby reducing the bond. The diagnosis of workmanship issues and shortcomings in interpretation of new technology needs to be used to evolve the curriculum, or to devise supplementary activities to reinforce it.

7.10 Orientation to women

It has been observed that women are more dedicated, responsible and sensitive to achieving quality construction. In a few places, masons were complaining that they interfere with their work. This means that these women were committed to see what they had learned was implemented in their houses.

7.11 Decentralization and Community Based Organisations/ Non-Governmental Organisations

Some of the good lessons learnt during the reconstruction and rehabilitation programme were the benefits of decentralisation of all the activities, assisting people at their home sites, maintaining livelihoods and community support networks and enabling them to rebuild early.

Community-based and non-government organizations (CBOs and NGOs) played an effective role in the overall reconstruction and rehabilitation programme by systematically mobilizing the community. As these organisations came mostly from within local communities, they had a better understanding of need and supply and penetration in the community. They also better understand the sensitivities and cultural aspects. They played an effective role in information dissemination and oriented and mobilised the community. It is hoped that these CBOs/NGOs will help the sustainability of seismic safety in rural areas in Nepal.

The greatest contribution to sustainability is the widespread participation in responsibility for safer buildings by not leaving it to a few contractors or to NGOs, or to the government. Those who had to learn how to comply with standards to get their financial assistance were the owners themselves and their masons. This means that over 500,000 people had to actively seek information, training or advice on seismic-resistant construction, whether formally from a HRC, or informally from a neighbour, or by visiting a model house. From this wide base of awareness and knowledge there is a greater chance of long-term adoption and replication.

VII. Conclusion

This paper has discussed the expected challenges of reconstruction after the 2015 Nepal Earthquake, the policy directives and strategy to tackle the challenges, the tools required for awareness raising, and the reconstruction and the impediments. The paper takes example of the 2005 Kashmir earthquake where one of the authors worked extensively for gap assessment, capacity building of local stakeholders, awareness raising and sensitisation, and in construction of demonstration buildings. The lesson learnt was that in a massive disaster where there is an informal construction regime, the owner-driven reconstruction is the most suitable approach. However, for success of the programme it has to be facilitated by the formal sector. A timely response, a dedicated interface between house-owners and formal sector, room to adapt the programme with changing needs, a well-managed training programme and decentralisation of power are equally important factors to be considered.

The owner-driven approach helps the building owners to adopt their cultural identities in the construction, and continue traditional construction with incorporation of aseismic features. It also helps to expedite the construction pace and equity, and the sustainability of the process in the rural areas without creating massive infrastructure. However, it should be noted that same could not be a case in urban reconstruction due to different socio-economic regime, legislative system, land and planning issues.

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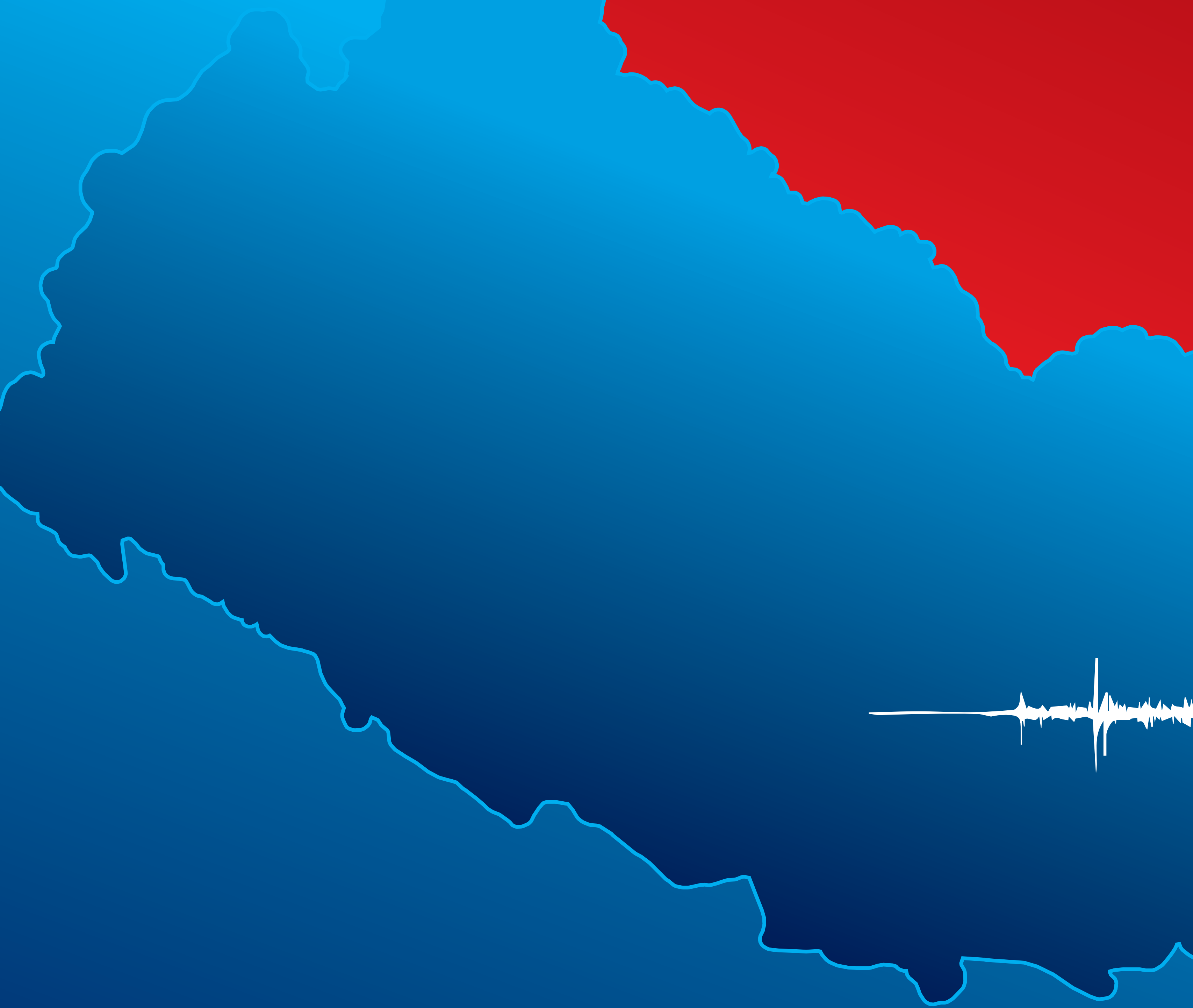
15825 Shady Grove Road, Suite 100 · Rockville, Maryland 20850, USA

Tel: 301.590.3939 · Fax: 301.948.7174 Email: marketing@sheladia.com



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