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Study of Seismic Capacity of Existing RC Buildings with Masonry Infill Damaged by Past Earthquakes

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ABSTRACT

Many of the buildings which experienced damage in recent earthquakes such as the 2015 Nepal Earthquake were RC buildings with unreinforced masonry infill walls. This study proposes a simplified procedure to estimate the in-plane seismic capacity of masonry infilled RC buildings based on concepts of the Japanese seismic evaluation standard. The seismic capacity and observed damage correlation of a database including 370 existing RC buildings with masonry infill that experienced earthquakes in Taiwan, Ecuador and Nepal is investigated. The I_s index, which represents the seismic capacity of buildings in the Japanese standard, showed good correlation with observed damage and proved to be effective as a simple method to estimate seismic capacity. The method was then applied to 103 existing buildings in Bangladesh which have not experienced a major earthquake recently. The results emphasize the necessary and urgent seismic evaluation and retrofitting of buildings in Bangladesh.

1 INTRODUCTION

Japan experienced many devastating earthquakes in the last century and developed a practical standard for seismic evaluation (The Japanese Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings (JBDPA 2001). The standard has proved its effectiveness in the Great East Japan Earthquake of 2011 where most of the evaluated buildings, which if necessary have been retrofitted based on the standard evaluation, showed good performance and retrofits prevented severe structural damage (Maeda et al 2012).

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Nevertheless, a procedure that incorporates the effect of masonry infill walls as lateral force resisting members is not mentioned in the standard because masonry walls are not commonly constructed in Japan. Because of this, many of the damaged buildings in earthquakes such as the 2008 China Wenchuan Earthquake, 2015 Nepal Earthquake and 2017 Mexico Earthquake were unevaluated reinforced concrete buildings with partitions made of masonry walls as shown in Figure 1. Masonry partition walls were commonly considered to be non-structural elements and the structures were designed as RC moment resisting frames ignoring the influence of walls. But masonry infill walls can completely change the behaviour of structures as noted by many researchers in several studies such as (Paulay and Preistley 1992). There is a need to estimate the capacity of different orientations of masonry infill walls in other countries to identify buildings that are vulnerable to damage during earthquakes. In addition to the masonry infill walls, the seismicity level is unique to each country. Thus, an appropriate seismic capacity based on the seismic demands of a region would need to be estimated based on past damage data.

The purpose of this study is twofold: First, propose a simplified procedure to estimate in-plane strength and ductility of masonry infill based on a review of experimental results. Second, evaluate the proposed procedure to estimate the seismic capacity of 470 existing RC buildings in several countries (Taiwan, Ecuador, Nepal and Bangladesh) based on the evaluation methods of the Japanese standard. The seismic capacity of the buildings in the database and correlation to observed damage is investigated and recommendations for seismic criteria are discussed.

Figure 1: Damage of RC building with masonry infill walls in China 2008 Wenchuan Earthquake

2 OVERVIEW OF THE JAPANESE STANDARD

The JBDPA standard (JBDPA 2001) has three screening levels. The 1st level is the simplest and most conservative, and the 3rd level is the most complex with detailed calculations. In the 1st level screening, only the strength of concrete and the sectional areas of columns and walls are considered to estimate the seismic capacity. This study will focus on the 1st level evaluation since the investigated database of existing buildings have simple drawings showing only basic information.

Only an overview of the concept of the standard is presented here. The seismic capacity of a building is expressed by the I_s-index and is calculated by Eq. 1 for each story.

$$I_s = E_o \cdot S_D \cdot T \tag{1}$$

S_D and T are reduction factors that modify the basic seismic index of a structure (E_0) because of structural irregularity and deterioration after construction, respectively. E_0 is the product of the strength index (C), ductility index (F) and story index (n+1/n+i) as shown in Eq. 2.

$$E_0 = \frac{n+1}{n+i} \cdot (C_1 + \alpha_2 C_2 + \alpha_3 C_3) \cdot F_1 \tag{2}$$

C is the strength index that denotes the story-shear coefficient of each structural member. F is the ductility index of each member ranging from 0.8 (extremely brittle) to 3.2 (most ductile), depending on the sectional properties such as bar arrangement, shear-to-flexural-strength ratio etc. $(n+1)/(n+i)$ is the story index which accounts for the mode shape of the response along the building height. α is the effective strength factor which reduces the effective strength of ductile members at ultimate deformation of stiff members.

3 SEISMIC CAPACITY OF MASONRY INFILL

3.1 Lateral strength and C-index of masonry infill

The proposed evaluation considers masonry infill and boundary columns as two separate elements. This is assumed because masonry infill eventually delaminates from the surface of the surrounding columns and may fail before the boundary columns. Thus, the C-index of masonry infill and C-index of columns are calculated separately. If ductile enough, the columns would continue to carry the lateral load and would fail at a larger drift as shown in Figure 2.

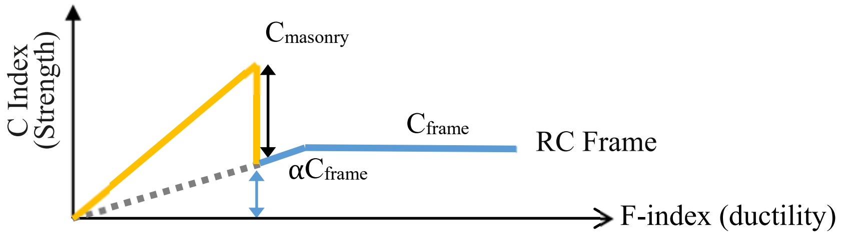


Figure 2: Main concept of calculating C-index of masonry infill

$C_{masonry}$ for the 1st level evaluation for every element is shown in Eq. 3.

$$C_{masonry} = \frac{Q_u}{\sum W} = \frac{\tau_{inf} \cdot A_{inf}}{\sum W} \tag{3}$$

Q_u is the ultimate lateral load-carrying capacity of the vertical members of a story. $\sum W$ is the weight of the building supported by the story. τ_{inf} is the shear strength of masonry. A_{inf} is the cross-sectional area of the masonry infill panel taken as the product of l_{inf} (infill panel length) and t_{inf} (thickness of infill). The calculation of the cross-sectional area, A_{inf}, and the weight of the building are straightforward. However, the estimation of the shear strength of the infill, τ_{inf}, varies greatly based on the type and quality of masonry.

This study briefly discusses the results of past experimental studies in order to understand the parameters influencing the shear strength of masonry infill. Figure 3 shows the relation between the shear strength of the masonry infill and the masonry prism compressive strength, f_m, based on a database of experimental results from 9 different researchers as summarized in (Alwashali 2018). The database consists of single-story, single-span RC frames with masonry infill tested under static cyclic loading with several types of masonry bricks. As shown in Fig. 3, the shear strength of the masonry infill, τ_{inf}, generally ranges between 0.2N/mm^2 and 1N/mm^2. The shear strength, τ_{inf} commonly ranges between $0.04f_m$ and $0.1f_m$.

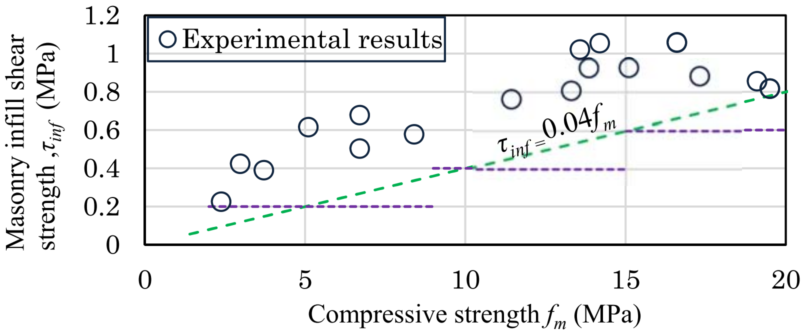


Figure 3: Relation of prism compressive strength to shear strength of masonry infill

Another important parameter influencing the shear strength of the masonry infill is the ratio of lateral strength of the boundary frame to the shear strength of the masonry infill. In general, a strong RC frame surrounding the masonry infill will increase the confinement of masonry infill and thus increase its shear strength. To classify the frames as weak or strong, the β-index is used which is defined as the ratio of the expected bare frame lateral strength to the expected masonry infill strength as shown in Equation.4.

$$\beta = Vf / V\,\mathrm{inf} \tag{4}$$

V_f is the lateral strength of the boundary frame which is calculated as the ultimate flexural capacity, assuming plastic hinges form at the ends of the columns (or plastic hinges at the end of the beams in the case of weak beam and strong column). V_{inf} is the expected lateral capacity of the masonry infill computed by Equation 5. This is a simple prediction assuming τ_{inf} is equal to $0.05 f_m$.

$$V_{\mathrm{inf}} = 0.05\, f_m \cdot t_{\mathrm{inf}} \cdot l_{\mathrm{inf}} \tag{5}$$

Figure 4 shows the relation between the *β-index* and the shear strength of the masonry, (τ_{inf} is normalized by the prism compressive strength, f_m). An increase in the strength of the surrounding frame will also improve the shear strength of masonry infill.

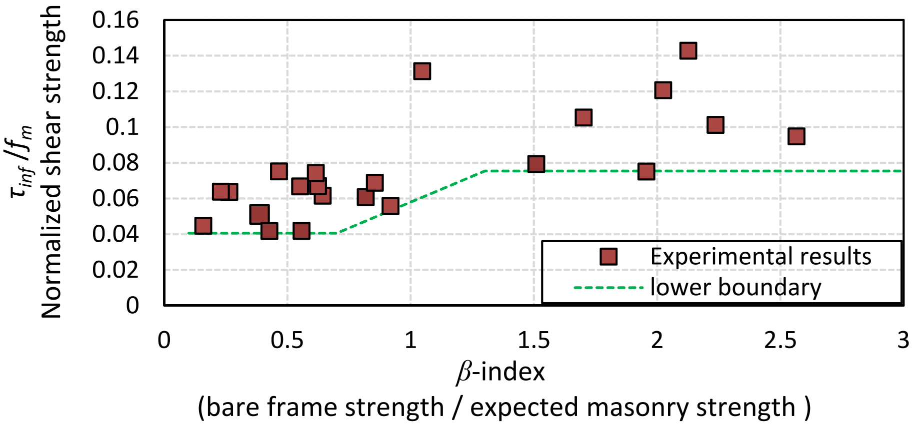


Figure 4: Relation of β index to shear strength of masonry infill

Evaluating the *β-index* requires some investigative effort such as knowing the reinforcement details of the RC frame. In this paper, the scope focuses on the simplified 1st level evaluation, thus ignoring the influence of the *β-index* and the lower boundary of shear strength is used. A more detailed evaluation method considering the *β-index* is proposed elsewhere (Alwashali 2018).

Additionally, acquiring the data of the prism compressive strength of masonry infill of existing buildings may be difficult during the 1st level evaluation process. Therefore, in the absence of material tests the shear strength of masonry infill is proposed to be the lower boundary of the expected masonry compressive strength as shown Table 1 and Figure 3.

Table 1: Proposed shear strength of masonry infill for 1st level evaluation

Compressive strength of masonry fm (N/mm^2)	Proposed shear strength of masonry infill (N/mm^2)
$3 < f_m < 9$	0.2
$9 < f_m < 15$	0.4
$f_m > 15$	0.6

3.2 Proposed F-index of masonry infill

This paper briefly discusses the results related to deformation limits for masonry infill based on a database of experimental studies that are summarized in (Alwashali 2018). R_{max} and R_u represent the story drift at maximum strength and story drift when the lateral strength degraded to 80% of maximum lateral strength, respectively. R_{max} has an average of 0.72% and standard deviation of 0.36%. R_u has an average of 1.7% and standard deviation of 0.77%.

There are several parameters influencing the deformation limits of masonry infill such as brick type, mortar strength and the relative strength of the surrounding frame. An important parameter is the *β-index* (indicating the relative strength of frame to masonry infill strength as shown in Eq. 4). Fig. 5 shows the relation between the *β-index* and R_u. A higher *β-index* (relatively stronger frame) would correspond to more ductility and $\beta<0.5$ (relatively weak frame) would indicate less ductility. Such a relation between deformation limits and β-index was also noted in other seismic evaluation standards such as (ASCE/SEI 41 2007).

Also, experimental results showed that specimens with values of $\beta<0.5$ (relatively strong infill and weak frame) showed brittle response and a sudden drop of strength after reaching its maximum strength as shown in Fig. 6 which shows test specimen F-0.4 in (Alwashali 2018).

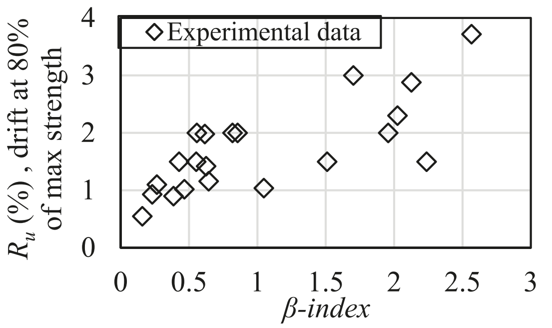


Figure.5: Relation of between β-index and R_u drift

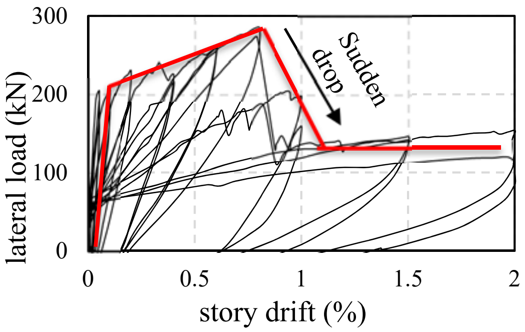


Figure. 6: Specimen with $\beta<0.5$ (weak frame)

The 1st level screening in the Japanese standard conservatively limits the ductility index to a maximum value of $F=1$ for ductile vertical-resisting elements. This corresponds to a story drift of about 0.4%. In other words, the columns and RC walls are assumed to fail as brittle elements at a story drift of about 0.4%. In this study, since it focuses on the 1st level evaluation, the F-index for masonry infill is taken as 1 and the influence of the β-index is conservatively ignored. A more detailed evaluation of the F-index for masonry infill for the 2nd level screening is discussed in another study by the author (Alwashali 2018).

4 APPLICATION OF THE EVALUATION METHOD

Three different recent earthquakes in Taiwan, Ecuador and Nepal are investigated. Those countries were selected based on the availability of documented damage data. The data are collected from an open data website named Datacenterhub (2015, 2016). The data is from field surveys of RC buildings damaged by the earthquakes collected as a reconnaissance effort led by ACI and several other organizations. The data of the buildings contains locations with GPS coordinates, simple sketches of plans for each building showing cross-sectional areas of columns and masonry infill, and photos of damage.

The classification of damage level of the buildings in the database is as follows:

I. Light: Hairline (crack width < 0.13 mm) inclined and flexural cracks were observed in structural elements.

II. Moderate: Wider cracks or spalling of concrete.

III. Severe: At least one element had a structural failure.

Since the database does not have detailed drawings nor material specifications, the following assumptions are used in the calculations:

a) The shear strength of columns commonly ranges 0.7N/mm^2~1.5N/mm^2, which is shown in the Japanese standard (JDBPA 2001). In this study, an average of 1N/mm^2 is used if no material property data exists.

b) The effective strength factor used for columns, α, is 0.7, (Fig. 2 and Eq. 2), which is recommended in an experimental study by (Sen et al 2018).

c) The average weight per unit area of RC buildings is 11kN/m^2 in the absence of data.

d) The strength contribution of masonry infill walls with large openings (greater than 40%) is ignored.

e) S_D and T (reduction factors for structural irregularity and deterioration after construction respectively) are taken as 1 for simplicity.

f) Masonry infill is commonly confined by the boundary columns and thus in-plane failure is considered to occur prior to the out-of-plane failure.

4.1 Taiwan Earthquake 2016

4.1.1 Overview of the data and the earthquake

An earthquake of magnitude 6.7 occurred in Meinong, Taiwan on February 6, 2016. The earthquake caused large-scale damage in Tainan city which was 40 km from the epicenter. The data of damaged existing RC buildings is provided by (Datacenter hub 2016). 65 RC buildings with masonry infill are investigated in this study and the locations are shown in Figure 7.

Several ground motion stations in Tainan city recorded values of PGA between 0.2g and 0.4g. The maximum recorded PGA was 0.45g and was measured by station CHY 62. The response acceleration plots use 5% damping and are shown in Figure 8. Most of the response spectra have values of acceleration less than 0.8g for short periods (less than 0.5 seconds), except for station CHY 62.

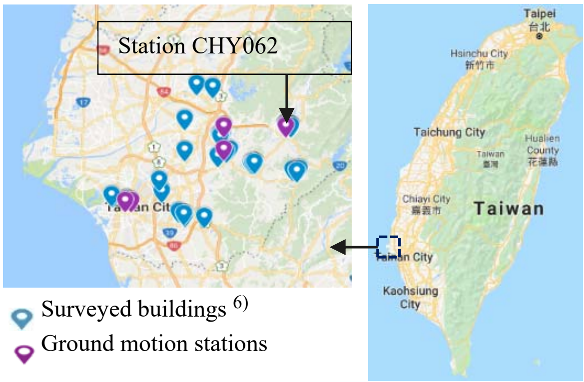


Figure. 7: Locations of the investigated buildings

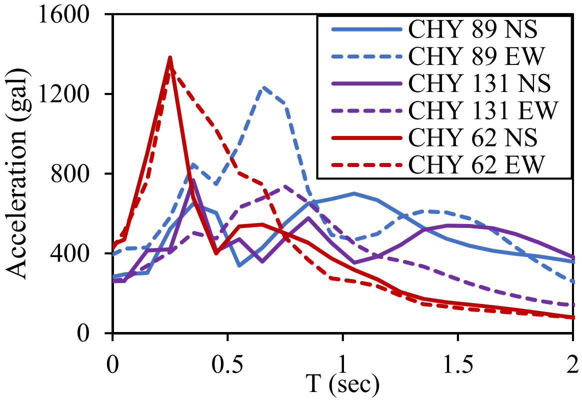


Figure 8: Acceleration response spectra in Taiwan EQ.

The number of stories ranged between 1 and 5 stories, with the majority of buildings having 2 or 3 stories. Details of the investigated buildings are stated in (Alwashali 2018). Masonry infill walls in Taiwan are commonly made of red clay bricks and are 200 to 300mm thick. The types and strengths of masonry infill were not stated in the database, but as stated in several studies such as (Chiou et al 2017), an expected shear strength of 0.4 N/mm^2 is used for seismic evaluation.

4.1.2 Seismic capacity results

Figure 9 shows the I_s-indices for NS and EW directions in the 1st story of the investigated buildings with observed damage levels. Figure 10 shows the variation of I_s-index with number of buildings. There is a clear trend between damage level and low values of I_s-index. In Japan, the I_{so}-index (demand criteria) is 0.8 and 0.6 for 1st and 2nd level screening, respectively.

Table 2 shows the average and standard deviation of the I_s-index for the investigated buildings. Figure 11 shows the log-normal distribution. It should be noted that the curve for severely damaged buildings in Figure 11 shows the ratio of severely damage buildings to all buildings which is adjusted by multiplying the ratio 17/65 (severely damaged/ all surveyed buildings). An I_s-index of 0.5 or 0.6 would be sufficient to avoid severe damage. However, the problem is that most of the investigated buildings, as shown in Fig. 11, have seismic capacities less than 0.6 and retrofitting such large stock of buildings would be expensive.

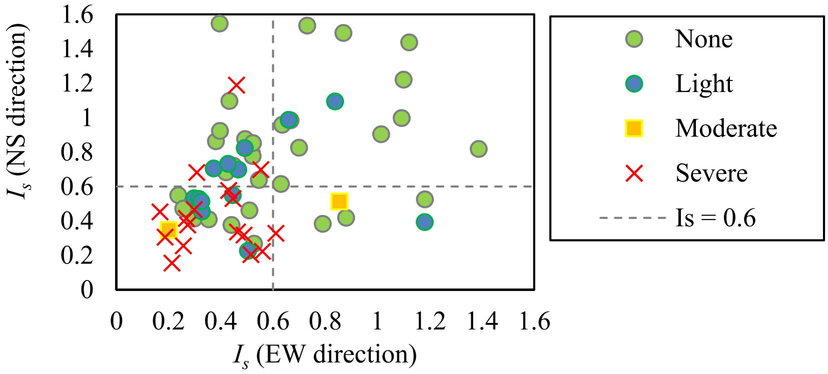
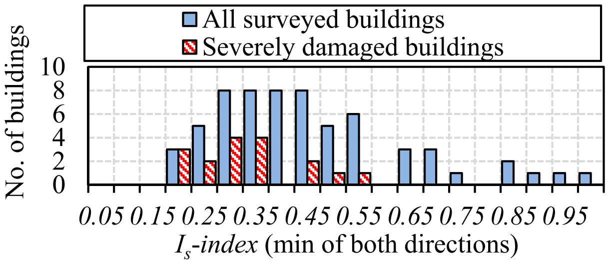


Figure. 9: Relation of I_s-index and damage in Taiwan EQ



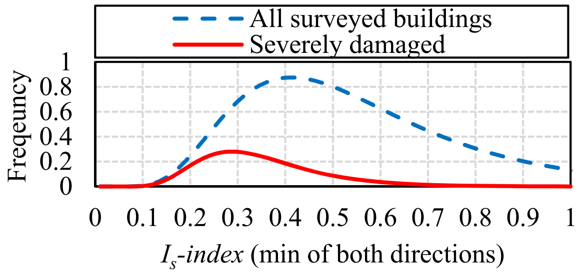


Figure. 10: Distribution of Is index in Taiwan

Figure. 11: Normal distribution of I_s index in Taiwan

Table.2 Average I_s-index of buildings in Taiwan EQ 2016

	All buildings	Severely damaged
Number of bldgs.	65	17
Average I_s-index (min direction)	0.458	0.306
Standard deviation	0.221	0.11

4.2 Ecuador Earthquake 2016

4.2.1 Overview of the data and the Earthquake

An earthquake with a moment magnitude M_w =7.8 occurred in Ecuador on April 16, 2016. The reconnaissance data consists of 171 low-rise RC buildings in the cities of Manta, Portoviejo, Chone, and Bahía de Caráquez which are located in the province of Manabí (Datacenter hub 2016), shown in Figure 12.

Masonry infill type is not stated in the database, but as noticed from photos of the survey, both concrete blocks and burnt clay bricks are commonly used. In a study by (Cevallos et al 2017), it was found that solid clay bricks had unit compressive strengths between 7.3 and 7.9 MPa and most concrete block units between 1.0 and 1.5 MPa. Thus, the masonry infill shear strength was taken as 0.2 N/mm^2 using the lower boundary for shear strength in Table 1. The acceleration response spectra using 5% damping is shown in Figure 13. The acceleration record comes from ground motion station AMNT and it is the nearest station to the surveyed buildings in Manta city. The acceleration response for buildings with short periods (less than 0.5 seconds) in NS and EW directions exceeded 1g.

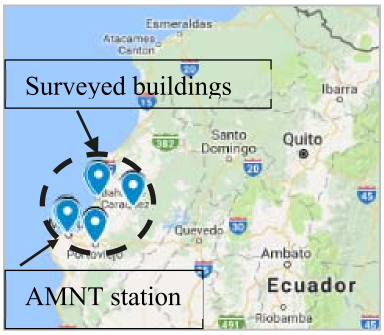


Figure. 12: Locations of the investigated buildings

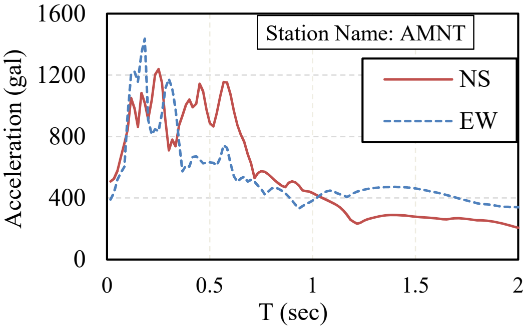


Figure. 13: Acceleration spectra in Ecuador EQ

4.2.2 Seismic capacity results

Figure 14 shows the I_s-indices for NS and EW directions in the 1st story of the investigated buildings with observed damage levels. Figure 15 shows the variation of I_s-index with number of buildings. Table 3 shows the average and standard deviation of the I_s-index of the investigated buildings. The average I_s-index of the investigated buildings in Ecuador is 0.32, which is lower than Taiwan. Figure 16 shows the logarithmic normal distribution of the I_s-index. Like Taiwan, buildings with a seismic capacity greater than 0.5 avoided severe damage. However, most of the buildings lie below this range as shown in Figure 14.

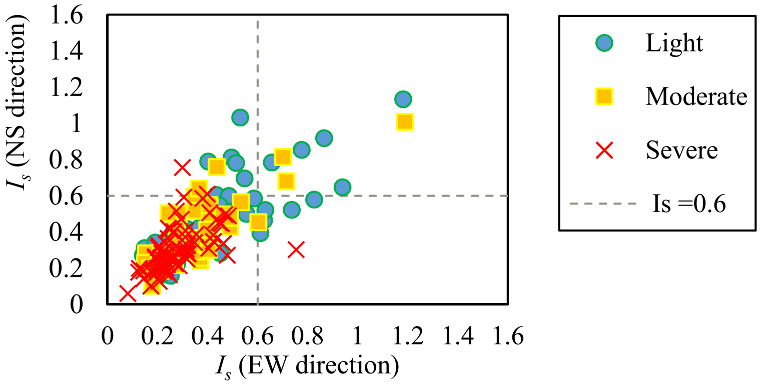
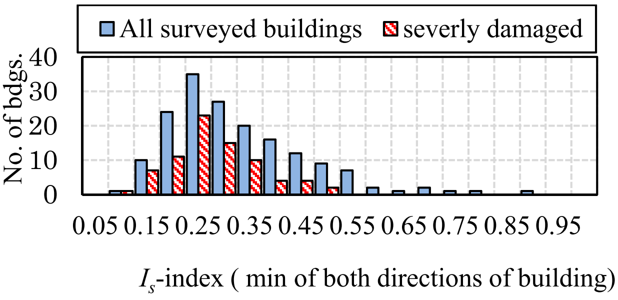


Figure. 14: Relation of I_s-index and damage in Ecuador EQ

Table 3: Average Is index in Ecuador EQ

	All buildings	Severely damaged
Number of bldgs.	171	77
Average I_s-index (min direction)	0.316	0.255
standard deviation	0.159	0.087



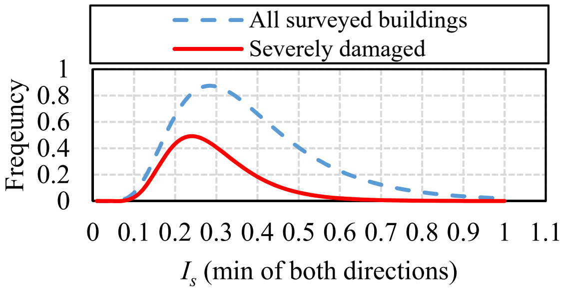


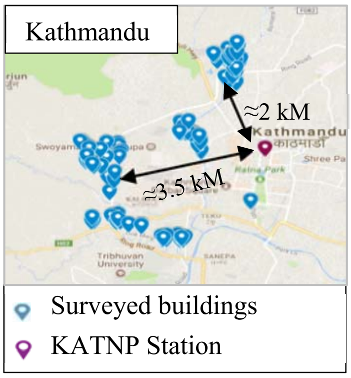
Figure 15: Distribution of I_s-index in Ecuador EQ

Figure: 16 Normal distribution of I_s-index

4.3 Nepal Earthquake 2015

4.3.1 Overview of the data and the Earthquake

A strong ground shaking with a moment magnitude Mw= 7.3 struck near the center of Nepal on April 25, 2015. Surveys of 134 RC buildings with masonry infill located in the capital city, Kathmandu (Datacenter hub 2015) are shown in Figure 17. The response spectra using 5% damping are shown in Figure 18. The response acceleration spectra for short periods for NS and EW directions is 0.3g and 0.6g, respectively. The ground motion station is relatively near the investigated buildings (Fig. 17).



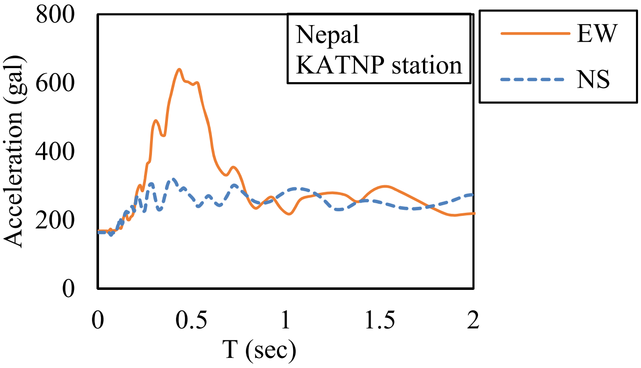


Figure 17: Locations of the investigated buildings

Figure 18: Acceleration response spectra in Nepal EQ.

The thicknesses of the masonry infill walls were 110 mm to 220 mm. The types and strengths of masonry infill were not stated in the database, but it was noticed from the photos of the survey that solid burnt clay bricks were the most common. The prism compressive strengths of masonry infill within the same region was investigated in other studies by (Pradhan et al 2017), where the compressive strength was about 4.1 MPa. Therefore, the masonry`s shear strength is taken as 0.2N/mm^2 as proposed in Table 1.

4.3.2 Seismic capacity results

Figure 19 shows the I_s-indices for NS and EW directions in the 1st story of the investigated buildings with observed damage levels. Figure 20 shows the variation of I_s-index with number of buildings. Table 4 shows the average and standard deviation of the I_s-index of the investigated buildings. Figure 21 shows the log-normal distribution of the I_s-index. The average seismic capacity of buildings in Nepal is lower than that of buildings in Taiwan and Ecuador. Like Ecuador and Taiwan, the buildings in Nepal with I_s-indices greater than 0.5 avoided severe damage. It should also be noted that the response acceleration in Nepal is much smaller than that from the Taiwan and Ecuador earthquakes as shown in Fig. 18. Thus, an I_s-index in Nepal of 0.5 is relatively high compared with the earthquake demand and this point needs further investigation.

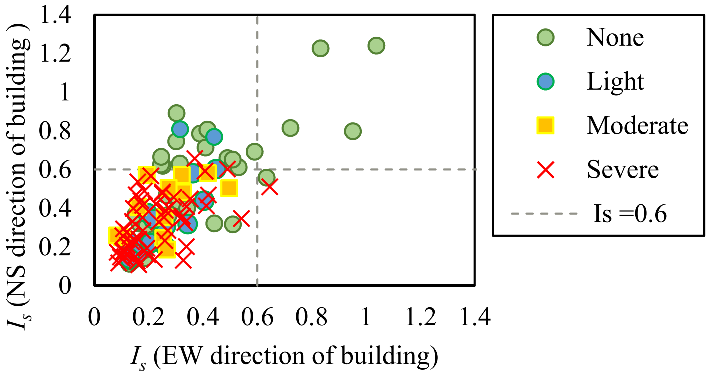
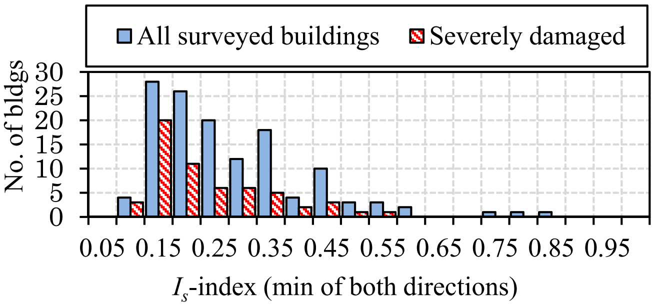


Figure. 19: Relation of I_s-index and damage in Nepal EQ



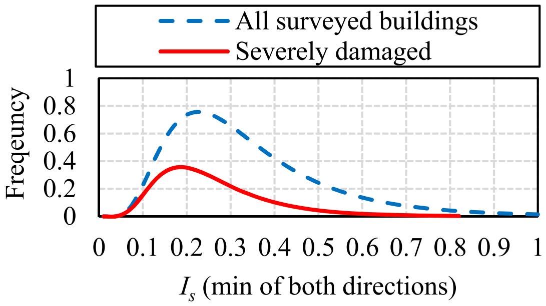


Figure 20: Distribution of I_s-index of buildings

Figure: 21 Normal distribution of I_s-index

Table 4: Average I_s-index of buildings in Nepal EQ

	All buildings	Severely damaged
Number of bldgs.	134	58
Average I_s-index (min direction)	0.261	0.209
Standard deviation	0.157	0.108

5 APPLICATION IN COUNTRIES WITH NO RECENT EARTHQUAKES

5.1.1 Overview of the data in Bangladesh

Bangladesh is a densely populated country located in a moderately seismic-prone region. The data in this study are collected in Dhaka city, the capital of Bangladesh.

Data from 103 existing RC buildings were collected from two sources:

1- CDMP (Comprehensive Disaster Management Program) collected data between 2010 and 2015 in a collaborative project between the Bangladeshi Ministry of Disaster Management and UNDP (United Nations Development Program). The building data contains sketches of plans of buildings that include dimensions of columns and masonry infill walls. More details included in the database are mentioned in (Alwashali et al 2018).

2- A site investigation for a Japanese project called SATREPS ,TSUIB 2015 consisted of a team of researchers from Tohoku University including the author and governmental engineers from the Public Works Department in Bangladesh. Four existing RC buildings were surveyed in January 2018. Detailed drawings of the buildings are mentioned in (Alwashali et al 2018).

In general, the surveyed buildings have two thicknesses of masonry wall. The exterior walls are 250mm thick and the interior walls are 125mm thick. The walls are made of burnt clay brick. The compressive strength of these types of bricks is expected to be 15 MPa. The mortar strength is expected to be between 3 and 5 MPa. The expected masonry prism compressive strength may vary between 6 and 10 MPa. Therefore, the shear strength of the masonry infill is taken as 0.2 N/mm^2 as proposed in Table 1.

The distribution of the seismic capacities of the investigated buildings in Dhaka that are calculated based on the proposed method is shown in Figure 22. The average I_s-index is 0.19 with a standard deviation of 0.11.

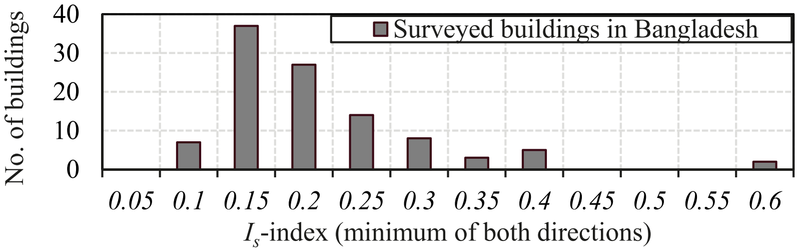


Figure 22: Distribution of I_s-index of the investigated buildings in Dhaka city

5.1.2 Seismic capacity of buildings and comparison with nearby countries

There have been no major earthquakes in Bangladesh in the last 100 years. As Bangladesh and Nepal both border India, there is a similar construction practice for RC buildings with masonry infill walls.
The design spectral acceleration curves are calculated based on the 2015 BNBC Code (draft) for soil type SC for Dhaka city. The design spectra is compared with the response spectra of the 2015 NepalEQ in Figure 23.

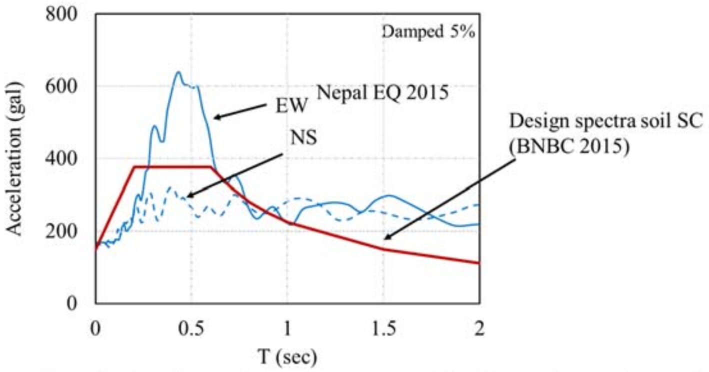


Figure 23 Response spectra for Dhaka (based on 2015 BNBC draft) and Nepal Earthquake

A comparison of the observed damage in buildings in Nepal with the seismic capacities of the investigated buildings in Dhaka is shown in Figure 24. Bangladeshi buildings may suffer severe damage if an earthquake were to occur. These results emphasize the importance and urgency for earthquake retrofitting in Bangladesh, otherwise the damage and causalities could be more than that of the Nepal 2015 earthquake.

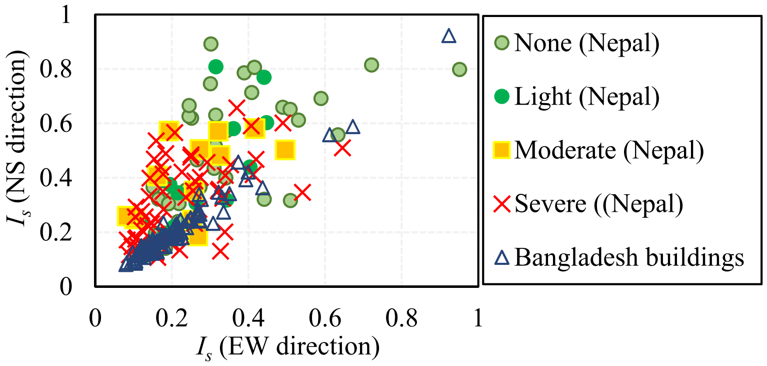


Figure 24 Comparison of Existing RC buildings in Bangladesh and damage observed in Nepal Earthquake

6 CONCLUSION

This study proposes a simplified procedure to include masonry infill walls into the Japanese Seismic Evaluation Standard of existing RC buildings (JBDPA, 2001). The seismic capacity (I_s-index) of masonry infilled RC buildings in several countries are investigated and the following are the main points:

- There is a clear relation between low values of seismic capacity and damage level in the investigated buildings from three previous earthquakes (Nepal EQ 2015, Ecuador EQ 2016 and Taiwan EQ 2016). Severe damage is concentrated in buildings with values of the I_s-index less than 0.3. On the other hand, buildings with an I_s-index greater than 0.6 avoided severe damage. An I_s-index of 0.6 for masonry infilled buildings would be sufficient to avoid severe damage.
- The unfortunate reality is that most of the investigated buildings have seismic capacity indices less than 0.6. Retrofitting so many buildings would be a high economical cost and would be difficult to apply in the near future, especially in developing countries.

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