- Technical Paper-

STUDY ON SEISMIC CAPACITY OF EXISTING RC BUILDINGS WITH MASONRY INFILL BASED ON PAST EARTHQUAKES DAMAGE

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ABSTRACT

Many of the buildings which experienced damage in the recent earthquakes such as 2015 Nepal Earthquake, were RC buildings having partitions of masonry 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 for a database of 370 of existing RC buildings with masonry infill that experienced earthquake in Taiwan, Ecuador and Nepal is investigated and recommendations for seismic criteria are discussed.

Keywords： Existing RC buildings, Masonry infill, Seismic capacity evaluation.

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) [1]. The standard has proved its effectiveness in the Great East Japan Earthquake of 2011 where most of the evaluated buildings, which if necessary have retrofitted based on the standard evaluation, showed good performance and prevented severe structural damage [2].

Nevertheless, a procedure that take the effect of masonry infill walls as lateral force resisting members is not mentioned in the standard, since masonry walls are not commonly constructed in Japan. In this regard, many of the buildings which experienced damage in the many earthquakes such as 2008 China Wenchuan Earthquake, 2015 Nepal Earthquake and 2017 Mexico Earthquake, were reinforced concrete buildings having partitions of masonry walls as shown in Fig.1.

Fig. 1 Damage of RC building with masonry infill in China 2008 Wenchuan earthquake

Those masonry partitions walls were commonly considered as non-structural elements and the structures were designed as RC moment resisting frames ignoring their influences. However, masonry infill walls can completely change the behavior of structures as noted by many researchers in several studies such as [3]. There is a need to evaluate capacities 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 varies in other countries and Japan. Thus, an appropriate seismic capacity based on the seismic demand of a region would need to be estimated based on past damage experiences.

The purpose of this study is twofold: First, propose a simplified procedure to estimate in-plane strength and ductility of masonry infill based on review of experimental results. Second, the proposed evaluation was conducted to evaluate the seismic capacity for 370 existing RC buildings in several countries (Taiwan, Ecuador and Nepal) based on the concepts of the Japanese standard. The seismic capacity of the buildings in the database and correlation of damage is investigated and recommendations for seismic criteria of the investigated countries are discussed.

2. OVERVIEW OF THE JAPANESE STANDARD

The JBDPA standard [1] has three screening levels. The 1st level is the simplest and most conservative, where the 3rd level is the most complex with detailed calculations. In the 1st level, 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 1st level evaluation since the investigated database of existing buildings have only 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.

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

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

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

Where, C index is strength index that denotes the base-shear coefficient of each structural member. F index denotes 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, member proportion, shear-to-flexural-strength ratio etc. $(n+1)/(n+i)$ is story index that is a modification factor 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 surrounding columns as two separate components. This is assumed because masonry infill at some drift limit get separated from the surrounding columns and could fail before the surrounding columns. Thus, the C-index of masonry infill and C-index of columns are calculated separately. If the columns are ductile enough, then the columns would continue to carry the lateral load and would fail at a larger drift as shown in Fig. 2.

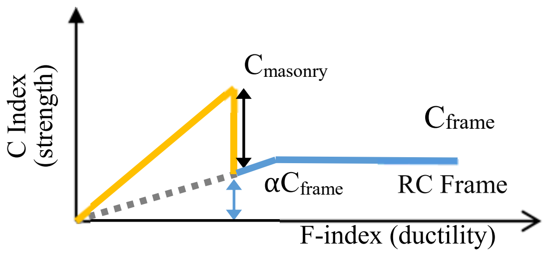

Fig.2 Main concept of calculating C-index of infilled frame

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

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

Q_u is ultimate lateral load-carrying capacity of the vertical members in the story concerned. $\sum W$ is the weight of the building supported by the story concerned. τ_{inf} is the shear strength of masonry. A_{inf} is the cross-sectional area of masonry infill panel taken as product of l_{inf} (infill panel length) and t_{inf} (thickness of infill panel).

The calculation of cross-sectional area, A_{inf}, and weight of building are straightforward. However, the estimation of shear strength of infill, τ_{inf}, varies greatly based on type and quality of masonry infill.

This study briefly discuss results of past experimental studies, in order to understand the parameters influencing shear strength of masonry infill.

Fig. 3 shows the relation between shear strength of masonry infill and masonry prism compressive strength, f_m, based on a database of experimental results from 9 different researchers and summarized in [4]. The database are of specimens with single span and single story of RC frame with masonry infill tested under static cyclic loading with several types of masonry bricks. For more details of specimens refer to [4]. As shown in Fig. 3, the shear strength of masonry infill, τ_{inf}, generally ranges between 0.2N/mm^2~1N/mm^2. The shear strength, τ_{inf} commonly ranges $0.04f_m$~$0.1f_m$.

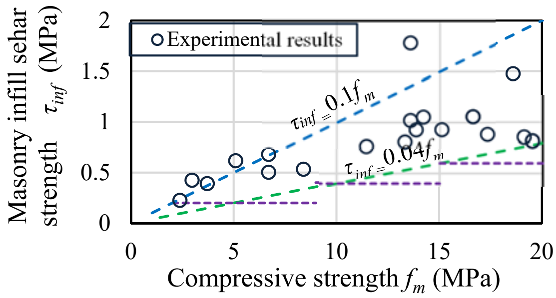

Fig.3 Relation of prism compressive strength to shear strength of masonry infill

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

$$\beta = Vf \,/\, V\,\mathrm{inf} \qquad (4)$$

Where V_f is the lateral strength of boundary frame which is calculated to be the ultimate flexural capacity, assuming plastic hinges at the ends of the column as in bare frame (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 infill, computed by Eq. 5. This is a simple prediction assuming the τ_{inf} as $0.05f_m$. This value gives an average value of experimental to analytical results of 0.83 as shown in the study [4]

$$V_{inf} = 0.05\, f_m \cdot t_{inf} \cdot l_{inf} \qquad (5)$$

Fig. 4 shows the relation between *β-index* (ratio of frame strength to masonry strength) and increase in masonry shear strength, τ_{inf} (Normalized shear strength by dividing by prism compressive strength, f_m).

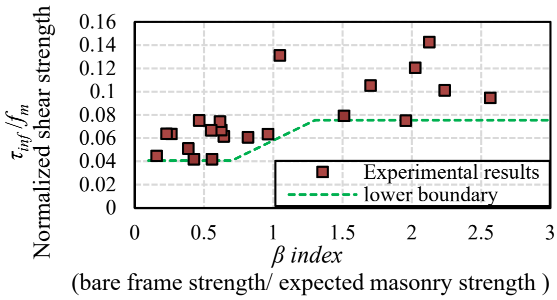

Fig.4 Relation of β- index to shear strength of infill

However, evaluating *β-index* needs some computational effort since it needs reinforcement details of the RC frame. In this paper, the scope of study focuses on simplified 1st level evaluation, thus ignoring the influence of *β-index* and taking the lower boundary is considered here. A more detailed evaluation method considering the *β-index* is proposed elsewhere [4].

In addition, acquiring the data of the prism compressive strength of masonry infill in the site, might be difficult during 1st level evaluation process. Therefore, for simplicity in the absence of material test results from the field, the shear strength of masonry infill in the first level method is proposed to be taken based on the lower boundary of expected masonry compressive strength as shown Table 1 and Fig. 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 discuss the results related to deformation limits for masonry infill based on database of experimental studies, that are summarized in [4]. 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 1.71% and standard deviation of 0.77%.

There are several parameters influencing the deformation limits of masonry infill such as brick types, mortar strength and relative strength of surrounding frame. One of the most influencing parameter is *β index* (indicating the relative strength of frame to masonry infill strength as shown in Eq. 4). Fig. 5 shows the relation between *β index* and R_u. Increasing *β index* (relatively stronger frame) will improve the ductility and for β <0.5 (relatively weak frame) have less ductility. Such a relation between deformation limits and β index was also noted in other seismic evaluation standards such as ASCE/SEI 41 [5].

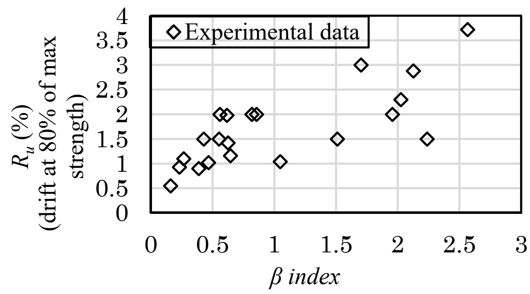

Fig.5 Relation of between β index and R_u drift

In addition, experimental results showed that specimen with low value of β <0.5 (relatively strong infill and weak frame), commonly had brittle and sudden drop of strength just after reaching its maximum strength as shown in Fig.6 showing specimen F-0.4 in [4].

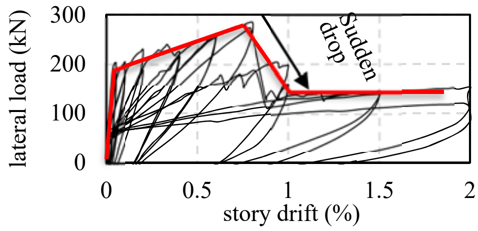

Fig. 6 Specimen with β <0.5 (weak frame) [4]

The Japanese standard for 1st level, from conservative point of view, ignores ductility index of elements such as columns with ductility values larger than F=1. 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 1st level evaluation, the F index for masonry infill is taken as 1 and the influence of β index is ignored and considered as conservative assumption. A more detailed evaluation for masonry infill for 2nd level is discussed in another study [4].

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 open data website named Datacenterhub [6, 7, 8]. The data came from field surveys of RC buildings damaged by the earthquakes as a reconnaissance effort by several organizations. The data of buildings contains location of buildings, simple sketch of plan for each building showing cross-sectional areas of columns and masonry infill and photos of damage.

The classification of damage level of buildings used 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 of buildings does not have detailed drawings and 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 also indicated the standard [1]. In this study, an average of 1N/mm^2 is taken for the case of unavailable data.
b) The effective strength factor used for columns, α, is taken as 0.7, (see Fig.2), that is based on study by [9].
c) The average weight per unit area for RC buildings is taken as11kN/m^2 in the absence of data.
d) The strength contribution of masonry infill walls with large opening (greater than 40%) is ignored. The A_{inf} is calculated by deducting cross-sectional area of the openings. However, the location of opening is also important, for more details refer to study [4].
e) S_D and T which are reduction factors for structural irregularity and deterioration after construction respectively, (see Eq.1) are taken as 1 for simplicity.
f) Masonry infill are 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 Feb 06, 2016. The earthquake caused large-scale damage in Tainan city, shown in. Fig.7, which is about 40 km from the epicenter. The data of damaged existing RC buildings is provided by Datacenterhub [6]. 65 RC buildings with masonry infill are investigated in this study and shown in Fig. 7. The number of stories ranged between 1~5 stories, with the majority of buildings between 2~3 stories. Further details of the investigated buildings are stated in [4].

Several ground motion stations in Tainan city which recorded values of PGA between 0.2~0.4g, and the maximum recorded PGA was around 0.45g in station CHY 62. The response acceleration plots using 5% damping and recorded from stations near the investigated buildings are shown in Fig. 8. Most of response spectra from stations near the buildings have a are less than 0.8 g for short periods (less than 0.5 seconds), except for station CHY 62.

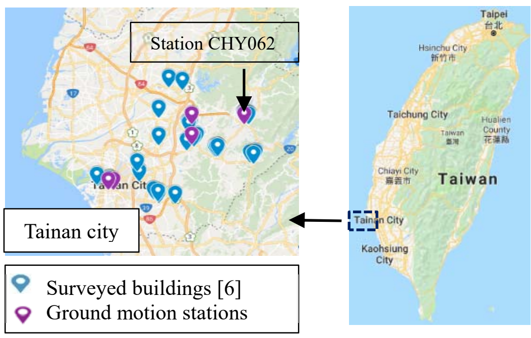

Fig. 7 Locations of the investigated buildings

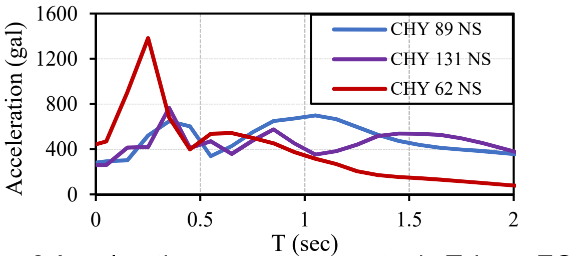

Fig. 8 Acceleration response spectra in Taiwan EQ.

Masonry infill walls in Taiwan were commonly made of clay red bricks and had thicknesses in the range of 200 mm~ 300mm. The type and strength of masonry infill was not stated in the databases, but as stated in several studies such as [10], the expected shear strength of 0.4 N/mm,^{2} is used for here for seismic evaluation.

4.1.2 Seismic capacity results

Fig. 9 shows the relation of I_s index for both NS and EW directions for 1st story of the investigated buildings to damage levels. Fig. 10 show the relation of I_s index with distribution of number of buildings. There is a clear relation between damage level and decrease of I_s index. In Japan, the I_{so}-index (demand criteria) is set as 0.8 and 0.6 for 1st and 2nd level, respectively.

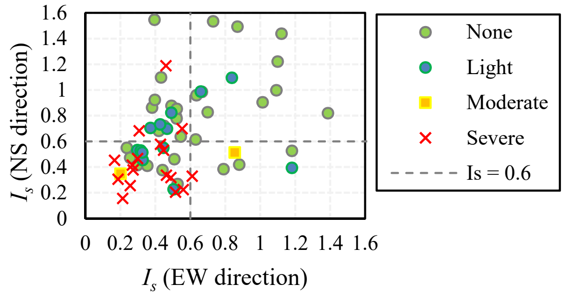

Fig. 9 Relation of I_s index and damage in Taiwan EQ

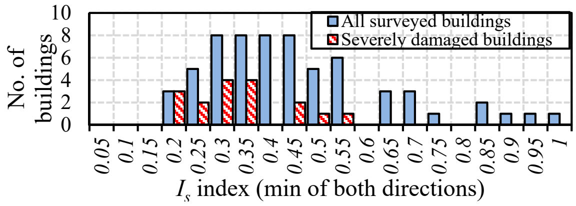

Fig. 10 Distribution of I_s index in Taiwan EQ

Table 2 shows the average and standard deviation of the I_s index for the investigated buildings. Fig. 11 shows the log normal distribution. It should be noted that the curve for severely damaged building in Fig. 11 is adjusted by multiplied by the ratio of severely damage buildings (17/65 severely damaged/ all surveyed buildings). I_s index of 0.5~0.6 would be sufficient to avoid the severe damage. However, the problem is that majority of the buildings, as shown in Fig.11, have seismic capacity of less than 0.6 and retrofitting such large stock of buildings would be of a high cost.

Table.2 Average I_s index 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

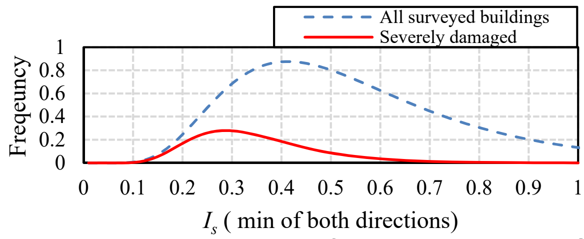

Fig.11 Normal distribution of I_s index in Taiwan EQ

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 16th, 2016. The data consists of 171 low-rise RC buildings with masonry infill in the cities of Manta, Portoviejo, Chone, and Bahía de Caráquez, which are located in the province of Manabí [7], shown in Fig 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 [11], it was found that solid clay brick had unit compressive strengths between 7.3 and 7.9 MPa and most concrete block units between 1.0 ~1.5 MPa. Thus, masonry infill shear strength was taken as 0.2 N/mm^2 using the lower boundary for shear strength in Table 1.

Ecuador acceleration response spectra using 5% damping are shown in Fig. 13. The acceleration record comes from ground motion AMNT and it is the nearest station to the surveyed buildings in Manta city. The response acceleration for short periods (less than 0.5 seconds) for NS and EW directions just exceeded 1g.

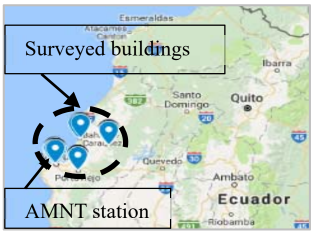

Fig. 12 Locations of the investigated buildings

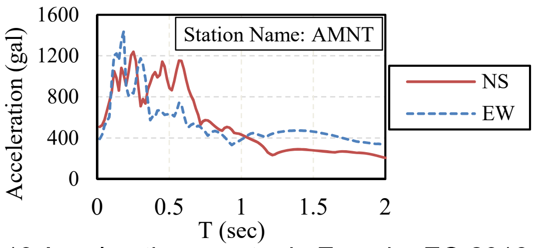

Fig. 13 Acceleration spectra in Ecuador EQ 2016.

4.2.2 Seismic capacity results

Fig. 14 shows the relation of I_s index for both NS and EW directions for 1st story of the investigated buildings with damage levels. Fig. 15 shows the relation of I_s index with distribution of number of buildings.

Table 3 shows the average and standard deviation of the investigated buildings. The average of I_s index for the investigated buildings in Ecuador is 0.316, which is slightly lower than Taiwan. Fig.16 shows the logarithmic normal distribution of I_s index. Similar to Taiwan`s earthquake, buildings with seismic capacity I_s greater than 0.5 avoided severe damage. However, most of the buildings lays below this range as shown in Fig.16.

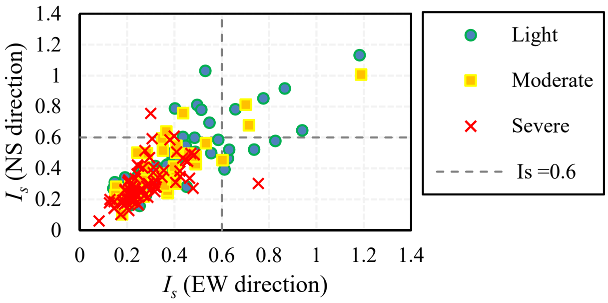

Fig. 14 Relation of I_s index to damage in Ecuador EQ

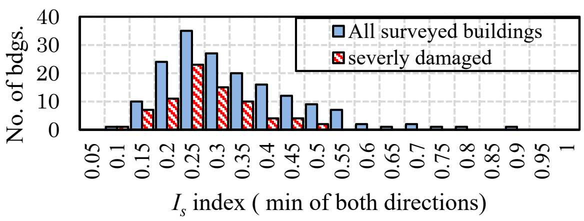

Fig. 15 Distribution of I_s index in Ecuador EQ

Table.3 Average I_s 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

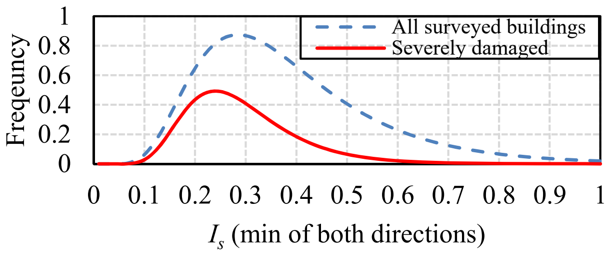

Fig. 16 Normal distribution of I_s index in Ecuador

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 25 April 2015. The data consists of surveys of 134 RC buildings with masonry infill surveyed in the capital city Kathmandu [8] and shown in Fig.17.

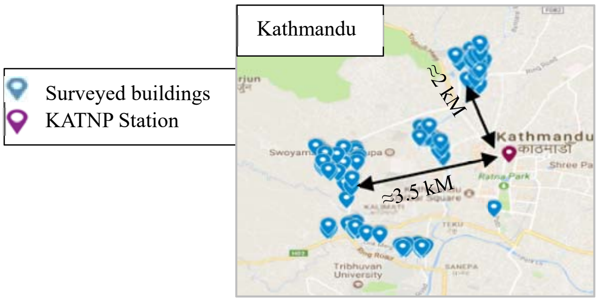

Fig. 17 Locations of the investigated buildings

Nepal response spectra using 5% damping are shown in Fig. 18. The response acceleration spectra at for short periods for NS and EW direction is about 0.3g and 0.6g, respectively. The ground motion station is relatively near the investigated buildings (see Fig.17).

The thicknesses of the masonry infill walls were commonly 220 mm~ 110 mm. The type and strength of masonry infill was not stated in the database, but it is noticed from the photos from the survey that solid burnt clay bricks was the most common. The prism compressive strength of masonry infill in the same region was investigated in other studies by [12], where the compressive masonry prisms was about 4.1 MPa. Therefore, the masonry shear strength is taken as 0.2N/mm^2 as proposed previously in Table 1.

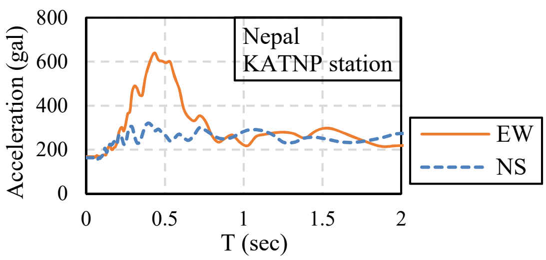

Fig. 18 Acceleration response spectra in Nepal EQ.

4.3.2 Seismic capacity results

Fig. 19 shows the relation of I_s index for both NS and EW directions for 1st story of the investigated buildings to damage levels. Fig. 20 show the relation of I_s index to distribution of number of buildings. Table 4 shows the average and standard deviation of the investigated buildings. Fig. 21 shows the log-normal distribution of I_s index. The average I_s for buildings in Nepal is lower than buildings in Taiwan and Ecuador.

Similar to the Ecuador and Taiwan, the buildings in Nepal with about I_s of 0.5 in Nepal avoided severe damage. It should also be noted that the earthquake response acceleration in Nepal is much smaller than Taiwan Earthquake and Ecuador as shown in Fig.18. Thus, the I_s demand in Nepal of about 0.5 is relatively high compared to its earthquake level and this point needs further investigation.

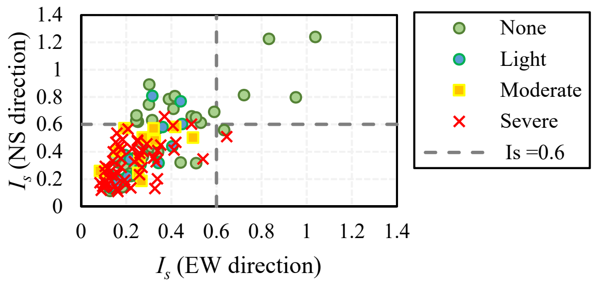

Fig. 19 Relation of I_s index and damage in Nepal EQ

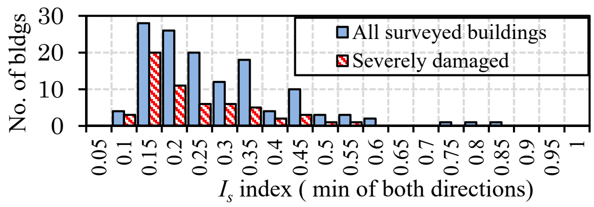

Fig. 20 Distribution of I_s index in Nepal EQ

Table.4 Average I_s index 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

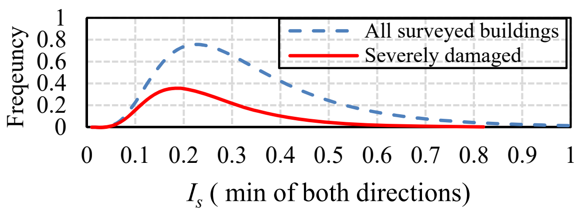

Fig. 21 Normal distribution of I_s index in Nepal EQ

5. CONCLUSION

This paper presented a simplified method to assess the seismic capacity I_s index (1st level) of masonry infilled RC buildings in several countries and the following are the main conclusions:

1- There is a clear relation between the decrease of I_s index and damage level in the investigated buildings in the three past earthquakes, which are: Nepal EQ 2015, Ecuador EQ 2016 and Taiwan EQ 2016.

2- Severe damage is concentrated in buildings with I_s of less than 0.3. On the other hand, buildings with 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.

3- The problem is that majority of the investigated buildings have seismic capacity (I_s index) of less than 0.6. Retrofitting such large stock of buildings would be of a high economical cost and is difficult to apply in the recent future in developing countries.

It should be noted that several assumptions are taken for calculations as noted before, such as in of plane failure precedes the out of plane failure based on observed damage in [6,7,8]. However, this is not a general rule, and should be taken with precautions especially with masonry infill with gaps around the frames, large openings and masonry walls without bounding frame.

In addition, the shear strength of RC column is taken at 1 N/mm^2. This is then multiplied by α index taken as 0.7 in Eq. 2, which indicates that the actual shear strength of columns is taken as 0.7N/mm^2. This assumption need further investigation for improvement of the accuracy of the obtained results.

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