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SEISMIC VULNERABILITY ASSESSMENT METHOD OF LOW-RISE RC BUILDINGS WITH MASONRY INFILL WALLS

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Abstract: A practical seismic evaluation method that is more effective especially in developing countries is proposed. The proposed method is a recalibration of Shiga figure (Shiga, et al) to be applicable to RC buildings with masonry infill using mathematical models analysis. The damage evaluated by the proposed method is applied to the fragility curves of HAZUS and probability of damage is studied. Average structural damage and economic loss of a whole city could be easily estimated in a short time based on the probability of damage of each zone in the proposed method. Finally, the proposed method is then compared to actual earthquake damage which showed quite good agreement.

1. INTRODUCTION

RC buildings with masonry infill are a common practice in many developing countries. Poor performance of RC buildings with masonry infill was noticed in many earthquakes, recently in China 2008 Wenchuan earthquake and Haiti earthquake 2010; see Figure 1.The performance of these buildings could be improved by special detailing of frames, strengthen walls by reinforcement and other retrofitting techniques. However, the problem is that existing buildings makes a huge considerable number therefore seismic evaluation method is necessary to screen vulnerable buildings.

Figure 1 Damage of Building with masonry infill in China 2008 Wenchuan earthquake

In developing countries, the main issue is usually economic cost. Professional engineers and the time needed to seismically evaluate a whole city will be of a high cost. In addition, if almost all buildings are considered unsafe then the possible action of the government and people is no action because building a new city seems much easier. Therefore a proposed method should filter buildings into categories according to their vulnerability level. In other words, buildings with higher possibility of collapse and severe danger are filtered and have higher priority to be retrofitted. From the above points it is concluded that a simple, low cost and fast seismic screening method is needed as a first screening method.

The original concept of the method proposed in this study is not new and was introduced first by Shiga (Shiga 1968) for the Japanese buildings. The Shiga map screens the buildings into zones with different vulnerability levels according to their column sectional area, wall sectional area and floor area. This method was based on actual data of damaged buildings from Tokachi-Oki earthquake in 1968. A similar method was presented by Hasan and Sozen (Hasan 1997) using the damage data from Erzincan earthquake in 1992 in Turkey. Certainly there isn't better than a detailed analysis of each building and judgment of professional engineers. However, issues such economic costs and the time needed that were mentioned before makes these methods practical for preliminary screening of vulnerable building. The problem is that this method is only applicable to its region because of different seismicity level, material properties and structural details. Actual earthquake damage data is needed to construct Shiga map. As for countries with infrequent earthquakes (return period of 50~100 years), damage data is usually unavailable. Waiting for an earthquake to construct such method is not an option, 2010 Haiti disaster is a recent example. This paper presents a recalibration of Shiga`s method to be suitable to RC buildings with infill walls in different seismic regions. To make this study more realistic, a case study of RC buildings with masonry infill walls in Jordan is considered.

2. PROPOSAL OF SEISMIC EVALUATION METHOD

2.1 Method Flowchart:

The main Flowchart of the proposed method is shown in Fig. 9 Jordan will be used as an example for the proposed method

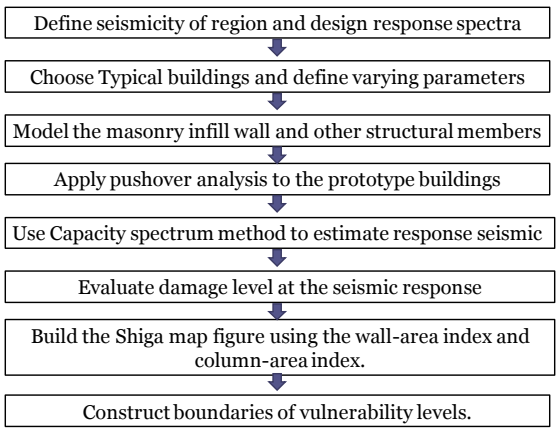


Figure 2 Flowchart of the proposed method

2.2 Define seismicity of Jordan

Jordan is a Middle East country located along the seismically active Dead Sea Transform Fault that extends 1000 km from the Red Sea to Turkey. The seismicity in Jordan is thought to be as moderate. Seismic hazard of Jordan with maximum PGA of about ~0.25g near the Dead Sea is shown in figure 3 (Earthquake hazard report 2007). Current estimates predict a major earthquake in the region roughly every 200 years. During the past ten years, an earthquake of magnitude Mw = 5.1 occurred in the Dead Sea basin on February 11th, 2004 and followed by an Mw = 4.7 earthquake on July 7th, 2004.

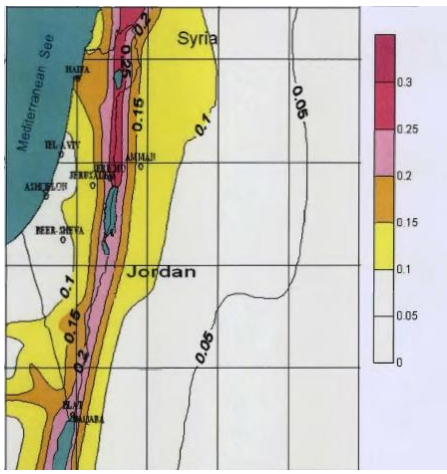


Figure 3. Hazard map with probability of occurrence of 10% in 50 years (Earthquake hazard report 2007)

Different regions have different seismicity levels in Jordan. UBC code (UBC 1997) is used to construct the design response spectrum for two soil types for PGA=0.2g, soil Type SB (rock) and soil Type Sc (very dense soil and soft rock), shown in Fig. 4. In this study, the response spectra of Sc Soil type were used.

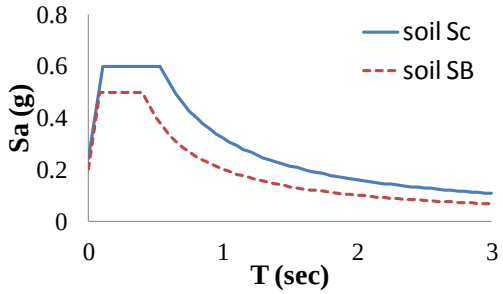


Figure 4 Design response spectra for two soil types Jordan

2.3 Typical buildings in Jordan

RC structures are widely used in Jordan. Typical building plan is shown in Fig. 5. The exterior infill walls are composed usually of 3 layers; stone facing, plain concrete and hollow concrete blocks as shown in Fig. 6. The exterior walls are of thickness ranging from 300~350 mm. These walls are bounded by slender RC columns. The interior columns are usually more reinforced than exterior columns. Exterior masonry walls are sometimes allowed to work as a bearing wall for buildings less than 12 m in height. This resulted in a large number of low rise buildings with masonry infill used usually as residential and commercial buildings in the main cities. This practice is not based on a structural analysis; it is based on past experiences and practices in surrounding countries. As for the partition walls inside the building, hollow concrete blocks of thickness ≈100mm is used. This partition walls are placed randomly and might not be bounded by any columns. The influences of the partition walls are ignored in this paper, only their weight were included. Slabs are ribbed slab (One-way joists) of thickness of 250mm~300mm. Typical beam and column sizes and reinforcement are shown in figure 7. RC walls are commonly used around stair cases and elevators in recently designed buildings (after 2000), see Fig. 4. These RC walls are usually placed in one direction which is mainly the transverse direction.

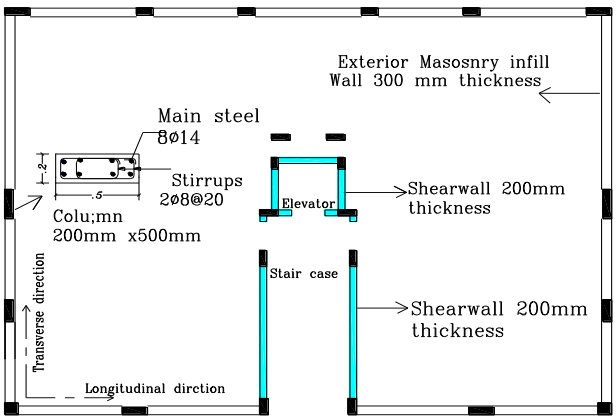


Figure 5 Typical plan for existing building

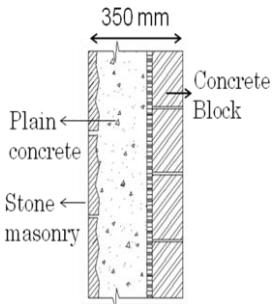


Figure 6 Masonry infill wall in exterior frames

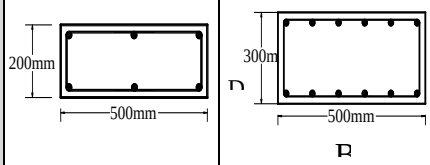
	Typical Column	Typical Beam
Dimension		
Main Bars.	6D16	Top&Bottom 12D16
Hoop	Ø8@200mm	Ø10@200mm

Figure 7. Typical column and beam

2.4 Prototype buildings characteristics

3 prototype buildings with different floor areas were used for analysis. Figure 8 shows the structural plan of prototype building B. Analysis was done for the X- direction (direction with no RC walls in the staircase) which is usually the weakest direction. Many cases were assumed for each building, assuming number of stories ranging from 1 till 4 stories and varying the places and number of masonry infill walls. Since exterior facings of buildings have usually many opening, exterior infill walls with opening of $1m^2$ in each wall and thickness 300 mm were assumed, see Fig. 9. The interior partition walls were ignored in this paper, only their weight was included. Exterior columns (bounded by infill walls) and Interior columns are assumed to have dimensions of 200mm x 500mm and 300mm x 500mm respectively

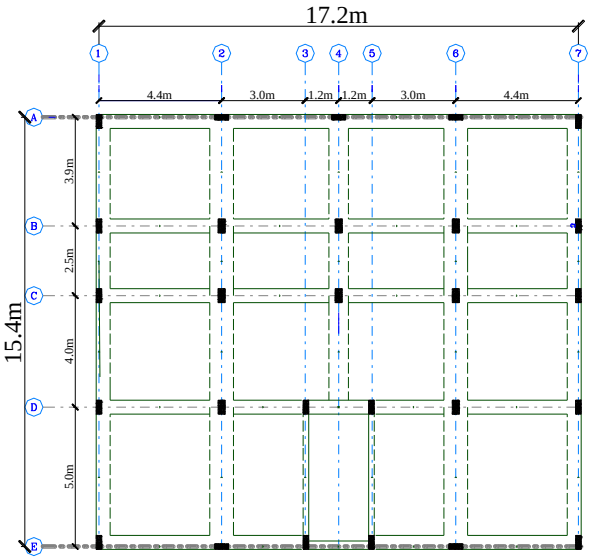


Figure 8 structural plan of prototype building B

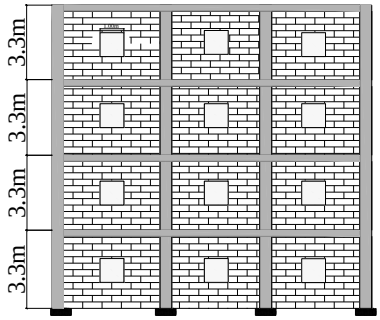


Figure 9 Typical elevation plan of exterior frame

2.5 Modeling of structure

In this study a simple two-dimensional pushover analysis using computer program SNAP is used. Infill walls are idealized as diagonal strut, beams and columns are idealized by two nonlinear rotational springs at their ends, a nonlinear shear spring in the middle and a linear axial spring, see Figure 10 and Figure 11. A tri-linear relation is used for rotational and shear springs, see Figure 12 and Figure 13. The contribution of slab to the beams flexural strength was ignored. The beam-column connection is assumed to be rigid. The distribution of lateral forces along the height in the pushover analysis is based on the Ai distribution prescribed in the AIJ provision (AIJ 1999). The cracking and yield moment of rotational spring and shear spring of columns and beams are estimated using AIJ standard (AIJ 1999). The infill wall is modeled as an equivalent diagonal compression strut with width, W_{ef}, calculated using Eq(1) and Eq(2) which are based on recommendations given in FEMA 356 (FEMA 2000). The equivalent struts have the same thickness and modulus of elasticity as the infill wall it represent.

$$W_{ef} = 0.175(\lambda_h H)^{-0.4}\sqrt{H^2 + L^2} \qquad (1)$$

$$\lambda_h = \left[\frac{E_w t_w \sin(2\theta)}{4E_c I_c H_{\inf}}\right]^{\frac{1}{4}} \qquad (2)$$

Where: E_w and E_c are the moduli of elasticity of the infill wall and the concrete .H_{inf}, H and L are the net height of the infill wall, the storey height, and the bay length of the frame. θ = arctan(H/L) (the inclination of the diagonal). t_w = is the thickness of the infill wall, I_c = moment of inertia of the column of the frame,

The axial strength of the equivalent strut of the infill panel is determined by dividing the expected infill shear strength, V_{ine} , by Cosθ (θ= arctan(H/L). The expected infill shear strength (V_{ine}) is determined by Eq(3) and Eq(4) using the methodology given in Section 7.5.2.2 in FEMA 356 (FEMA 2000) :

$$V_{ine} = V_{me}.An \qquad (3)$$

$$Vme = \frac{0.75\left(Vte + \frac{P}{An}\right)}{1.5} \qquad (4)$$

Where; V_{me} :bed joint shear strength, P : gravity load, An : area of net mortared of infill wall and V_{te} : Average bed-joint shear strength. Due to lack of research on the wall infill properties used Jordan, bed joint shear strength, V_{te}, is assumed 0.7 N/mm^2 and Elasticity of infill wall, E_W, of 9500N/mm^2. These values could be easily acquired in future research. To account for the influence of openings in the walls, initial stiffness and maximum force were reduced using the factor $\lambda_{opening}$ calculated using Eq(5) based on the work of Dawe and Seah (Dawe1988).

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$$\lambda_{opening} = 1 - \frac{1.5L_{opening}}{L\inf}; \lambda_{opening} \geq 0 \qquad (5)$$

Figure 14 shows the backbone curve of infill wall. The d value shown in the Figure 14 is taken using Table 7-9 in FEMA356 (FEMA 2000) for assumed ratio of frame to infill strengths ≤ 0.7 (the frame is assumed to have small strength compared to the infill panel).

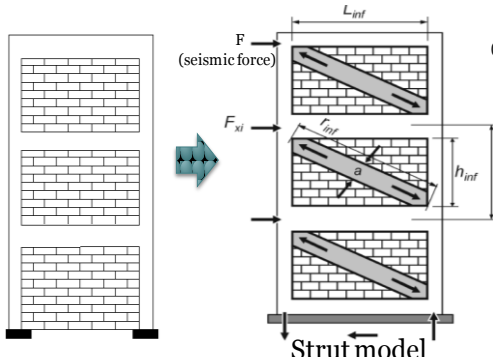

Figure10 Strut model for masonry infill walls

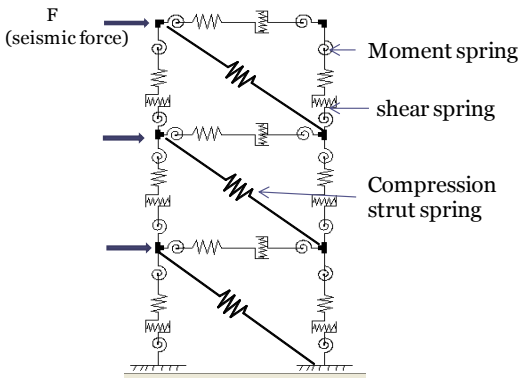

Figure 11 Model of Pushover analysis

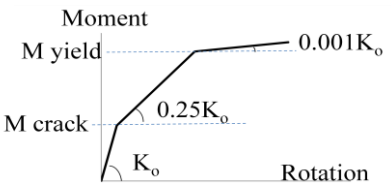

Figure 12 Rotational spring backbone curve

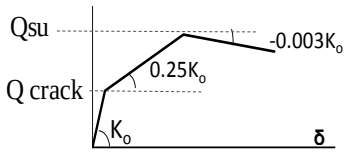

Figure 13 Shear spring backbone curve

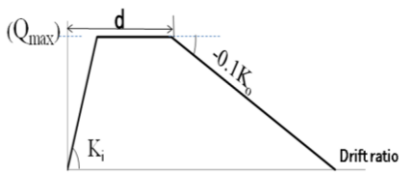

Figure 14 Strut spring back bone curve

2.6 Seismic response and damage evaluation:

The seismic response of the structure is calculated using methodology for calculating target displacement given in Section 3.3.3.3.2 in FEMA 356(FEMA 2000).

The damage evaluation is judged by approximating the amount and level of damage in the infill walls and columns in the first and second story of each building at the seismic response displacement given by the pushover analysis. The damage states employed are similar to those proposed in HAZUS99 (FEMA 1999) with some modification for the severe damage in which total and partial collapse is added. Where *Slight Structural Damage*: Diagonal (sometimes horizontal) hairline cracks on most infill walls; cracks at frame-infill interfaces. *Moderate Structural Damage*: Most infill wall surfaces exhibit larger diagonal or horizontal cracks; some walls exhibit crushing of brick around beam-column connections. Diagonal shear cracks may be observed in concrete beams or columns. *Extensive Structural Damage* (named as *severe damage* in this paper): Most infill walls exhibit large cracks; some bricks may dislodge and fall; some infill walls may bulge out-of-plane; few walls may fall partially or fully; few concrete columns or beams may fail in shear resulting in partial collapse and in imminent danger of collapse. Structure may exhibit permanent lateral deformation.

However, it is difficult to state when exactly the masonry infill wall could be called moderately damage or severely damaged in the backbone curve of figure 14. In figure 15 a suggested figure for damage evaluation is proposed. The elastic region till about 0.8 Q_{max} is considered with none damage, from 0.8Qmax till about half of C value is considered having slight damage. Severe damage starts from the point where the wall drops to 80% of its maximum shear strength.

The d value shown in figure 14 in many prototype buildings were calculated to be about 0.4% of inter-story drift using Table 7-9 in FEMA356. The moderate damage in many cases studied in this paper starts at drift of about 0.25%. This values quite well agrees with many experiments carried by (Merhabi et al 1996) were the first major crack occurred at drift between 0.17%~0.36% for strong infill and weak frame specimens.

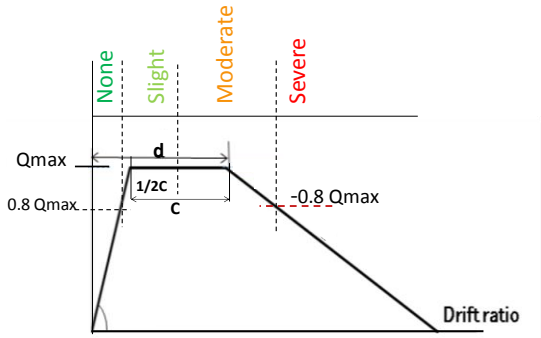

Figure 15 Proposed damage states of masonry infill

When more than half of the masonry infill walls strength degrades to 80% of its shear strength the buildings is considered severe. This might be quite conservative for buildings having strong frames which could carry the seismic demand even after the masonry walls are crushed. However, it is thought to be appropriate in the case study of Jordan where the masonry infill walls support part of the gravity load and the surrounding RC frame is lightly reinforced and weak, therefore the degradation and crushing of masonry infill walls could cause instability of structure and imminent danger of partial or total collapse.

To make the method clear, the proposed method is illustrated by example using one of the cases of the prototype buildings shown in figure 8:

This case is of a four storied structure having masonry infill walls in all its exterior axes. Axes A and E (see figure 8) have masonry walls with openings of 1m^2, columns and beams are modeled as stated previously. Axes B, C and D have no infill and modeled as moment frames. A displacement-controlled pushover analysis is applied to the longitudinal direction. The bilinear idealization of the capacity curve represented by base-shear and roof displacement is shown in figure 16. The total weight of the building was calculated to be 12900 kN with average weight per unit area of 12.5 kN/m^2.

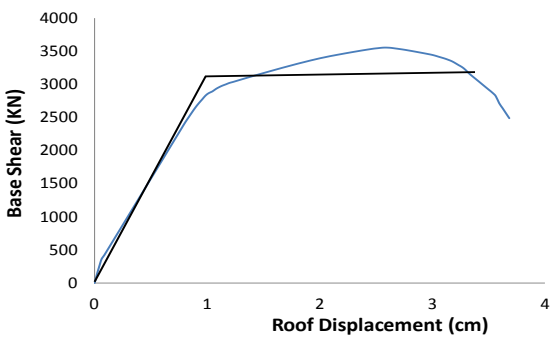

Figure 16 Bilinear idealization of the capacity curve

The performance point was calculated using the target displacement method for nonlinear analysis to be of 3.28cm of roof displacement. At this performance point more than 90% of the number of walls in the 1st floor has degraded to less than 80% of their maximum shear strength. Force-Displacement curve of strut spring of one of the walls in 1st floor is shown in figure 17. In additional, plastic hinges were formed on both ends of few columns of 1st floor. As for the 2nd floor all walls has reached the moderate region proposed in figure 15. Upper floors are still in their slight region. Since most of the infill walls of 1st story are evaluated as severe at performance point, this building is considered as severe.

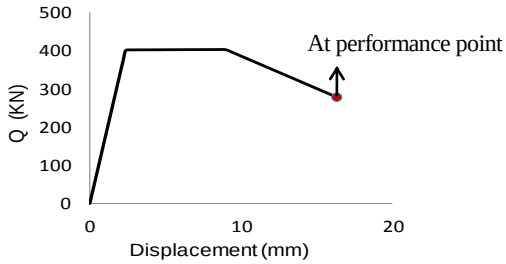

Figure 17 Force-Displacement curve of one of the masonry infill walls of 1st floor in the illustrated example

The Total area of floors, total cross sectional area of columns of 1st floor and total cross sectional area of masonry infill walls of this case (cross sectional area of openings in walls are deducted) of 1st floor are 1059.5m^2, 3.1m^2 and 6.63 m^2 respectively. The Column index, CI, which is percentage ratio of the cross sectional area of columns in 1st floor to the total floors area, is 0.29. The Wall index, WI_{inf} which is the percentage ratio of the cross sectional area of infill walls in 1st floor to the total floors area is 0.63.

3. Results:

Properties and results of the prototype buildings are shown in Table 1. The Column index, CI, is percentage ratio of the cross sectional area of columns of 1st story to the total floors area of the prototype buildings. The Wall index, WI_{inf} is the percentage ratio of the cross sectional area of infill walls (the length of the openings in infill walls are deducted from the total infill length) of the total of 1st floor to the total floors area of the prototype buildings.

The Column index, Wall index (WI) and damage expected of each building are shown in Figure 18. Fig. 18 is based on the concept of Shiga map. Hasan and Sozen (Hasan1997) used a similar figure for actual damage of buildings in Erzincan earthquake. The figure 18 is divided into vulnerability zones by the lower boundary line and upper boundary line. There are 3 Vulnerability Zones, for which zone A (danger zone) is the most vulnerable and zone C (safe zone) is the least. The boundary lines here are assumed based on the damage evaluated. However, the boundary lines assumed here could be adjusted based on the probability of expected damage and acceptable seismic hazard which are discussed in next section (section of probability of damage)

Thinking of results as capacity provided by the building versus seismic demand by earthquake, then capacity is thought to be provided by columns and infill walls strength, which is product of the shear stress and cross sectional area of columns and walls shown of the left side of Eq (6). The

Table 1 Properties and results of the prototype buildings

Building No.	Total Column area m^2	Total Infill Wall area m^2	Total Area m^2	Column index%	Infill Wall index %	Evaluated Damage level
Building A- 1 stories (case 2)	3.8	11.04	438	0.87	2.52	none
Building A- 1 stories (case 3)	3.8	4.44	438	0.87	1.01	slight
Building A- 2 stories (case2)	3.8	11.04	876	0.43	1.26	slight
Building A -2 stories (case 1)	3.8	13.92	876	0.43	1.59	Slight
Building A- 2 stories (case 3)	3.8	4.44	876	0.43	0.51	moderate
Building A- 3 stories (case 1)	3.8	13.92	1314	0.29	1.06	moderate
Building A- 3 stories (case 2)	3.8	11.04	1314	0.29	0.84	moderate
Building A- 4 stories (case 1)	3.8	13.92	1752	0.22	0.79	severe
Building A- 4 stories (case 2)	3.8	11.04	1752	0.22	0.63	severe
Building A- 4 stories (case 3)	3.8	4.44	1752	0.22	0.25	severe
Building B- 1 stories (case 1)	3.1	6.63	271.44	1.14	2.44	none
Building B- 1 stories (case 2)	3.1	3.72	271.44	1.14	1.37	slight
Building B- 2 stories (case 1)	3.1	6.63	542.88	0.57	1.22	slight
Building B- 2 stories (case 2)	3.1	3.72	542.88	0.57	0.69	moderate
Building B- 3 stories (case 1)	3.1	6.63	794.64	0.39	0.83	moderate
Building B- 3 stories (case 2)	3.1	3.72	794.64	0.39	0.47	severe
Building B- 4 stories (case 1)	3.1	6.63	1059.52	0.29	0.63	severe
Building B- 4 stories (case 2)	3.1	3.72	1059.52	0.29	0.35	severe
Building C- 1 stories (case 2)	1.48	3.48	101.2	1.46	3.44	none
Building C- 2 stories (case 1)	1.48	3.48	202.4	0.73	1.72	slight
Building C- 3 stories (case 2)	1.48	3.48	303.6	0.49	1.15	moderate
Building C- 4 stories (case 2)	1.48	3.48	404.8	0.37	0.86	moderate

demand needed is the product of the total weight of the building and response acceleration coefficient of earthquake (marked as C_a) and reduction factor (marked as D_s) to account for the ductility which the building posses.

Capacity ≥ Demand

$$\tau_c . A_C + \tau_w . A_W \geq W. C_a . D_s \qquad (6)$$

Where A_c: is the area of columns(mm^2), τ_c: is the shear strength of columns (N/mm^2), A_w: is the area of masonry infill (mm^2), τ_w: is the shear strength of masonry infill walls (N/mm^2) , W: is total weight of building, Ca: response acceleration coefficient of earthquake.

Total weight of the buildings (W) is the product of total area of floors (A_f) and average weight per unit area. The average weight per unit ranged for the studied cases between 12 kN/m^2~14 kN/m^2. Taking 13 kN/m2 as the average weight per unit area and assuming reduction factor Ds = 1 Therefore;

$$\tau_c . A_C + \tau_w . A_W \geq 0.013 N/mm^2 . A_f . C_a \qquad (6)$$

The response acceleration coefficient for Soil C spectra used in this case study for buildings with short periods (low-rise buildings) is 0.6g. Dividing both sides by A_f (area of floor), Therefore:

$$\tau_c . \frac{A_C}{A_f} + \tau_w . \frac{A_w}{A_f} \geq 0.0078 N/mm^2 \qquad (7)$$

Eq(7) is expressed in terms of column index (CI percentage ratio of column area to total floor area) and Wall index (WI_{inf}: percentage ratio of infill wall area to total floor area) and shown in Eq (8):

$$\tau_c . CI + \tau_w . WI_{inf} \geq 0.78 N/mm^2 \qquad (8)$$

The column shear strength τ_c and Wall shear strength τ_w of lower boundary shown in figure 18, are $1N/mm^2$ and $0.6N/mm^2$, respectively

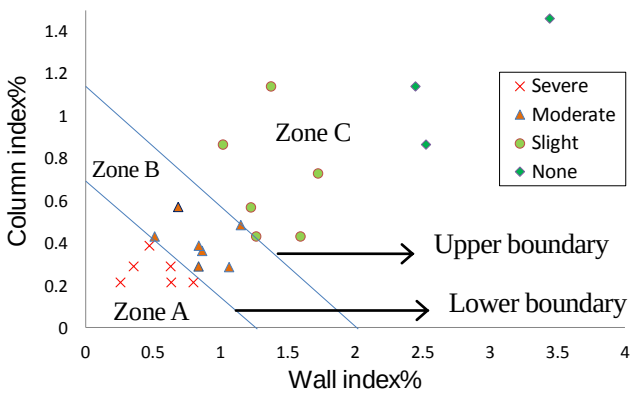

Figure 18 Proposed evaluation map

To make this method more practical; Eq (9) and Eq (10) are proposed. The inventory buildings having CI and WI_{inf} in index *less than 0.78N/mm²* are more vulnerable and therefore have higher priority to undergo a detailed analysis and seismic retrofit. Buildings having CI and WI_{inf} index *more than 0.78* N/mm^2 in Eq (10) have lower priority to be

checked. This boundary lines could be changed based on probability of damage and resources available for the seismic evaluation discussed later.

$$1.0\ CI+0.6\ WI_{inf} \geq 0.78\ N/mm^2 \quad (9)$$

$$0.65\ CI+0.37\ WI_{inf} \geq 0.78\ N/mm^2 \quad (10)$$

It should be noted here that Jordan has regions with different seismicity levels and soil condition. Therefore, different regions have different vulnerability zones and boundaries.
It is suggested that infilled frames with openings exceeding 50 percent of the panel area should be ignored.

4. Damage Probability:

Buildings evaluated to be in zone A (danger zone) does not imply that all buildings would be severely damage but it means that buildings in zone A are more probable to have severe damage. Vice versa is also true, a building in zone C (safe zone) does not imply that it would not be damaged but means that it less likely to be severely damaged.
Probability of damage could be checked using varies of methods; one of the most widely used is fragility curves. Fragility curves for the case of buildings in Jordan are still in its early stages of research. In this paper, HAZUS fragility curves are used to estimate damage probability.

4.1 Introduction of HAZUS fragility curves

HAZUS is a project conducted for the National Institute of Building Science (NIBS), under a cooperative agreement with the Federal Emergency Management Agency (FEMA). HAZUS is an estimation methodology for loss estimates from hazards and economical damage for structures and infra-structures in the United States. The probabilities of structural and non-structural damages to structures and life-lines as well as direct and indirect economical losses induced by natural disasters such as earthquake, flood and storm can be estimated.

In this paper, fragility curves for ground shaking damage for Low-rise RC buildings with masonry infill and pre-code design level is used. Pre-code damage functions was selected since it was recommended by HAZUS for older buildings that were not designed for seismic loads which is similar to the case of Jordan buildings built before seismic code was implemented in 2005. However, the fragility curves of HAZUS are derived for US buildings, thus the applicability of it to buildings in Jordan needs further research.

The fragility curves of HAZUS uses for 4 levels of damage states of structural damage; slight damage, moderate damage, extensive damage (severe damage) and complete damage. The description of slight damage and moderate damage of this paper are the same of that of HAZUS as stated before. The Complete damage state in HAZUS is defined as buildings which have collapsed or have partial collapse. The fragility curves are characterized in terms of spectral displacement as shown in figure 19.

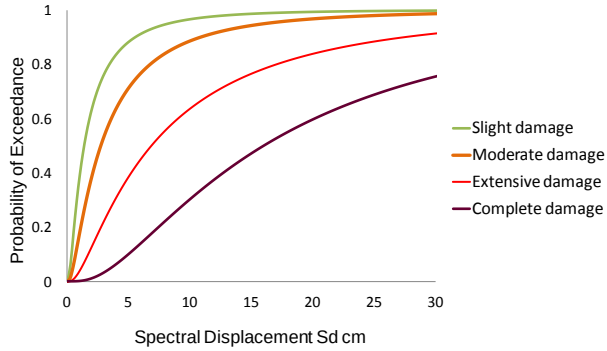

Figure 19 Fragility curves of Pre-code Low-rise RC buildings with unreinforced masonry infill

The performance point of each of the previous prototype buildings are transferred into spectral displacement using of Eq (11)

$$\Delta(Sd) = \frac{\Sigma(m_i \cdot \delta_i^2)}{\Sigma(m_i \cdot \delta_i)} \quad (11)$$

Where m_i is the weight of each floor and δ_i is displacement of each floor at the response point.
Figure 20 shows the damage state probability for the example building mentioned in the previous section which had performance spectral displacement of 2.9cm (3.24 cm roof displacement).

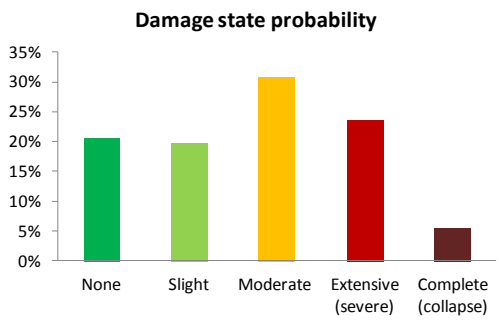

Figure 20 Damage probability of illustrated example

Using the fragility curves the damage state probability is studied for all selected buildings` cases in each zone. The four buildings just below the lower boundary that are in red circle marked as (a) in figure21 had an average probability of severe damage and collapse damage of *25.3% and 6.7%* respectively. This means that if you have about 100 buildings having similar Column index and wall index similar to those in circle(a) then 25 buildings will be heavily damaged and 7 buildings will collapse. Obviously, the weaker the building (smaller column and wall indices) than those in circle (a), then severe damage and collapse probability increases.

As for buildings in the orange circle marked as (b) in figure 21, the average probability of severe damage and collapse are *17.4% and 2.5%* respectively. This means that many buildings in this region can be heavily damaged, but only less than 2.5% will collapse.

As for buildings in the green circle marked as (c) in figure 21, the average probability of severe damage and collapse are *5.2% and 0.2 %* respectively. Buildings in this region are unlikely to collapse or partially collapse.

All the low-rise RC buildings with masonry infill walls in the city could be screened in a short time using only the column and wall indices. Those buildings then are checked are plotted proposed Shiga map for case of Jordan and classified into zones. Based on the percent of number of buildings in each zone then average structural damage and economic loss could be easily estimated. In additional, screening the city using this study helps to estimate roughly the resources needed for buildings` retrofit and risk mitigation.

The lower and upper boundaries proposed here could be modified easily based on the philosophy of the acceptable damage in the risk mitigation project and also based on the available resources for seismic evaluation and retrofit.
Understanding the probability of damage of each zone is an indispensable part of the proposed method

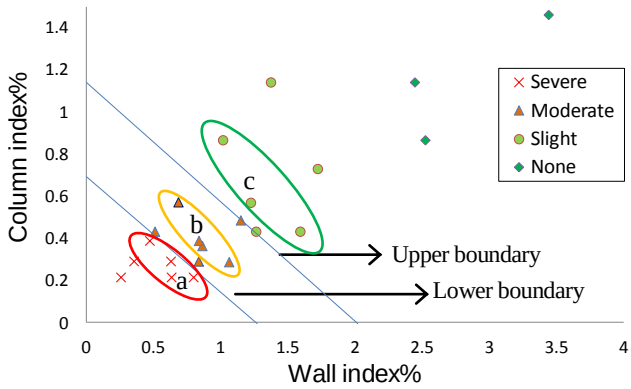

Figure 21 selected buildings for checking damage probability

5. Comparison with earthquake damage:

There is no recent major earthquake in Jordan. The nearest country with earthquake damage data is Turkey. RC buildings with masonry infill walls are commonly used in Turkey. Jordan and Turkey are Middle Eastern countries and they also shares the dead fault which runs through Jordan till the south-eastern part of Turkey (see figure 22). However, similar structure detailing and material quality in both countries is Not checked yet.

Figure 22 Dead Sea fault passing Jordan

5.1 Erzincan Earthquake Turkey; Data and Comparison

Erzincan 1992 earthquake is of magnitude 6.8 and PGA of ≈0.5g. The acceleration time history for NS and EW direction is shown in figure 23 and figure 24. The Architecture institute of Japan, AIJ, investigated a heavily damaged area in Erzincan earthquake (AIJ Report1993). It was reported that out of the 424 buildings investigated 28 (6.6%) collapsed and 68 (16.8%) were heavily damaged.
Erzincan`s response spectra (see figure 25) at short period is NS≈0.8g and EW ≈ 1.2g *which is much larger* than response spectra of 0.6g used in the proposed method especially in EW direction.

The damaged buildings data are collected by METU (Middle East Technical University) and AIJ and were mentioned by Hasan and Sozen (Hasan1997). Data contain also buildings with RC walls; therefore the data are filtered for buildings having only RC columns and Infill walls. The proposed boundaries are compared and plotted with data of damaged buildings by Erzincan earthquake 1992 in Turkey, see fig ure26.

The METU definition of damage state is different from the one used in this paper. Definitions of the damage state in the METU data as follows: Light: Reinforcement exposed but not buckled near joint faces. Fine flexural cracks in structural and nonstructural elements, Moderate: Reinforcement buckled near joint faces and/or inclined cracks in structural walls. Severe: Structural failure of individual elements.

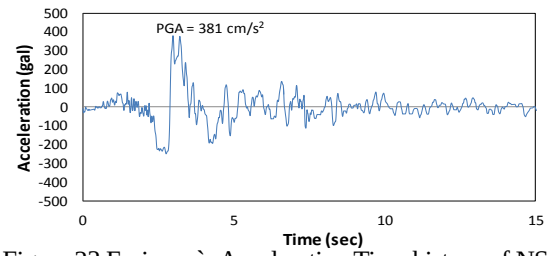

Figure 23 Erzincan`s Acceleration Time history of NS

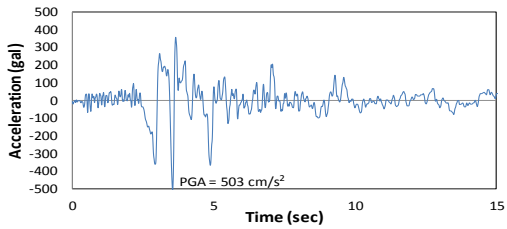

Figure 24 Erzincan`s Acceleration Time history of EW

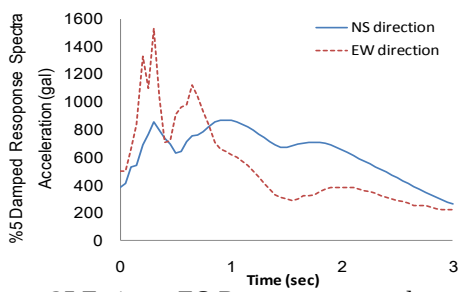

Figure 25 Erzincan EQ Response spectra damped 5%

The boundaries from the proposed buildings showed good agreement with damage of the Erzincan`s buildings. However, the Erincan`s response spectra (about 1g) is much larger than the response spectra in Jordan (about 0.6g). Then why good correlation was observed?

First the METU definition of damage state is different from the one used in this paper. Definition of the damage state of moderate state in METU indicates the reinforcement bars could be seen and they have already buckled. This moderate damage state of METU is thought to be as severe damage states used in this study and therefore the moderate damaged buildings in figure 26 could be considered severe by damage states of this study.

Another possible reason is that the duration time for major ground shaking of Erzincan EQ is less than 10 seconds (very short) & only one displacement cycle of large amplitude as shown in figure 23 and figure 24. Therefore cyclic degradation of masonry walls was not significant which helped to show good performance.

In addition, the strength of materials used in Erzincan building is unclear and could be much stronger the strength values assumed for buildings in Jordan.

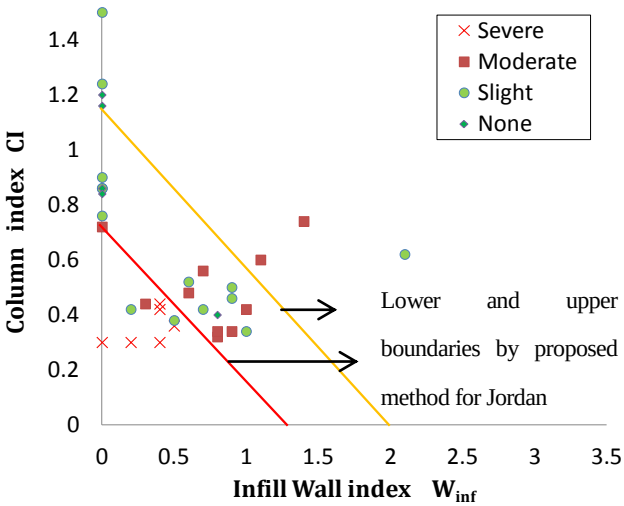

Figure 26 Comparison with data of damaged RC buildings with masonry infill of Erzincan earthquake 1992

6. CONCLUSIONS

Due to economical and time problems, a first screening method that is more practical and simple was introduced. The proposed method is fast and could be adjusted to be suitable to other regions with different seismicity and structural detailing. Buildings identified as vulnerable should undergo a more detailed procedures for further seismic evaluation.

An average structural damage and economic loss of a whole city could be easily estimated in a short time based on the probability of damage of each zone in the proposed method. In additional, screening the city using this study helps to estimate roughly the resources needed for buildings` retrofit and risk mitigation.

In future research, more prototypes with different Column and Wall indices should be checked to increase the accuracy of constructing the vulnerability zones.

Further improvements needs to be studied in further research such as adding regularity reduction index for buildings with irregular shapes. Material quality index should also be introduced to account for buildings construction quality and deterioration after construction. Even though captive columns are not commonly observed in Jordanian buildings, but captive columns should also be investigated in further research. In addition, Jordan`s material properties experiments are needed to account for the assumed values in this paper.

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