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STUDY OF SEISMIC EVALUATION METHODS OF RC BUILDINGS WITH MASONRY INFILL WALLS; A CASE STUDY OF BUILDINGS IN JORDAN

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ABSTRACT: This paper investigates the seismic evaluation of several prototype RC buildings with masonry infill walls. First Existing buildings in Jordan will be examined using the seismic vulnerability assessment methods of Japanese Standard (JBPDA) and the American standard, ASCE 31. Results of both standards are compared and advantages and disadvantages are discussed. Secondly, a more practical seismic method that is more effective especially in developing countries is proposed. Finally, conclusions and future research points of the proposed method are discussed.

Key Words: Seismic Evaluation, Masonry infill walls, RC Buildings in Jordan,

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 Fig1 and Fig 2.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.

The main objective of this paper to evaluate seismic capacity of selected buildings using existing evaluation methods and to propose a simpler and more practical method that could be used in developing countries. To make this study more realistic, existing RC buildings with masonry infill walls in Jordan is taken as an example.

This study is divided into two main parts. In Part 1: First, Characteristics of buildings and seismicity in Jordan is presented. Second, the ASCE (ASCE31, 2003) method and Japanese JBDPA (JBDPA 2001b) will be applied on selected existing buildings in Jordan. The ASCE method and Japanese method were chosen because they are the most famous evaluation methods. Results of the two standards are compared and discussed.

In Part 2, a more practical method is proposed. Actually the original concept of this method 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 area, wall area and floor area. However, 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` method to be suitable to RC buildings with masonry infill walls in different seismic regions.

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

Fig. 2 Haiti earthquake 2010 (Photo by David Lattanzi , http://nees.org/resources/1797)

PART1: APPLYING EXISTING SEISMIC EVALUATION METHOD

Seismicity and Characteristics of buildings in Jordan:

Jordan is located along the seismically active Dead Sea Transform Fault that extends 1000 km from the Red Sea to Turkey. Current estimates predict a major earthquake in the region roughly every 100 years. It is not until 2004 that a seismic code for Buildings based on UBC code 1997 was implemented. . In additional, Jordan does not have a seismic evaluation standard method and needs it as fast as possible to take precautions and start retrofitting vulnerable buildings.

RC structures are widely used in Jordan. Typical building plan is shown in Fig. 3. The exterior infill walls are composed usually of 3 layers; stone facing, plain concrete and in some cases hollow concrete blocks as shown in Fig. 3. 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.

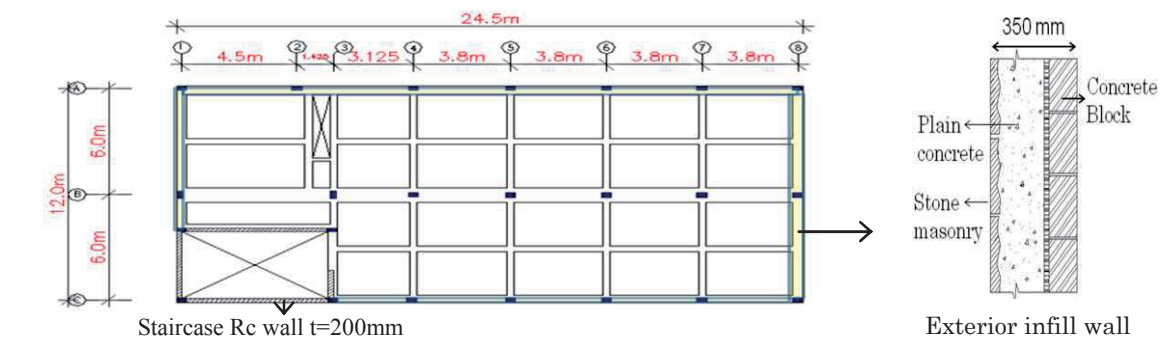


Fig. 3 Typical building plan (building No.6)

Typical beam and column sizes are shown in Fig. 5. 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.

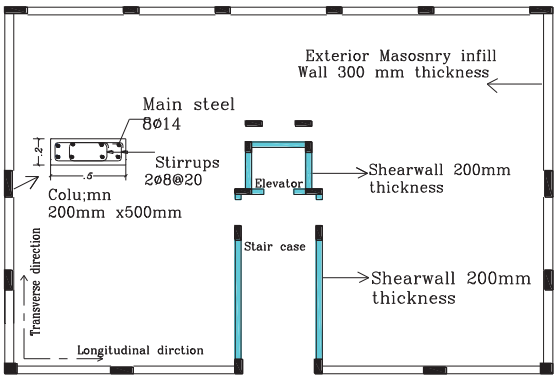


Fig. 4 Typical plan of building No.3

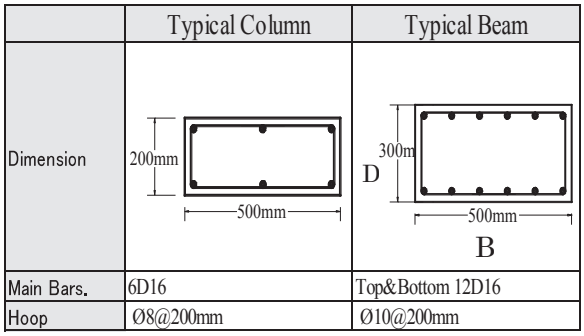


Fig. 5 Typical detail of Column and Beam

Method of analysis:

8 existing buildings are chosen with different usages and floor areas as shown in Table 1. The structural plan of Building No.6 and No.3 are shown in Fig. 3 and Fig. 4 respectively. The first story of the 8 selected buildings is checked using the Japanese standard (JBDPA) and the American standard ASCE 31. An overview of the two methods is mentioned below.

Table 1 list of selected buildings

Build No.	No. of stories	Floor Area m^2	Function of building
1	4+1Basement	150	Residential
2	4 Stories	270	Residential
3	4+1Basement	256	Hotel
4	4 Stories	886	School
5	3 Stories	350	Residential
6	3 Stories	300	Residential
7	1 Story	120	Commercial
8	4+1Basement	420	Residential

Overview of the Japanese method

The JBDPA standard, Japanese Standard for Seismic Capacity Evaluation of Existing Reinforced Concrete Building, (JBDPA 2001b) has 3 screening levels, with the 1st level as the simplest and most conservative and the 3rd level is the more complex. However, the 2nd level is the common procedure used for seismic evaluation and retrofitting buildings in Japan. Therefore, this study used the 2nd level procedures.

The I_s-Index which represent seismic performance of structure can be calculated by Eq.(1) at each story and each direction. E_0 is a basic structural index calculated by Eq.(2).

$$Is = E_0 \times S_D \times T \tag{1}$$

$$E_0 = \phi \times C \times F \tag{2}$$

C-Index is strength index that denotes the lateral strength in terms of shear force coefficient. *F*-Index denotes the ductility index 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. The product of the Strength and ductility is actually representing the Energy absorption capability of the building as shown in figure 1. *Ø* is story index that is a modification factor to allow for the mode shape of the response along the building height. S_D and *T* are reduction factors to modify E_0 in consideration of structural irregularity and deterioration after construction, respectively. Only main concept of method is presented, for more details refer to the main reference.

The Seismic Evaluation Standard recommends as the demand criterion that I_s-Index higher than 0.6 should be provided to prevent major structural damage or collapse. This criterion is based on the correlation study from the past earthquake damage in Japan. Therefore, in countries with lower seismicity level the demand criterion of I_s index 0.6 could be reduced. An applicable demand of I_s index needs further research in countries with moderate or low seismicity. In this study, criteria I_s index of 0.6 is used.

C index and F index of columns and RC walls Equations in JBDPA standard was used. However, equations to calculate the masonry walls strength and ductility are not mentioned in the standard because masonry walls are not a common practice in Japan. Therefore values proposed by the Chi-Chi Earthquake Report by AIJ (AIJ Report 2000) were used. In that report a value of average shear stress τ= 0.6 N/mm^{2}for Masonry walls without openings and a value of shear stress τ = 0.2 N/mm^2 for walls with openings was employed. Ductility Index F of masonry infill of F=0.8 was assumed.

Overview of the ASCE 31 method

The ASCE 31(ASCE31, 2003) is composed of three levels for seismic evaluations. Tier 1 which is composed of a checklist statements and quick checks. If there are potential deficiencies by Tier 1 then those deficiencies are checked using Tier 2 or Tier 3. Tier 2 is a linear static or dynamic analysis of a mathematical model of the building. Tier 3, is the more complex level, in which nonlinear analysis is used. In this study, Tier 1 is applied to the selected buildings.

Prior to completing Tier 1, requirements should be selected, such as Level of performance, Level of seismicity and Building type. In this study, Level of performance is chosen to be Life Safety level. Level of Seismicity is chosen as moderate seismicity with S_{Ds} (Design short-period spectral response acceleration) = 0.5g and S_{D1} (Design spectral response acceleration at 1 second) = 0.2g which corresponds to moderate seismicity spectral acceleration with Soil of class B (rock soil). These parameters are assumed based on the approximate seismicity and Soil type of Jordan. Building type C3 is selected which is a Concrete frame with infill masonry shear walls and stiff diaphragms

As mentioned above, Tier 1 has different checklists. Most checklists can be done by simple calculations and site investigations. In this paper, only an overview of Shear stress check will be mentioned. For more detailed explanation, refer to the main reference.

To calculate average shear stress, first the Pseudo lateral force (V) is calculated by the Eq(3)

$$V=C.S_a.W \tag{3}$$

Where: C= Modification factor to related expected maximum inelastic displacement to displacements calculated for linear elastic response, C is taken from tables in the standard

S_a= S_{D1}/T and less than S_{DS}. S_{D1} and S_{DS} are mentioned previously. T is the fundamental period of building. W is the effective seismic weight of the building.

The pseudo lateral force is then distributed vertically. Then the average shear stress in walls, v_j^{avg}, could be checked using the Eq(4)

$$v_j^{avg} = \frac{1}{m}\left(\frac{Vj}{Aw}\right) \tag{4}$$

Where: Vj : Story shear at level j, Aw: summation of the horizontal cross-sectional area of the shear walls in the direction of loading and m: component modification factor which depends on the type of walls taken from tables in the standard. m is taken as 1.5 for Unreinforced masonry walls. Openings are deducted when computing Aw. The criteria states that average shear stress in masonry walls of concrete units shall be less than 0.48 Mpa (70 psi). Fig. 7 shows the main concept of the Eq(4). Comparing Fig. 6 and Fig. 7, the two methods ASCE 31 and JBDPA use the *Equal energy principle* but each method expresses it in a different way.

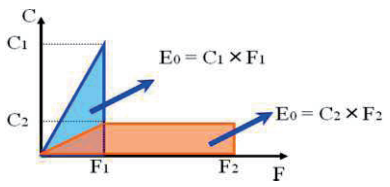


Fig. 6 JBDPA method basic concept

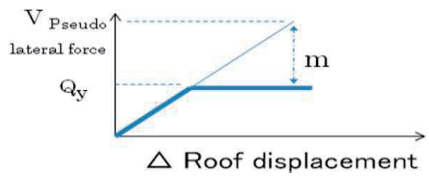


Fig. 7 ASCE 31 basic concept

Results

The results using the second screening method of the Japanese method is shown in Table 2. The S_D index (Shape regularity index) is greater than 1 in some buildings because those buildings have basement. As shown in the table, Almost all buildings had I_s index of less than 0.6 in at least one direction. An exception is building No. 7 which only a 1 story building.

As for results of Tier 1 of ASCE 31 method, all selected buildings passed all checklists except for shear stress checklist. The results of the weakest direction of the shear stress check are shown in Table 3. Some of the buildings have RC walls in both directions. The ASCE 31 standard doesn't indicate

calculation procedures of buildings with composite structural system of RC walls and masonry walls. Therefore, the area of the RC walls is assumed equivalent to 3 times of the area masonry walls and added to the total area of the masonry walls. This assumption is based by comparison between the m factor and maximum shear stress (indicated in the ASCE 31 standard) of unreinforced masonry walls and RC walls.

Table 2 Results using JBDPA method

Build No.		Longitudinal Direction		Transverse direction	
	S_D	E_o	I_s	E_o	I_s
No.1	1.15	0.35	0.41	0.53	0.61
No.2	1	0.26	0.26	0.54	0.54
No.3	1.2	0.31	0.37	0.75	0.9
No.4	0.92	0.47	0.43	0.49	0.45
No.5	0.92	0.47	0.43	0.33	0.3
No.6	1	0.7	0.7	0.4	0.4
No.7	1	0.82	0.82	0.65	0.65
No.8	1.1	0.5	0.55	0.47	0.52

All selected buildings failed in shear stress check, less than criteria of 0.48Mpa, except building No.7. In this case, according to ASCE 31 standard, this deficiency should be check by Tier 2. However, procedures on how to model the masonry walls in the mathematical model are not stated explicitly. Therefore results of Tier 2 and Tier3 could be different from an engineer to another depending on his model and assumptions.

In Fig. 8, the results are normalized by the criteria standard of both standards which Is index of 0.6 in JBDPA and 0.48 Mpa (70 psi) in the ASCE 31 standard. Both methods have similar results. However, The Japanese Criteria used here is I_s index of 0.6 which is actually for high seismic regions. In the other hand, using the ASCE 31 method, moderate seismicity was chosen for the selected buildings. Therefore, in this study, the ASCE 31 is a bit more conservative than JBDPA standard.

Table 3 Stress check results using ASCE method

Build No.	Axis	Length of RC walls (m)	Thickness of RC wall (mm)	Length of Masonry walls (m)	Thickness of Masonry walls (mm)	Area of walls (mm^2)	Average shear stress v^{avg} (Mpa)	Remarks
1	Longitudinal	-	-	11.3	300	3390000	0.80	N.G
2	Longitudinal	-	-	20.5	300	6150000	0.70	N.G
3	Longitudinal	1.8	200	18.8	300	6720000	0.73	N.G
4	Longitudinal	19	200	33.7	300	21510000	0.62	N.G
5	Transverse	5	200	20.4	300	9120000	0.51	N.G
6	Transverse	4.8	200	16.6	300	7860000	0.53	N.G
7	Transverse	-	-	9.5	300	2850000	0.33	O.K (1 story building)
8	Transverse	9.6	200	19.6	300	11640000	0.53	N.G

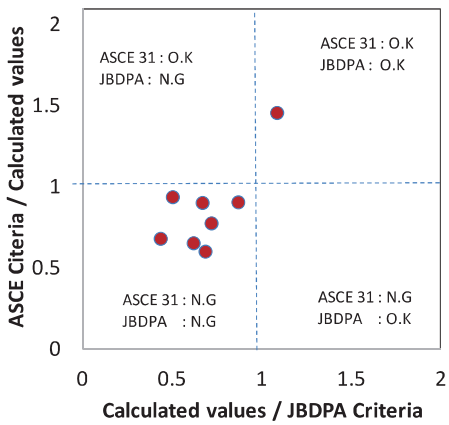


Fig. 8 Comparison of results of ASCE 31 and JBDPA standard

From the results of previous section, it appears that almost all buildings are unsafe and the government should start screening and seismic retrofitting plans. However, in developing countries like Jordan, 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 Jordan 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. From the above points it is concluded that a simple, low cost and fast seismic screening method is needed as a first screening method.

PART2: PROPOSAL OF SEISMIC EVALUATION METHOD

In Japan, a figure called Shiga map (Shiga 1968) was one of the important methods in the development of seismic evaluation method for low rise RC buildings. The Shiga map screens the buildings into zones with different vulnerability levels according to their Column area, Wall 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 of buildings 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. These methods are only applicable to its region because of different seismicity level, material properties and structural details.

In this study, a method of recalibrating the Shiga map for different regions with different structural details is presented.

Method of Analysis:

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

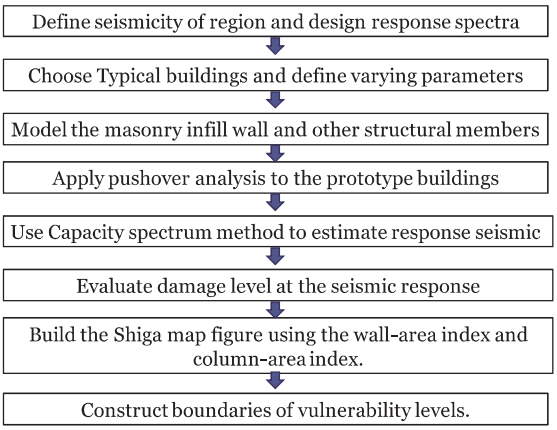


Fig. 9 Flowchart of the proposed method

Define seismicity of Jordan

Different regions have different seismicity levels in Jordan. UBC code (UBC 1997) is used to construct the design response spectrum for two soil types, soil Type SB (rock) and soil Type Sc (very dense soil and soft rock), shown in Fig. 10. In this study, the response spectra of Sc Soil type were used.

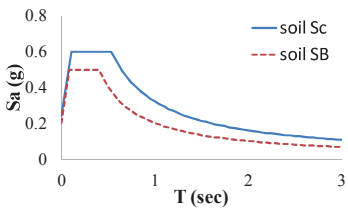


Fig. 10 Design response spectra for different soil types in Jordan

Prototype buildings characteristics

3 prototype buildings with different floor areas were used for analysis. Fig 11 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. 4 cases were assumed for each building, assuming number of stories ranging from 1 till 4 stories. Exterior infill walls with opening of $1m^2$ in each wall and thickness 300 mm were assumed, see Fig. 12. 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.

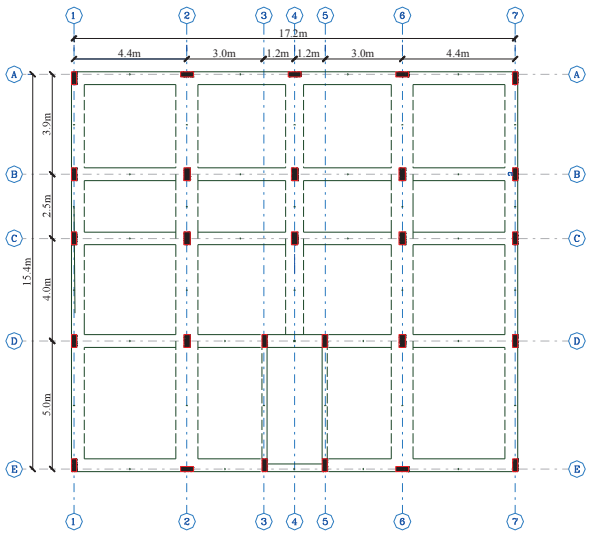


Fig. 11 structural plan of prototype building B

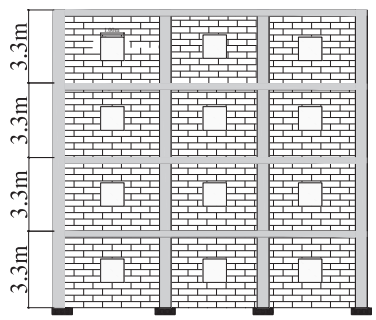


Fig. 12 Elevation and exterior infill walls

Modeling of structure

In this study a simple two-dimensional pushover analysis using computer program SNAP is carried out for the longitudinal frames, see Fig. 13. 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. A tri-linear relation is used for rotational and shear springs exterior, see Fig. 14. 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(5) and Eq(6) 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.

$$W_{ef} = 0.175(\lambda_h H)^{-0.4}\sqrt{H^2 + L^2} \tag{5}$$

$$\lambda_h = \left[\frac{E_w t_w \sin(2\theta)}{4E_c I_c H_{\inf}}\right]^{\frac{1}{4}} \tag{6}$$

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 transforming the expected infill shear strength, V_{ine} , which is determined by Eq(7) and Eq(8) using the methodology given in Section 7.5.2.2 in FEMA 356 (FEMA 2000) :

$$V_{ine} = V_{me}.An \tag{7}$$

$$Vme = \frac{0.75\left(Vte + \frac{P}{An}\right)}{1.5} \tag{8}$$

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 reduced using the factor $\lambda_{opening}$ calculated using Eq(9) based on the work of Dawe and Seah (Dawe1988).

$$\lambda_{opening} = 1 - \frac{1.5L_{opening}}{L\inf}; \lambda_{opening} \geq 0 \qquad (9)$$

Fig 14-c) shows the backbone curve of infill wall.The d value shown in the Fig 14-c 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).

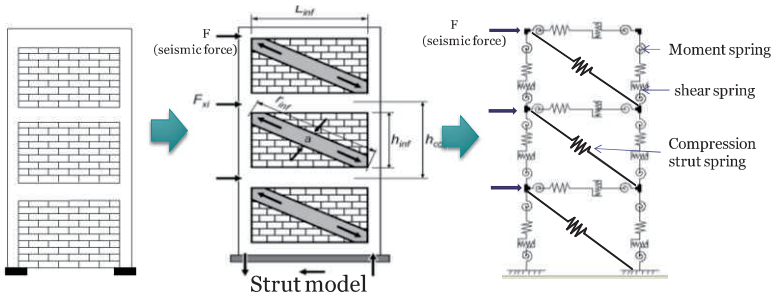
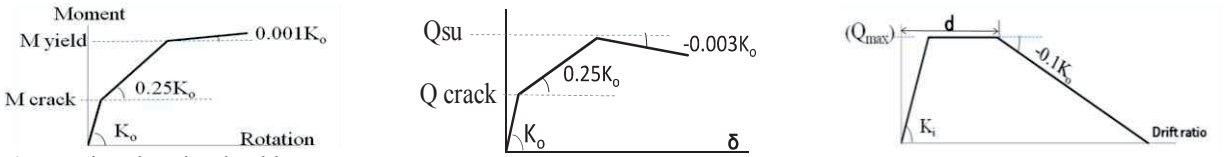


Fig. 13 Model of Pushover analysis



a) Rotational spring backbone curve b) Shear spring backbone curve c) Strut spring back bone curve

Fig. 14 Backbone curves of springs

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 floor of each building at the seismic response displacement using the pushover analysis. The damage states used are those used in HAZUS99 (FEMA 1999). 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. Structure may exhibit permanent lateral deformation.

Results:

Properties and results of the prototype buildings are shown in Table 4. The Column index, CI, is percentage ratio of the cross sectional area of columns of 1st floor 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 to the total of 1st floor to the total floors area of the prototype buildings. It should be noted that the length of the openings in infill walls are deducted from the total infill length.

The Column index, Wall index (WI) and damage expected of each building are shown in Fig 15. Fig. 15 is based on the concept of Shiga map. Hasan and Sozen (Hasan1997) used a similar figure for actual damage of buildings in Erzincan earthquake. Vulnerability Zones are divided into 3 zones, for which zone A is the most vulnerable and zone C is the least. In future research, more prototypes with

different Column and Wall indices should be checked to increase the accuracy of constructing the vulnerability zones.

Table 4 Properties and results of the prototype buildings

Building No.	Total Column area m^2	Total Infill Wall area m^2	Total Floor Area m^2	Column index%	Infill Wall index %	Evaluated Damage level
Building A (1 stories)	3.8	11.04	438	0.87	2.52	none
Building A (2 stories)	3.8	11.04	876	0.43	1.26	slight
Building A (3 stories)	3.8	11.04	1314	0.29	0.84	severe
Building A (4 stories)	3.8	11.04	1752	0.22	0.63	severe
Building B (1 stories)	3.1	6.63	271	1.14	2.44	none
Building B (2 stories)	3.1	6.63	543	0.57	1.22	slight
Building B (3 stories)	3.1	6.63	795	0.39	0.83	moderate
Building B (4 stories)	3.1	6.63	1060	0.29	0.63	severe
Building D (1 stories)	1.48	3.36	101	1.46	3.32	none
Building D (2 stories)	1.48	3.36	202	0.73	1.66	slight
Building D (3 stories)	1.48	3.36	304	0.49	1.11	moderate
Building D (4 stories)	1.48	3.36	405	0.37	0.83	moderate

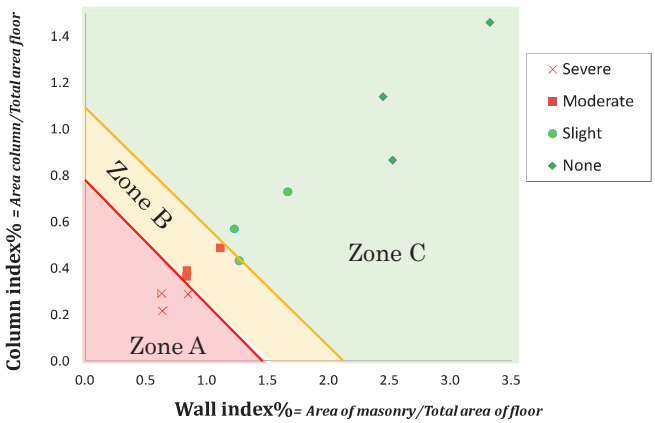


Fig. 15 Proposed evaluation method

To make this method more practical; zones` boundary lines are converted to their line equation and normalized. The inventory buildings having CI and WI_{inf} in Eq(10) index *less than 1*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 1* in Eq(11) have lower priority to be checked . It is very important to note that the proposed method is a first screening method and a further analysis is also necessary for buildings marked with less vulnerable using the proposed method. It should be noted here that Jordan has regions with different seismicity levels and soil condition. Therefore, the boundaries and vulnerability zones proposed here might be conservative to regions with lower seismicity.

$$1.2CI+0.6WI_{inf} \geq 1 \tag{10}$$

$$0.9CI+0.45WI_{inf} \geq 1 \tag{11}$$

The proposed boundaries are compared and plotted with data of damaged buildings by Erzincan earthquake 1992 in Turkey, see Fig 16. The damaged buildings data are taken from data collected by Hasan and Sozen (Hasan1997). However, Erzincan earthquake`s data contain also buildings with RC walls, therefore the data are filtered for buildings having only RC columns and Infill walls.

The boundaries from the proposed buildings showed good agreement with damage of the Erzincan`s buildings. This agreement doesn't mean that this method is applicable to other regions with different seismicity and structure details and material. This agreement might mean that buildings in Jordan and Turkey have similar structure detailing; this is possible since both countries are Middle Eastern countries and lies near to each other. It also means that design response curve used in this analysis might coincide well with that of the Erzincan`s earthquake.

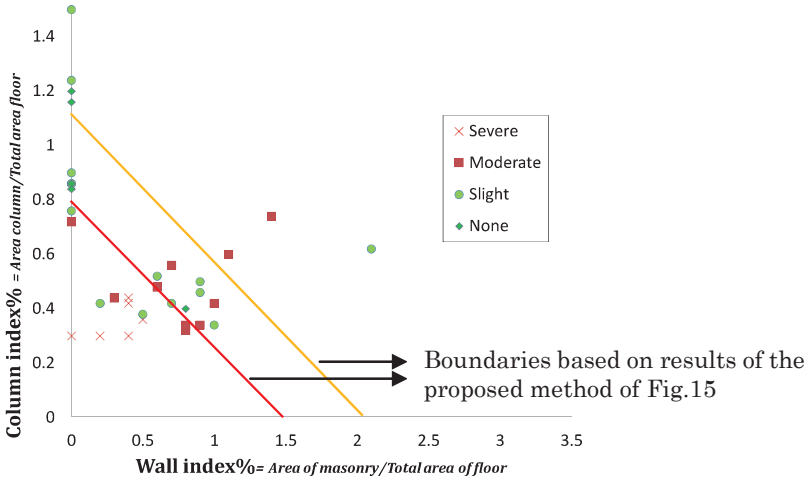


Fig. 16 Damaged RC buildings with masonry infill of Erzincan earthquake 1992 in Turkey

CONCLUSIONS

Selected existing buildings of Jordan were checked using the Japanese standard JBPDA and American standard ASCE 31. Almost all buildings failed using both methods. 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.

However, further research is needed to improve the results, 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, Experiments of Jordan`s material properties are needed to account for the assumed values in this paper.

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