SEISMIC EVALUATION OF REINFORCED CONCRETE BUILDINGS WITH MASONRY INFILL WALL

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Abstract

Reinforced concrete buildings with masonry infill is common practice in many developing countries. However, many practicing engineers still assumes that the walls are non-structural walls due to complexity of evaluating the failure modes of masonry infill and its interaction with the surrounding frame. A practical and simpler seismic evaluation method is needed to assess masonry infill walls. The Japanese seismic evaluation method is a practical seismic evaluation method. However, procedure to calculate the masonry walls strength and ductility are not mentioned in the standard because masonry walls are not a common practice in Japan.

First, past experimental data and in-plane strength estimation methods are reviewed and compared. A simpler equation with quite good accuracy based on experimental data is proposed to calculated the in-plane shear capacity. Then, reduction factor of lateral strength due to openings proposed by different researchers is compared with experimental data to check their applicability. Deformation capacity and ductility of RC frames with masonry infill is studied based on experimental data. It was found that the deformation capacity is of a wide range and influencing parameters need further research. Finally, recommendations on how to address the influence of masonry infill into the Japanese seismic evaluation are proposed.

Keywords: RC frames, Unreinforced masonry infill, seismic evaluation, In-plane strength, Deformation capacity.

1. Introduction;

Recent earthquake such as Nepal 2015 and Haiti 2010 proved the necessity of a practical seismic evaluation method to evaluate seismic capacity of buildings and if necessary to retrofit, in order to avoid repeating catastrophic disasters in other developing countries.

Many of the reinforced concrete buildings in developing countries use masonry as partition walls (see Fig.1). The influence of masonry infill walls and how it greatly changes the behavior of structure and its beneficial influences and negative influences is well recognized from past experience of earthquake disasters and mentioned by several researchers.

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

However, many practicing engineers still assumes that the walls are non-structural walls due to complexity of evaluating the failure modes of masonry infill and its interaction with the surrounding frame. Even with detailed analysis such as FEM models, the masonry infill has many discrepancies in material that makes it difficult to capture the exact failure mode, shear strength and deformation capacity. In additional, the existing buildings make a huge considerable number and a detailed analysis using micro-modelling or complex models takes a considerable amount of time, highly experienced researchers and resources. A practical seismic evaluation method is necessary to screen vulnerable buildings.

In Japan, seismic capacity evaluation and strengthening have been applied to existing buildings especially after the 1995 Kobe Earthquake. Japan with its long experience with devastating earthquakes has developed a practical standard for seismic evaluation (JBDPA 2001) (The Japanese standard for Seismic Evaluation of Existing Reinforced Concrete Buildings) [1]. This standard has proved its effectiveness in the Great East Japan Earthquake 2011 were most of the evaluated buildings and if necessarily retrofitted based on the standard evaluation showed fairly good performance and prevented severe structural damage [2] (Maeda et al 2012). However, masonry infill influences are not addressed in the Japanese standard since the masonry partition walls are not used in Japan.

The purpose of this study is to propose a practical and simpler methods to estimate the strength and deformation capacity of unreinforced masonry infill in RC frames based on past research and experiments conducted by different researchers. Then, recommendations are presented on how to address the influence of masonry infill into the Japanese seismic evaluation.

2. Overview of the Japanese standard;

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 as the more complex and more 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. The 2nd level is the common procedure used for seismic evaluation and retrofitting of buildings in Japan.

According to the standard, the seismic performance index of a building is expressed by the Is-index calculated by Eq. (1) for each story and each direction.

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

S_D and T are reduction factors to modify E_0 in consideration of structural irregularity and deterioration after construction, respectively. E_0 is a basic seismic index of structure which is in general the product of product of strength index (C), ductility index (F) and story index (φ) as shown in Eq. (2). C-Index is strength index that denotes the lateral strength of the buildings in terms of story shear coefficient. F-Index denotes the ductility index of the building 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. φ is story index that is a modification factor to allow for the mode shape of the response along the building height

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

Figure 3 shows the basic concept of Eq. (2) which is actually based on the equivalent energy concept.

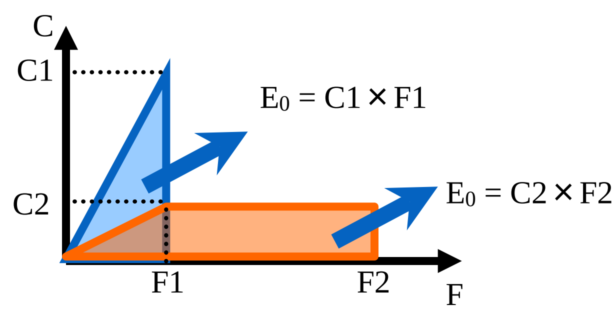

Fig. 2 Main Concept of basic seismic index Eo

Is- index criteria for second level screening should be equal or higher than 0.6 to prevent major structural damage or collapse. This criterion is based on the correlation study from the past earthquake damage and the calculated indices for the damaged buildings.

Calculations of C index and F index of columns and RC walls for first and second screening are direct forward and stated in the JBDPA standard [1]. However, procedure to calculate the masonry walls strength and ductility are not mentioned in the standard because masonry walls are not a common practice in Japan. This study will address recommendations on the calculation of C strength index and F ductility index for masonry infill.

3- In-plane Strength Capacity of unreinforced masonry infill;

3.1 Literature review considering the in-plane lateral strength;

The in-plane capacity of masonry infill depends mainly on the type of failure mechanism developed. The failure mechanism types and identification is slightly different between building standards or researchers. The most recognized failure modes are corner compression failure mode and sliding shear failure mode. Diagonal tension of masonry infill is considered as a serviceability limit and not considered failure mechanism since lateral load is still carried by the masonry infill. In this study, the most commonly strength calculation methods that are cited by many researchers will be mentioned and compared to experimental results.

3.1.1 Compression failure mode;

FEMA 306 [3] adopted a modified version of the method suggested by Stafford-Smith to calculate the compression failure of the equivalent diagonal strut. The shear force (horizontal component of the diagonal strut capacity) is calculated as Eq. (3), Eq. (4), Eq. (5);

$$\mathrm{V} = \mathrm{W_{ef}}.\mathrm{t}_{inf}.fm_{90}.cos\theta \tag{3}$$

$$\mathrm{W_{ef}} = 0.175.(\lambda_h H)^{-0.4}.d_m \tag{4}$$

$$\lambda_h = \left[\frac{E_w t_w \sin(2\theta)}{4E_c I_c H_{\mathrm{inf}}}\right]^{\frac{1}{4}} \tag{5}$$

where;

W_{ef} = equivalent strut width calculated using Eq (3) which is based on the work of Mainstone [4].

t_{inf} = infill thickness, Ew and Ec are the moduli of elasticity of the infill wall and the concrete. H_{inf}, H are the net height of the infill wall, the story height. θ= arctan(H_{inf}/L_{inf}) (the inclination of the diagonal).

I_c = moment of inertia of the column of the frame, d_m = diagonal length of masonry infill

fm_{90} = expected strength of masonry in horizontal direction, which may be set at 50% of the expected prism compressive strength

Stafford-Smith and Coull [5] used also the equivalent strut analogy and estimates the strength of masonry infill as Eq. (6);

$$\mathrm{V} = fm.\mathrm{t}_{inf}.\frac{\pi}{2}.\sqrt[4]{\frac{4.E_c.I_c.H}{E_m.t}} \tag{6}$$

where; *fm* is the prism compressive strength of masonry

Liauw and Kwan [6] proposed limit analysis with different masonry infill compression failure modes and the minimum of this failure modes (stated in Eq. (7) Eq. (8) & Eq. (9)) is the expected in-plane strength. This method not only estimates the masonry infill strength but calculate the in-plane strength of the whole system of RC frame and masonry infill.

$$V_1 = fm.t_{inf}.h_{inf}\sqrt{\frac{2(M_{pj}+M_{pc})}{fm.t_{inf}.h^2}} \qquad \textit{mode failure 1} \tag{7}$$

$$V_2 = \frac{fm.t_{inf}.h_{inf}}{tan\theta}\sqrt{\frac{2(M_{pj}+M_{pb})}{fm.t_{inf}.{h_{inf}}^2}} \qquad \textit{mode failure 2} \tag{8}$$

$$V_3 = .\frac{fm.t_{inf}.h_{inf}}{6}+\frac{4M_{pj}}{h_{inf}} \qquad \textit{mode failure 3} \tag{9}$$

Where; M_{pc} is the plastic moment capacity of the column, M_{pb} is the plastic moment of the beam and M_{pj} is the minimum of M_{pb} and M_{pc}.

Flanagan and Bennet [7] based on their experimental tests stated a different conclusion from previous researchers that corner crushing capacity doesn't seem to change because of frame properties and geometry and proposed simple equation Eq. (10) as;

$$V = K_{ult}.t_{inf}.fm \tag{10}$$

Where k_{ult} is an empirical value based on their experiments, it is suggested to be 246mm.

3.1.2 Sliding shear failure;

FEMA 306 [3] suggests that Mohr-Coulomb failure criteria can be used to assess the initial sliding shear capacity of the infill as Eq. (11);

$$V = \tau_o.t_{inf}.l_{inf}+\mu.N \tag{11}$$

Where μ is the coefficient of sliding friction along the bed joint, FEMA306 does not suggest any values for μ. However, The New Zealand Society for Earthquake Engineering, NZSEE 2006 [8] which uses the same procedure as FEMA 306 suggests in the absence of such site specific data assume $\mu = 0.8$. N is vertical load on infill wall and the τ_o is the cohesive capacity of the mortar beds which in absence of data can be taken as Eq. (12).

$$\tau_o = \frac{fm_{90}}{20} \tag{12}$$

Where fm_{90} = expected strength of masonry in horizontal direction, which may be set at 50% of the expected prism compressive strength.

Paulay and Priestley [9] to assess the sliding shear capacity uses also the Mohr-Coulomb failure concept. They assumed that panel carries no vertical load due to gravity because of difficulties in constructing infill with a tight connection with the overlying beam of the frame and also because vertical extension of the tension column will tend to separate the frame and panel along the top edge. However, its assumed that the vertical component of the strut compression force acts as vertical load on the infill. It is suggested that maximum sliding shear force of masonry infill is thus as Eq. (13);

$$V = \frac{\tau_o.t.l_{mf}}{\left(1-\mu\left(\frac{h}{l}\right)\right)}.l_{inf}.t_{inf} \tag{13}$$

Where they recommended values of $\tau_o = 0.03fm$ and $\mu = 0.3$

3.2 Past experimental results;

25 specimens consisted of single span and single story of RC frame with masonry infill under static loading from 10 researchers [10,11,12,13,14,15,16,17,18,19] are shown in Table .1. The data are chosen for different types of masonry infill to represent a general case for different masonry types used in the world. The type of failure mentioned by researchers in Table 1 is usually a mixture of compression and sliding failure. In another word, masonry infill failure modes interact together to form a complex failure mode with many variations.

3.2.1 Comparison of existing methods with past experimental results;

Table 2 compares the ratio of maximum load of analytical results calculated using the equations mentioned in previous section 2.1 to the experimental results. The max lateral load of experiments in Table 1 is the lateral load of the whole system of masonry infill and RC frame. The analytical equations calculate the strength of masonry infill without the surrounding RC frame (except for Liauw method [6] which calculates the strength of the whole system). Therefore, in Table .2 the max lateral strength of frame was added to the masonry infill strength assuming its bare frame.

Liauw method [6] overestimates the strength by an average of 1.25. The simple method proposed by Flanagan [7] greatly overestimates the strength by an average of 1.58. However, it should be noted that the scale of experiments used in this study differ from a researcher to another and thus this might affect the K empirical factor suggest by Flanagan [7] in Eq. (10).

The methods proposed to calculate sliding failure by of FEMA 306 in Eq. (11) and Paulay & Priestly[9] in Eq. (13) greatly underestimates the strength even for cases where sliding failure actually occurs.

Eq. (3) of FEMA 306 and Eq. (6) of Stafford-Smith, slightly underestimates the masonry lateral strength with an average ratio of analytical to experimental ratio of 0.83 and 0.82. Even though these methods are proposed to calculate compression failure mechanism, it also estimated the sliding capacity or mixture of compression and sliding mechanism quite well. However, these methods look quite complex and requires a detailed data of material properties such as elasticity of masonry infill which might be difficult to acquire in a first level seismic evaluation. A simpler equation with quite good accuracy is needed for a first level seismic evaluation.

The Shear strength capacity of masonry infill τ_{inf} depends mainly on its compressive strength. Based on the previous experimental data in table. 1, the average of $\tau_{inf} \approx 0.065fm$. Assuming $\tau_{inf} \approx 0.05fm$ and the masonry infill lateral strength to be as Eq. (14)

$$V = 0.05fm.t_{inf}.l_{inf} \tag{14}$$

As shown in fig.3, this simple equation gives good approximation for both sliding shear and corner compression failure mode with an average ratio of analytical to experimental ratio of 0.83 as shown in Table 2. The masonry infill has many discrepancies in material and failure mechanisms, this make it difficult to propose the exact failure mode which makes the proposed equation more effective for first level seismic evaluation than time consuming equations.

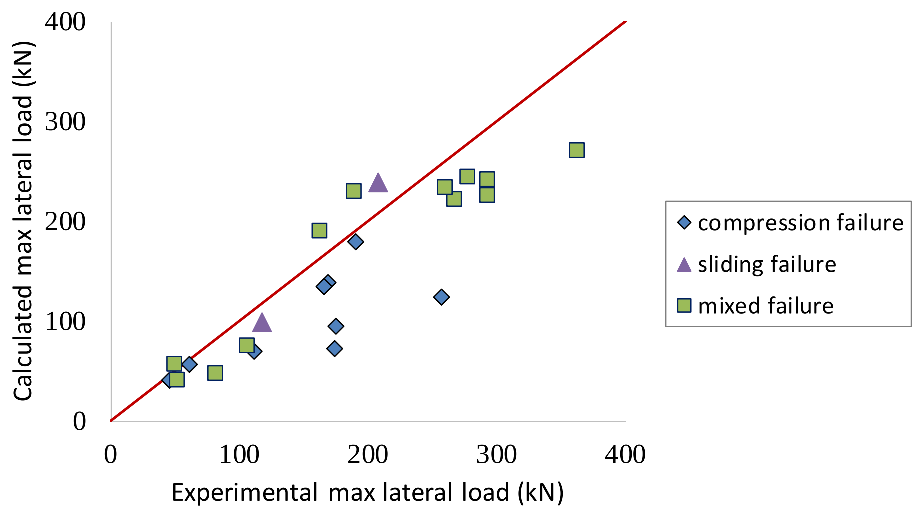

Fig.3 Comparison between proposed Eq. (14) and experimental results

Table. 1 Summary of experimental data of RC frame with masonry infill tested by different researchers

No.1	Researcher name	Test specimen name	Type of Infill	H_{inf} (m)	L_{inf} (m)	H/L ratio	t_{inf} (mm)	*fm* (MPa)	Max lateral load (kN)	R-max %	Ru %	Failure type & Damage observed
1	Mehrabi et al [10]	3	solid bricks	1.42	2.13	0.67	92	15.1	278	0.4	1.16	Diagonal cracks , slip at masonry joints & Shear failure of column
2		4	hollow bricks	1.42	2.13	0.67	92	10.6	162	0.63	1.45	Corner crushing with some slip at masonry joints
3		5	solid bricks	1.42	2.13	0.67	92	13.8	267	0.79	1.42	Diagonal cracks , slip at masonry joints & Shear failure of column
4		6	hollow bricks	1.42	2.13	0.67	92	10.1	207	0.61	1.78	Horizontal slip
5		7	solid bricks	1.42	2.13	0.67	92	13.6	490	0.71	1.04	Corner crushing
6		8	hollow bricks	1.42	2.13	0.67	92	9.5	190	0.91	1.82	Diagonal cracks , corner compression & Shear failure of column
7		9	solid bricks	1.42	2.13	0.67	92	14.2	293	0.48	1.98	Diagonal cracks , slip at masonry joints & Shear failure of column
8		10	hollow bricks	1.42	2.95	0.48	92	10.6	190	0.4	1.88	Corner compression & Diagonal cracks
9		11	solid bricks	1.42	2.95	0.48	92	11.4	293	0.74	1.5	Diagonal cracks , slip at masonry joints & Shear failure of column
10		12	solid bricks	1.42	2.95	0.48	92	13.6	363	0.55	1.02	Diagonal cracks , slip at masonry joints & Shear failure of columns
11	Maidiawati[11]	IF-FB	brick with plaster	1.00	1.46	0.68	140	2.9	174	0.5	1.6	Diagonal cracks and corner crushing and shear failure of column'(column shear ≈drift 2%)
12		IF-SBW/FM	Brick	1.00	1.46	0.68	44	16.3	175	0.5	1	Diagonal cracks and corner crushing and shear failure of column'(column shear ≈drift 2%)
13		IF_SB	brick with plaster	1.00	1.46	0.68	60	18.5	257	0.45	0.5	Corner crushing and shear failure of column'(column shear ≈drift 2%)
14	Jin et al [12]	IFRB	concrete block	0.61	0.89	0.69	48	6.7	61	0.4	1.5	Diagonal cracks and corner crushing
15		IFFB	concrete block	0.61	0.89	0.69	48	6.7	50	0.4	1.5	Diagonal cracks and slight horizontal slip
16	T. Suzuki et al [13]	1B-1S-H	concrete block	0.71	1.16	0.61	48	6.7	52	1	2	Diagonal crack and compression failure with some horizontal slip cracks
17		1B-1S-v	concrete block	0.71	1.16	0.61	48	6.7	46	1.5	2.8	Diagonal crack and compression failure
18	D. Kakaletsis et al [14]	S	hollow bricks	0.80	1.20	0.67	60	2.6	82	0.92	2.3	Diagonal crack and compression failure with some horizontal slip cracks
19	B. Blackard et al [15]	S	brick * double wythe	1.87	3.38	0.55	200	19.1	681	0.25	0.55	Diagonal cracks , Slip at masonry joints & Shear failure of column
20	Ali Mansouri et. al [16]	S	Brick	1.30	2.10	0.62	106	2.3	117	_	_	sliding along bed joint
21	Hanan AlNimry [17]	IF4	Stone and concrete	1.02	1.20	0.85	110	16.6	169	0.4	1.1	Spalling of stone masonry and compression failure of masonry infill
22		IF5	Stone and concrete	1.02	1.20	0.85	110	16.6	165	0.4	0.93	Spalling of stone masonry and compression failure of masonry infill
23	Imran et al [18]	Model 1	AAC blocks	1.50	1.50	1.00	100	3.0	111	0.88	2.88	Diagonal tension & Diagonal compression
24		Model 2	Brick	1.50	1.50	1.00	100	3.7	106	1.22	3	Sliding shear with some diagonal crushing
25	Zovkic et al [19]	Model 8	Brick	1.3	1.8	0.72	120	4.3	260	0.33	_	Diagonal & horizontal cracks, Sliding shear failure
Average										**0.64**	**1.60**	
standard deviation σ										**0.31**	**0.68**	

Where; H_{inf} = *is height of infill*, L_{inf}= *length of infill*, t_{inf}=*thickness of infill*, *fm= compressive strength of infill*,
R-max = drift angle at max load and Ru = drift angle when max load drops 20%

Table. 2 Comparison of ratio of analytical strength to the experimental max strength

No.	Researcher name	Test specimen name	FEMA 306 (Compression) Eq.(3)	Stafford -Smith & Coull (1991) Eq.(6)	Liauw & Kwan (1983) Eq. (7,8,9)	Flangan & Bennet (1999) Eq.(10)	FEMA 306 (Sliding) Eq.(11)	Paulay & Priestly (1991) (Sliding) Eq.(13)	Proposed method Eq.(14)
1	Mehrabi et al [10]	3	0.84	0.76	1.53	1.57	0.61	0.74	0.88
2		4	1.16	1.10	1.95	2.01	1.09	1.01	1.17
3		5	0.79	0.71	1.45	1.49	0.72	0.70	0.83
4		6	1.18	1.19	1.74	1.78	1.10	1.03	1.15
5		7	0.55	0.52	0.89	0.91	0.50	0.49	0.56
6		8	0.93	0.88	1.54	1.58	0.90	0.82	0.94
7		9	0.74	0.66	1.35	1.39	0.66	0.65	0.77
8		10	1.24	0.98	1.67	1.72	1.04	0.98	1.21
9		11	0.79	0.58	1.14	1.18	0.69	0.66	0.82
10		12	0.73	0.54	1.05	1.08	0.60	0.59	0.74
11	Maidiawatit et al [11]	IF-FB	0.42	0.50	0.63	0.82	0.33	0.37	0.41
12		IF-SBW/FM	0.55	0.64	0.93	1.25	0.39	0.47	0.54
13		IF_SB	0.46	0.50	0.85	1.23	0.32	0.40	0.48
14	Jin et al [12]	IFRB	0.92	1.00	1.23	1.99	0.81	0.87	0.93
15		IFFB	1.14	1.23	1.51	2.45	1.00	1.07	1.14
16	T.Suzuki et al [13]	1B-1S-H	0.79	0.80	1.16	1.95	0.61	0.69	0.79
17		1B-1S-v	0.90	0.92	1.31	2.21	0.69	0.79	0.90
18	D. Kakaletsis et al [14]	S	0.61	0.76	0.71	0.93	0.51	0.54	0.57
19	B. Blackard et al [15]	S	1.04	0.75	1.26	1.53	0.62	0.83	1.10
20	Ali Mansouri et. al [16]	S	0.92	1.03	1.07	1.13	0.73	0.78	0.84
21	Hanan AlNimry [17]	IF4	0.79	0.90	1.38	2.83	0.50	0.70	0.82
22		IF5	0.79	0.90	1.32	2.87	0.48	0.69	0.82
23	Imran et al [18]	Model 1	0.68	0.76	1.10	1.08	0.53	0.60	0.63
24		Model 2	0.77	0.86	1.32	1.31	0.58	0.67	0.71
25	Zovkic et al [19]	Model 8	0.94	0.96	1.15	1.20	0.81	0.85	0.90
Average			**0.83**	**0.82**	**1.25**	**1.58**	**0.67**	**0.72**	**0.83**
Standard Deviation			**0.22**	**0.21**	**0.32**	**0.57**	**0.22**	**0.19**	**0.22**

3.3 Reduction in lateral strength of masonry infill walls due to openings;

3.3.1 Literature review considering reduction of lateral strength due to openings;

Masonry infill in RC frame are used as partition walls and thought to be a non-structural element and therefore the presence of openings such as doors and windows are not an exceptional case but it is actually the norm case. Openings within the infill panels are the most significant parameter affecting the strength and seismic capacity of infilled systems. Although the strength of infills with openings is best assessed using rational strut and tie models with sub-components of materials or other advanced models, a simplified method to assess the reduction of strength for a first level seismic evaluation method is needed.

Based on the work of Dawe and Seah [20], the New Zealand Society for Earthquake Engineering (NZSEE) [8] recommends a simplified reduction factor to strength by factor named as λ_{op} as shown in Eq. (15)

$$\lambda_{op} = 1 - \frac{1.5L_o}{L_{inf}}; \qquad \lambda_{op} \geq 0 \tag{15}$$

where L_o is the maximum width of opening measured across a horizontal plane. Note the above equation implies that if the opening exceeds two-thirds of the bay width it may be assumed that the infill has no influence on the system. It should be noted that that this equation determines the reduction factor by the opening width and does not consider the effect of opening height.

Al-Chaar [21] conducted a large scale experiment and proposed an opening reduction factor to ultimate strength based on the ratio of area of opening to area of infill panel as in Eq. (16)

$$\lambda_{op} = 0.6\left(\frac{A_o}{A_p}\right)^2 - 1.6\left(\frac{A_o}{A_p}\right) + 1 \tag{16}$$

Where A_o and A_p are the area of opening and area of masonry infill panel respectively.
Tasnimi et al [22] based on experimental results on large-scale steel frames with clay brick masonry infills having openings, proposed a reduction factor λ_{op} as in Eq. (17) which have a similar concept of Eq. (16) of Al-Chaar [21];

$$\lambda_{op} = 1.49\left(\frac{A_o}{A_p}\right)^2 - 2.238\left(\frac{A_o}{A_p}\right) + 1 \tag{17}$$

3.3.2 Past experimental work considering the openings in masonry infill walls;

Several researchers have done experiments of steel frames having masonry infill with openings. However, only few researchers have done experiments of RC frames having masonry infill with openings. In this study、15 specimens consisted of single span and single story of RC frame with masonry infill having opening of different sizes and positions from 3 researchers , D. Kakaletsis et al [14], Ali Mansouri et al [16] and Blackard et al [15] are presented in Table 3. The reduction of strength is calculated based on ratio of the max lateral load of RC frame with masonry infill with opening to the max lateral load of RC frame with solid infill (no opening) which was tested in advance to

3.3.3 Comparison of existing methods with past experimental results;

Figure 4 shows the comparison of experimental reduction factor λ_{op} with analytical reduction factor λ_{op} calculated according to (NZSEE) [8] Eq. (15), Al-Chaar [21] Eq. (16) and Tasnimi et al [22] Eq. (17).

(NZSEE) Eq. (15) is the most conservative with and average ratio of analytical reduction factor λ_{op} to experimental reduction factor λ_{op} of 0.62 and standard deviation of 0.18. Al-Chaar [21] Eq. (16) showed good correlation with experimental results with an average ratio of analytical to experimental f actor of 0.92 and standard deviation of 0.14. Tasnimi et al [22] Eq. (17) has ratio of analytical to experimental factor of 0.81 and standard deviation of 0.15. Based on this study, AlChaar Eq. (16) is recommended to calculate reduction factor of strength for the first seismic evaluation.

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Table. 3 Summary of experimental data of masonry infill with openings

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Researcher name	Type of opening	Test specimen name	Size of infill panel			Opening			Max lateral load V (kN)		Reduction factor λ_{op}
			Height of infill H_{inf} (m)	Length of Infill L_{inf} (m)	Area infill (m2)	Location of opening	% Opening Area	Lo/L_{inf}	V_{op} opening kN	V_s solid infill kN	**V_{op}/V_s**
Ali Mansouri [16]	Door	DO	1.30	2.10	2.73	eccentric	16.5%	0.21	87.4	117.3	**0.75**
	Window	RWO	1.30	2.10	2.73	center	16.5%	0.36	91.2	117.3	**0.78**
	Window	LWO	1.30	2.10	2.73	center	27.5%	0.48	85	117.3	**0.72**
	Window	EWO	1.30	2.10	2.73	eccentric	16.5%	0.36	94.4	117.3	**0.80**
Blackard [15]	Window	SW	1.87	3.38	6.31	eccentric	11.6%	0.29	654	681	**0.96**
	Door	D	1.87	3.38	6.31	eccentric	16.0%	0.21	592	681	**0.87**
	Window	Lw	1.87	3.38	6.31	eccentric	19.0%	0.45	374	681	**0.55**
D. Kakaletsis [14]	Window	WO2	0.80	1.20	0.96	center	10.3%	0.25	66.5	81.5	**0.82**
	Window	WO3	0.80	1.20	0.96	center	15.7%	0.38	66.5	81.5	**0.82**
	Window	WO4	0.80	1.20	0.96	center	20.6%	0.50	65.1	81.5	**0.80**
	Door	DO2	0.80	1.20	0.96	center	20.9%	0.25	61.6	81.5	**0.76**
	Door	DO3	0.80	1.20	0.96	center	31.8%	0.38	57.1	81.5	**0.70**
	Door	DO4	0.80	1.20	0.96	center	41.9%	0.50	55.4	81.5	**0.68**
	Window	WX1	0.80	1.20	0.96	eccentric	10.3%	0.25	72.7	81.5	**0.89**
	Door	DX1	0.80	1.20	0.96	eccentric	20.9%	0.25	64.7	81.5	**0.79**

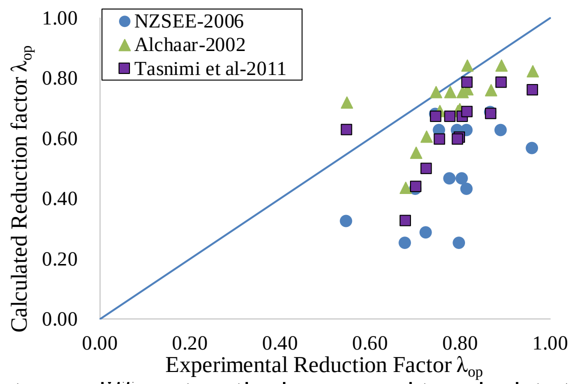

Fig.4 Comparison between different methods proposed to calculate the reduction of strength due to openings

3.4 Strength index C

3.4.1 Strength index C of masonry infill wall

The strength index C for each masonry infill wall is recommended to be calculated by the following Eq. (18):

$$C = \frac{V_{inf}}{\sum W} \tag{18}$$

Where $\sum$W = the weight of the building supported by the story concerned.

And V_{inf} is ultimate lateral load-carrying capacity of each masonry infill wall calculated using Eq. (19);

$$V_{\mathrm{inf}} = 0.05 fm.\, t_{inf}.\, l_{inf.} \lambda_{\mathrm{op}} \,. \tag{19}$$

Where fm , t_{inf} and l_{inf} are the compressive strength, thickness and length of masonry infill respectively.

λ_{op} is the reduction factor due to opening calculated using Eq. (16). Infilled frames with openings exceeding 50% of the panel area should be ignored.

It should be noted that partial infill panel such as cases in figure 5 and figure 6 is not considered as opening in this study and their beneficial influences should be ignored.

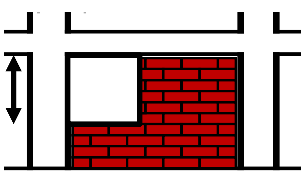

Fig.5 Partial infill masonry walls

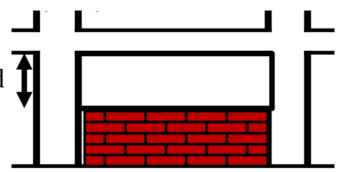

Fig.6 Partial infill masonry walls

3.4.2 Strength index C of RC columns surrounding the masonry infill;

Solid masonry infill alters the failure mode of surrounding RC frame. There are different failure mechanisms of surrounding RC column. In this study, 3 cases which commonly observed and are considered critical by other researchers [9] and [6].

Case 1: Plastic moment hinge at top and bottom ends of column; here the surrounding frame acts if it is a bare frame and plastic moment hinge are observed at the end of columns as shown in figure 7.

Case 2: Reduced clear height due to sliding failure mode of bed joints; If sliding of shear failure of masonry occurs, the structural mechanism changes from the strut diagonally braced frame in figure 7 to the knee-braced frame shown in figure 8. The masonry infill forces the column hinges to form at approximately mid-height and may result in column shear failure due to reduced height.

Case 3: Reduced clear height due to change of moment plastic hinges positions and shear failure of column. This the case of strong infill and weak frame, the presence of infills modifies and magnifies the shear demands on the frame members by shortening the distance between in-span plastic hinges which provides a high shear demand over a short column as shown in figure 9. High probability of the formation of this case occurs when the ratio of shear strength of the infilled wall to the bare frame is greater than a unity. In other words, this case could be avoided when the column shear strength sufficiently exceed the horizontal components of the force required for failure of the infill.

C-index of column surrounding a solid infill are calculated by the procedure stated in the JBDPA [1] for the 3 cases and the minimum is chosen, case 1 gives the minimum C-index.

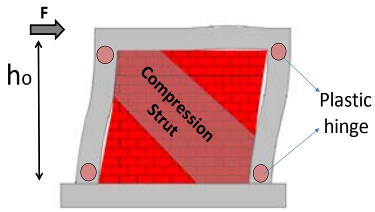

Fig.7 Failure mechanism; Case 1

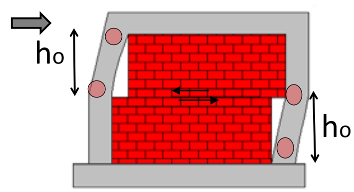

Fig.8 Failure mechanism; Case 2

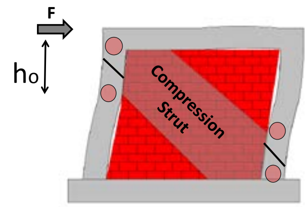

Fig.9 Failure mechanism; Case 3

As for partial infill panel such as cases in figure 5 and figure 6. The C index of RC column should be calculated for 2 cases and the minimum is considered. Case 1; The partial infill wall is very strong and stiff which alters the positions of plastic hinges therefore the clear height of the column is reduced as shown in figure 4 and figure5. Case 2; the partial infill wall is very light and flexible, therefore it does not alter the positions of plastic hinges and the clear height of column is taken if there is no infill. Calculating the C-index by the procedure stated in the JBDPA, case 2 gives the minimum C-index.

4. Ductility of RC frame with masonry infill walls;

Even though several researchers have studied and proposed models to estimate the in-plane strength of masonry infill, the deformation capacity of different failure modes and types of masonry material is not enough focused on.

4.1 Past experimental work

In this study, a backbone curve for RC frames with masonry infill is suggested as figure 10. The drift angle of R-crack, R-max and Ru are the drift angle at cracking point, maximum strength and when strength is degraded to 80% of the peak strength respectively as shown in figure 10.

Past experimental data of 25 specimens from 10 researchers [10,11,12,13,14,15,16,17,18,19] previously mentioned in Table .1 are used to approximate the drift angle. The data are chosen for different types of masonry infill to represent a general case for different masonry types used in the world.

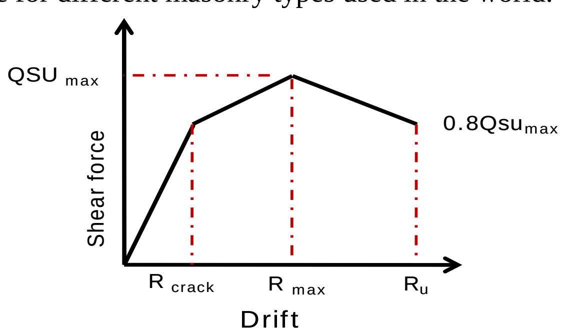

Fig 10. Backbone curve of RC frame with masonry infill

As for the R-crack, the FEMA 306 [3] states that diagonal cracking begins with the onset of nonlinear behavior at inter-story drifts of 0.25%. Mehrabi [10] states based on his experimental studies that first major crack in infill occurred at inter-story drift between 0.17% ～0.46%. In other experimental references the R-crack was not clearly stated. As for Drift angle (Rmax) at peak strength, based on the studied experiments, the Rmax drift has an average of 0.64% and most of values in the range of 0.4%~0.9% as shown in Table 1. The standard deviation was calculated to be 0.31. As for Ru drift angle (drift angle when strength is degraded to 80% of the peak strength) has an average of 1.6%. The data of Ru drift angle is of a wide range with a standard deviation of 0.68.

There are several parameters affecting the deformation of masonry infill such as compressive strength of masonry infill, ratio of strength masonry to strength of frame, slenderness ratio and failure mode. The influence of each parameter and their interaction with each other is still not clear and needs further study. It should be noted that the experimental data is of in-plane static loading, therefore the influence of out-plane loading to the in-plane deformation is not addressed.

4.2 Ductility F- index

The F-index of masonry infill needs further study and the available data of the parameters affecting the masonry infill deformation is considered not enough. However, in this study, calculation of the F-index of masonry infill is recommended to be taken as the minimum of the following 2 cases;

Case 1: Masonry infill failure occurs before the failure of surrounding frame; the F-index of masonry infill is recommended to be taken at R-max drift angle of 0.4% which is the lower bound of based on the previous experimental analysis which had an average of 0.64%. Lower bound is recommended since the out-plane is not considered in the previous experimental data. At R-max drift of 0.4% the damage imposed by the in-plane loading to the masonry infill will affect its capacity to withstand out-plane failure.

Case 2: Surrounding RC columns failure before the failure of masonry infill: The F-index of the surrounding frame is calculated based on the 3 cases mentioned previously in 3.4.2 and figure 7,8& 9. The minimum of the 3 cases should be considered the F-index of surrounding columns. The masonry infill F-index in this case is chosen to be the same as surrounding RC column since as soon the frame fails then masonry infill is not confined anymore and fails as well.

In the case of partial infill panel such as cases in figure 5 and figure 6. The F-index of RC column should be reduced using the reduced height due to partial infill.

5. Conclusion:

a) Methods to calculate in-plane lateral strength of masonry infilled panel by different researchers are viewed and compared with the experimental data. Liauw [6] and Flanagan [7] methods overestimates the strength by an average of 1.25 and 1.58 respectively. FEMA 306[3] equation for compression failure and of Stafford-Smith & Coull[5] slightly underestimates the masonry lateral strength with an average ratio of analytical to experimental ratio of 0.83 and 0.82.

b) These methods look quite complex and requires a detailed data of material properties such as elasticity of masonry infill which might be difficult to acquire in a first level seismic evaluation. A simpler equation with fairly good accuracy based on experimental data is proposed to calculated the in-plane shear capacity.

c) Then, the reduction of lateral strength due to openings in masonry infill panel is reviewed from different past experiments and compared to empirical methods proposed by different researchers based on ratio of opening area to masonry panel and length of openings. (NZSEE) [8] is the most conservative with and average ratio of analytical reduction factor λ_{op} to experimental reduction factor λ_{op} of 0.62. Al-Chaar [21] Eq. (16) showed good correlation with experimental results with an average ratio of analytical to experimental of 0.92 and standard deviation of 0.14.

d) The Japanese seismic evaluation method is a practical seismic evaluation method. However, procedure to calculate the masonry walls strength and ductility are not mentioned in the standard because masonry walls are not a common practice in Japan. Procedure to address the calculation of C strength index masonry infill walls and their surrounding framed based on minimum of different cases of failure mechanisms is proposed.

f) Ductility of RC frame with masonry infill is reviewed based on previous experimental data. As for Drift angle (Rmax) at Peak strength, based on the studied experiments, the Rmax drift has an average of 0.64% and most of values in the range of 0.4%~0.9%. As for Ru drift angle (drift angle when strength is degraded to 80% of peak strength) has an average of 1.6%. The data of Ru drift angle is of a wide range with standard deviation of 0.68.

e) The ductility F-index of masonry infill needs further study and the available data of the parameters affecting the masonry infill deformation is considered not enough. However, in this study, calculation of the F-index of masonry infill is recommended to be taken as the minimum of different failure mechanism.

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