Structural Analysis of
Typical Confined Masonry Structure

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SUMMARY:
A numerical study was conducted to evaluate the performance of one-story building of masonry walls confined with reinforced concrete frames under earthquake loads. In this study, nonlinear beam-column element is used to model RC members. Isotropic elastic shell element is used to model masonry walls before crack forms in the wall. After masonry wall cracks, the structural response becomes nonlinear and nonlinear strut element is used to model the wall. To simulate different loading stages and possible collapse mechanisms, three finite element models were developed. The approach using shell elements to model pre-crack wall panels combined with using nonlinear strut elements to model cracked wall panels seems to be able to simulate the actual condition of the structure. Good agreement was found between the numerical predictions and experimental results.

Keywords: confined masonry wall, numerical study, residential building, pushover analysis

1. INTRODUCTION

The confined masonry structure is formed by two main structural element types, that is, reinforced concrete (RC) elements and masonry wall panels. In conventional structural analysis, the contribution of masonry infill panels to stiffness and strength of the structure is often neglected. Masonry infill is only considered as an architectural and load element. The main reason for this approach is the absence of realistic and simple analytical model for the masonry element. Such an assumption may lead to substantial inaccuracy in predicting the lateral stiffness, strength, and ductility of the structure. It will also lead to uneconomical design of the frame since the strength and stiffness demand on the frame could be largely reduced if the strength and stiffness contribution of the masonry panel is considered.

In this study, structural models for both RC element and masonry panel were included. Nonlinear beam-column element is used to model the RC members. Isotropic elastic shell element is used to model the masonry wall for analysis in the linear condition before crack forms in the masonry wall. After masonry wall cracks, the structural response becomes nonlinear and strut element is used to model the masonry wall. Nonlinear pushover analyses are performed for the models.

The analysis results by numerical method are compared with experimental results reported in a companion paper (Kusumastuti, et al. 2012). A pushover analysis based on assumption that all masonry wall panels in the house model crack and develop compression strut were performed before the experimental work. The results of this analysis were used to estimate the ultimate load and displacement prior to the experiment. During the experiment, it was observed that not all masonry wall panels cracked. Therefore the analysis was refined with strut model applied only to panels that actually cracked and formed compression strut, while isotropic elastic shell element is used to model the remaining masonry wall panels.

2. STRUCTURAL ELEMENT MODEL

In this study, linear and nonlinear models for both RC members and masonry panels were included.

2.1. Reinforced Concrete(RC) Element Model

The RC elements are modelled as beam-column element with possible plastic hinge formation on element ends. The element stiffness and plastic hinge properties are based on actual concrete dimensions and material properties.

2.2. Masonry Element Model

At low lateral load and small lateral deformation, the masonry panel and RC frame act as monolithic composite structural element. As the lateral deformation increase, the masonry panel crack and form various possible failure mechanism. There are several failure modes for infill masonry wall as follows (Tomaževič 1999):

- Sliding shear failure of masonry walls
- Compression failure of diagonal strut
- Diagonal tensile cracking. This is not a general failure. Higher lateral forces can be supported by the above failure modes.
- Tension failure mode (flexural), which is not usually a critical failure mode for infill walls

Linear isotropic shell element is used to model the masonry wall before crack. This model is useful to study stress distribution in the wall panel before crack and to estimate the form and distribution of the compression struts.

On the basis of comprehensive experimental research, various models have proposed for masonry panel. Classical finite element model based on theory of elasticity can be used for prediction of linear behavior of the masonry panel before crack. After crack formation, nonlinear finite element with crack model can be used, however the analysis become very complicated. Based on experimental observation of confined masonry response after extensive crack formation, diagonally braced frame element has been proposed to model the confined masonry panel. The masonry panel is modelled as bracing or strut element. Some variations are proposed on how to assign the properties of the strut based on the actual dimensions and material properties of the masonry. The analysis model for masonry panel applied in this study is based on (Mostafaei and Kabeyasawa 2004). In this study, the masonry is modelled as strut with nonlinear force-displacement curve as shown in Fig. 2.1. The actual dimension and material properties of the masonry is used to determine the parameters of the force-displacement curves as presented in the following sections.

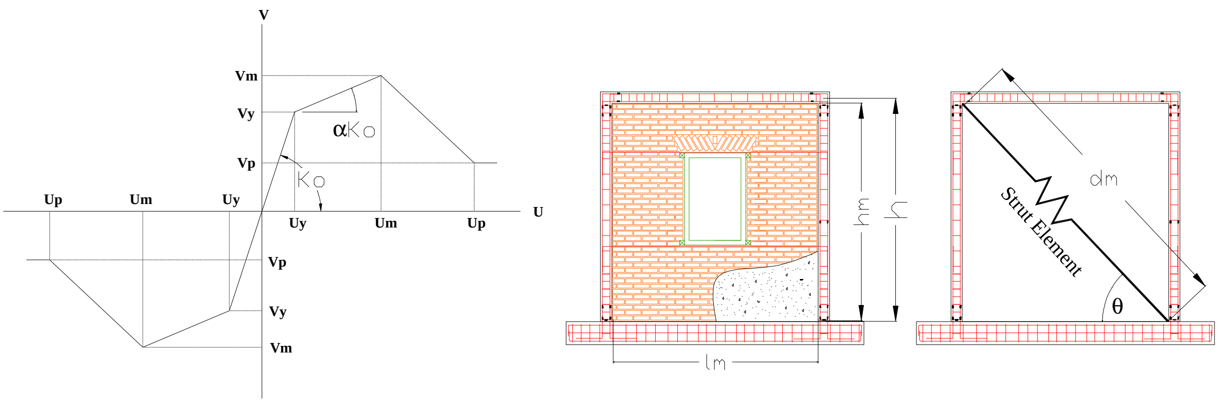


Figure 2.1. Masonry infill panel modelled as strut element

2.2.1. Shear strength of infill masonry wall (Vm)

The shear strength of masonry wall (V_m) is the minimum strength based on various possible failure modes of the masonry infill. Two failure modes are the most common, that is, diagonal compression failures and sliding shear. V_m is determined as the minimum strength according to these failure modes.

a. Shear strength based on diagonal compression failure

The compression strength of masonry wall can be calculated based empirical equation or base on compression test of masonry units. In this study, empirical equation by Eurocode 6 as presented in Eqn. 2.1 is used.

$$f'_m = K \cdot f_m^{0.65} \cdot f_{cm}^{0.25} \quad \text{(MPa)} \tag{2.1}$$

where,

f'_m = compression strength of masonry wall
f_m = compression strength of brick
f_{cm} = compression strength of mortar
K = constant for masonry wall, taken as 0.6 MPa0.1
provided that $f_{cm} \leq 20\,\text{MPa}$ and $f_{cm} \leq 2f_m$

Shear strength is computed from horizontal component of the diagonal compression strut

$$V_c = z\,t\,f'_m \cos(\theta) \tag{2.2}$$

where,

z is equivalent strut width estimated by the following equation given in FEMA 306 (1998):

$$z = 0.175(\lambda h)^{-0.4} d_m \text{ with } \lambda = \left[\frac{E_m t \sin(2\theta)}{4 E_c I_g h_m}\right]^{\frac{1}{4}}$$

h = column height between centerlines of beams
h_m = height of infill panel
E_c = expected modulus of elasticity of frame material
E_m = expected modulus of elasticity of infill material = 750f ‘m
I_g = moment inertia of column
d_m = diagonal length of infill panel
t = thickness of infill panel

b. Shear strength based on sliding shear failure

The maximum shear strength based on the Mohr-Coulomb criteria:

$$\tau_f = \tau_0 + \mu \sigma_n \tag{2.3}$$

where,

τ_0 = cohesive capacity of the mortar beds
μ = sliding friction coefficient along the bed joint
σ_n = vertical compression stress in the infill walls

Maximum horizontal shear force is as follows:

$$V_f = \tau_0\, t\, l_m + \mu N \tag{2.4}$$

where,

t = infill wall thickness
l_m = length of infill panel
N = vertical load in infill walls

In this study, N is estimated directly as a summation of applied external vertical load on the panel and the vertical component of the diagonal compression force R_C. The external vertical load is zero for the infill walls of the building, and only the vertical component of the strut compression force is considered. Therefore, maximum shear force can be calculated as:

$$R_c \cos(\theta) = \tau_0 \, t \, l_m + \mu \, R_c \sin(\theta) \qquad \text{or:} \tag{2.5}$$

$$V_f = \frac{\tau_0 \, t \, l_m}{(1 - \mu \tan(\theta))} \tag{2.6}$$

use $\tau_0 = 0.04 f'_m$; $\mu = 0.654 + 0.000515 f_{cm}$ and $\tan(\theta) = \frac{h_m}{l_m}$

c. Maximum shear strength
According to ACI 530-88, the maximum shear strength of confined masonry walls is

$$V_{max} / t \, l_m = 8.3 \, kg / cm^2 \tag{2.7}$$

The shear strength V_m used in the analyses is the minimum value from a, b and c above.

2.2.2. Maximum displacement and initial stiffness
The displacement at maximum load can be estimated by Eqn. 2.8 (Madan, et al. 1997):

$$U_m = \frac{\varepsilon'_m \, d_m}{\cos(\theta)} \tag{2.8}$$

where, $\varepsilon`_m$ is masonry compression strain at the maximum compression stress, $\varepsilon`_m$ = 0.0018. .

The initial stiffness K_0 can be estimated by Eqn. 2.9 (Madan, et al. 1997):

$$K_0 = 2(V_m / U_m) \tag{2.9}$$

The lateral yielding force and V_y and yielding displacement U_y can be computed by considering load-displacement model in Fig. 2.1 as follows:

$$V_y = \frac{V_m - \alpha K_0 \, U_m}{1 - \alpha} \quad \text{and} \quad U_y = \frac{V_y}{K_0} \tag{2.10}$$

The value α is assumed to be 0.2.

The V_p and U_p should be determined considering that the line connecting the peak of the envelope and the point (V_p, U_p) pass through the 80% post peak point. The drift at 80% post peak is estimated at 1%. Assuming $V_p = 0.3V_m$ lead to $U_p = 3.5(0.01h_m\text{-}U_m)$ (Mostafaei and Kabeyasawa 2004).

3. CONFINED MASONRY HOUSE ANALYSIS MODEL

A single story house with 6 x 6 meter^2 plan area was tested in full scale as a prototype of a simple house structure without roof elements as shown in Fig. 3.1. The model was constructed based on the requirements from the guidelines published by the Ministry of Public Works of Indonesia (The Ministry of Public Works, Indonesia 2009). The height of the house model is 3 meter. RC columns were provided at every wall intersections, thus limiting the masonry wall panels into less than 10 m^2. Details of the house model and the experimental test is presented in the in a companion paper (Kusumastuti, et al. 2012). The analyses were performed for the tested house model so that the analysis results can be verified with the experimental results.

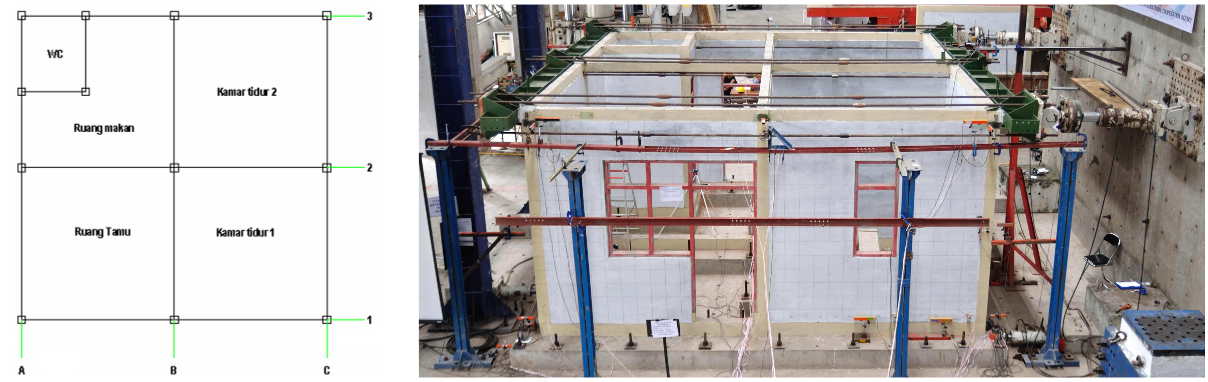


Figure 3.1. Plan view and photo of the masonry house model

3.1. Material Properties

The material properties used in the analysis models are based on material test from the experimental work as shown in Table 3.1.

Table 3.1. Material properties

Material	Properties
Steel rebar	Longitudinal: diameter 9.8 mm, yield stress f_y: 355.4 MPa
	Transversal: diameter 7.6 mm, yield stress f_y: 335.9 MPa
Concrete	Mixture by volume proportions 1:2:3 (cement : sand : aggregate)
	Compressive strength 19 MPa
Brick	Compressive strength 3.8 MPa
Mortar	Mixture by volume proportions 1:4 (cement : sand)
	Compressive strength 19.4 MPa

3.2. Analysis Models

As shown in Fig. 3.1, the house model is not symmetrical. The front wall contains windows and door openings so that the stiffness of the front wall is lower compared to the back wall. The stiffness of the back wall also increased by rest room walls located in the back of the house. The lack of symmetry may cause non uniform damage distributions and formation of compression struts may not occur at the same time for all wall panels. Since the formation of compression strut cannot be predicted easily, analyses are performed based on two finite element models as follows:

3.2.1. Model A: All strut elements for wall panels

Model A is based on assumption that all masonry panels crack and form diagonal compression strut before the house collapse. In this model RC elements are modelled as nonlinear beam-column as presented in Section 2.1. Masonry panels are modelled as strut element as presented in Section 2.2. Model A is shown in Fig. 3.2(a).

3.2.2. Model B: Shell and strut elements for wall panels

From the cyclic load experiment, it is observed that not all masonry panels cracked and form compression strut. Model B is developed with strut element distribution according to the actual strut formation in cyclic load experiment. In this model, RC members are modelled as nonlinear beam-column as described in Section 2.1. The masonry panels in front and middle grids are modelled as strut element as described in Section 2.2, while the rest of the masonry panels are modelled as elastic linear shell element. Model B is shown in Fig. 3.2(b).

3.3. Masonry Strut Properties and Load Displacement-Curve Parameters

Struts properties used in the analysis models are based on actual dimensions of the house model from the experimental work as shown in Table 3.2. A strut is defined for each masonry panel surrounded by

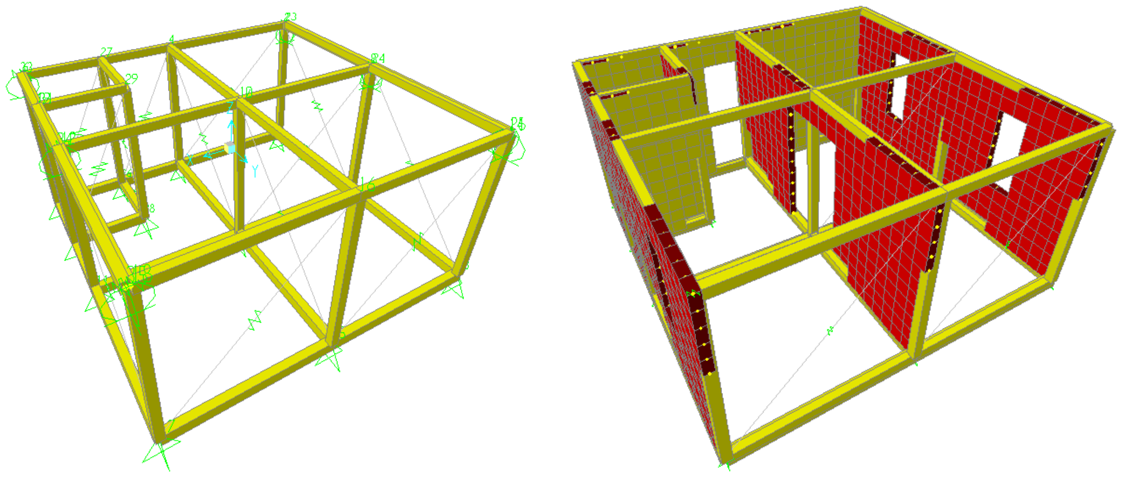
RC members. Therefore, based on column spacing of the house model, the house model has two strut types. Type I represent panels with 3 meter column spacing, while Type II represent panels with 1.5 meter column spacing. Strut load-displacement curve parameters are computed based on the method presented in Section 2.2. The load-displacement curve parameters for strut types are presented in Table 3.3.

Table 3.2. Strut properties

Parameters	Strut Type I	Strut Type II
h_m = height of infill panel (cm)	285.0	285.0
l_m = length of infill panel (cm)	285.0	135.0
d_m = diagonal length of infill panel (cm)	403.1	315.4
t = thickness of infill panel (cm)	14.0	14.0
θ = theta (degree)	45.0	64.7

Table 3.3. Strut load-displacement curve parameters

Parameters	Strut Type I	Strut Type II
V_m (N)	1.1×10^5	8.9×10^4
U_m(mm)	14.5	31.0
V_y(N)	8.3×10^4	6.6×10^4
U_y(mm)	5.4	11.6
V_p(N)	3.3×10^4	2.7×10^4
U_p(mm)	90.3	124.6



(a) Model A (b) Model B

Figure 3.2. Two analysis models of the house

4. ANALYSIS METHOD

The analyses performed were nonlinear pushover analyses. As in the cyclic load experimentation, the lateral loads were applied at the top side corners of the front row (Grid 1) and back row (Grid 3) walls. No direct lateral load was applied to the middle row (Grid 2) wall. The loads on Grid 1 and Grid 3 were incremented at the same rate.

The pushover analysis method is selected due to its ability to compare the performance of the walls from the experiments with the ones from numerical models. Moreover, the pushover analysis method is able to produce important parameters such as maximum capacity and maximum displacement. Given that the cyclic load experiment is a quasi-static test with a low rate velocity, the pushover analyses will produce results with satisfied accuracy. The load-displacement curves from pushover analyses are evaluated and compared with the envelopes of the hysteretic curve recorded during cyclic load experiment.

5. ANALYSIS RESULTS and COMPARISON with EXPERIMENTAL RESULTS

The analysis results from Model A and Model B, in term of load-displacement curves, are compared with experimental results in Fig. 5.1 and Table 5.1. Load-displacement are predicted and measured for back and front walls of the house. Load-displacement curves for front and back walls differ significantly due to significantly different stiffness of the two sides of the house.

5.1. Model A Analysis Results

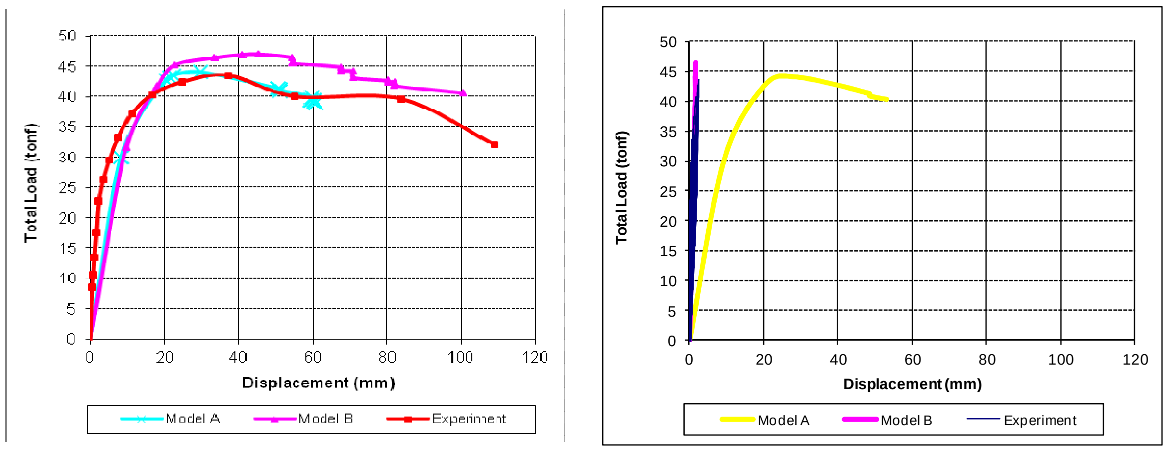
Fig. 5.1 shows the maximum load that can be resisted by front wall (Grid 1) based on Model A is 44.0 tons, which occurred at a displacement of 29.6 mm, or a drift of 0.95%. The maximum displacement, defined as displacement at 80% of the maximum load, is 60.9 mm or equal to 1.95% drift. Fig. 5.1 also shows that the back wall (Grid 3) has much higher stiffness compared to front wall (Grid 1), however the strength of the two sides are very similar.

5.2. Model B Analysis Results

Fig. 5.1 shows the maximum load that can be resisted by front wall (Grid 1) based on Model B is 47.0 tons, which occurred at a displacement of 45.2 mm, or a drift of 1.4%. The maximum displacement, defined as displacement at 80% of the maximum load, is 100.3 mm or equal to 3.2% drift. Figure 5.1 also shows that the back wall (Grid 3) has much higher stiffness compared to front wall (Grid 1) with very small displacement until the front wall collapse and the analysis cannot be continued. By comparison of Fig. 5.1(a) and Fig. 5.1(b), it can be observed that the roof beams do not have adequate strength and stiffness to redistribute the load from front wall to back wall.

5.3. Comparison of Experimental and Analysis Results

From the envelope of the hysteretic curve recorded during cyclic load experiment, it is observed that the maximum load that can be resisted by the house model experimental is 43.5 tonfs, which occurred at a displacement of 37.3 mm, or a drift of 1.2%. The maximum displacement, defined as displacement at 80% of the maximum load, is 100.0 mm or equal to 3.2% drift. The hysteretic envelopes from the experiment and load-displacement curves from the analyses are compared in Fig. 5.1(a) and Fig. 5.1(b) for front wall (Grid 1) and back wall (Grid 3), respectively. The governing parameters of the curves are summarized and compared in Table 5.1.



(a) Front wall (Grid 1)

(b) Back wall (Grid 3)

Figure 5.1. Comparison of experimental and analysis results

As indicated from Table 5.1, the results from the analyses show that Model B is fairly accurate in predicting the structural parameters compared to Model A. Although Model B gives higher maximum load compared to the actual parameters of the structure, Model B has a better prediction of the inelastic condition of the walls, especially for displacement. The approach using shell elements to model the wall elements that do not have damage seems to be able to simulate the actual condition of the structure.

Table 5.1. Comparison of maximum load and displacement of front wall (Grid 1)

Parameters	Experimental	Model A	Model B
Maximum load (tonfs)	43.5	44.0	47.0
Displacement at max load (mm)	37.3	29.6	45.2
Drift at max load (mm)	1.2%	0.95%	1.4%
Displ. at 80% max load (mm) (maximum displacement)	100.0	60.9	100.3
Drift at 80% max load (mm) (maximum drift)	3.2%	1.95%	3.2%

Model A can be used for structural analysis where the actual strut formation cannot be estimated accurately since the maximum load capacity give very close value compared to experimental result. For inelastic deformation capacity, Model A gives conservative values compared to experimental results.

6. CONCLUDING REMARKS

Numerical analyses for confined masonry structure that represents a typical one story house were conducted to study the response of the structure loaded laterally until collapse. In parallel, an experimental work was performed by applying cyclic lateral load to a full scale one story typical house.

The structural model consists of RC members and masonry wall panels. Nonlinear beam-column element is used to model the RC members. Isotropic elastic shell element is used to model the masonry wall for analysis in the linear condition before crack forms in the masonry wall. After masonry wall cracks, the structural response becomes nonlinear and nonlinear strut element is used to model the masonry wall. Nonlinear pushover analyses are performed for the models and the analysis results are compared with experimental results.

The numerical models were developed using material properties obtained from the material testing. Structural dimensions also closely follow the actual dimensions of the house model tested in the experimental work. Before experimental work, analysis was performed by assuming that all wall panels form strut before collapse of the house. From the experiment, it was observed that not all masonry wall panels cracked. Therefore the analysis was refined with strut model applied only to panels that actually cracked and formed compression strut.

Comparison of experimental and analysis results show that the envelope of the hysteretic curve recorded during cyclic load experiment is in good agreement with the pushover curves generated from analysis. The analyses show that the refined model gives a better prediction of the structural performance, especially in the inelastic range.

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