AN ANALYTICAL MODEL FOR CRACKING LOAD AND DISPLACEMENT OF CONFINED MASONRY WING-WALLS

Y. H. Tu[1], H. Y. Yeh[2] and T. H. Chuang[3]
[1] Associate Professor, Department of Architecture, National Cheng Kung University, Tainan, Taiwan, yhtu@mail.ncku.edu.tw
[2] Master of Science, Department of Architecture, National Cheng Kung University, Taiwan
[3] PhD Candidate, Department of Architecture, National Cheng Kung University, Taiwan

ABSTRACT
An analytical model for the seismic behavior of confined masonry (CM) wing-walls is proposed in this paper. CM wing-walls are the panel segments between openings and RC tie-columns. Based on the experimental results for seven full-scale CM wing-walls tested by the authors, the model describes a bilinear relationship of the load-displacement envelope curve. This paper focuses on the elastic section defined by the cracking load and displacement in the envelope curve and presents the rationale in setting up the analytical factors. The cracking load is estimated with the diagonal tension strength according to FEMA 356. The cracking displacement is estimated by considering CM wing-walls as diagonal compression struts. The compression strut analogy indicates that CM walls are subjected to additional axial compression induced by the lateral loading. Since axial stress plays an important part in the diagonal tension strength estimation, this paper proposes a method to derive the additional axial force from the lateral load. The distribution of lateral force between the masonry wall and tie-column is also examined, so the contribution of tie-columns can be included in the strength estimation. The proposed model has been validated with the experimental data. The model shows a conservative estimation for both cracking load and displacement, but the comparison with experimental load-displacement curves indicates a good fit.

KEYWORDS: confined masonry, seismic behavior, lateral strength, load-displacement relationship

INTRODUCTION
Confined masonry (CM) consists of masonry wall panels and cast-in-place reinforced concrete (RC) confining elements. The confining elements, including tie-columns and tie-beams, are built after the masonry panels. The vertical edges of the panels adjacent to tie columns are usually toothed as shear keys to integrate the masonry panels and tie columns into composite members. CM buildings are widely used in South Europe, Latin America, and Asia [1]. The confining elements usually have smaller cross-sectional dimensions than the beams and columns in an RC frame building [2]. The seismic design guide for low-rise confined masonry buildings [3] suggests a minimum tie-column/beam size of 150mm t with minimum 4-#3 reinforcing bars, where t denotes the wall thickness. CM buildings have also been constructed in Taiwan for decades. However, the differences between CM and RC buildings were not well known until recently. Local engineers tend to use CM for masonry panels in an RC frame building. In this case, the column and beam sections have larger dimensions, and also fail to satisfy the

requirement for standard CM buildings [3] that there should be confining members around the openings, as shown in Figure 1. The wall segments between openings and columns in Figure 1 are called "wing-walls". Despite the lack of confinement at the opening side, the wing-wall increases the stiffness and changes the behavior of the adjacent column.

Figure 1: Common types of CM wing-walls in Taiwan: a) Single Wing-Wall; b) Twin Wing-Wall

The behavior of regular CM walls has been studied experimentally by many researchers. Tomaževič and Klemenc [4][5] performed a static cyclic loading test of 1/5 scaled CM panels and a shaking table test of 1/5 scaled CM buildings. Kumazawa and Ohkubo [6] investigated the effects of lateral reinforcement placed in bed joints at the corners of the panel and anchored in the tie-columns. Yoshimura et al. [7] tested twenty CM wall specimens with different reinforcing methods for the tie-columns, wall panels, and wall-to-column connections. Sivarama Sarma et al. [8] studied the effects of the number of tie-columns and openings with ten specimens made of hollow blocks and solid bricks. Marinilli and Castilla [9] also studied the effects of the number of vertical confining elements. However, few studies have examined CM walls without confinement around the openings. Yáñez et al. [10] investigated the behavior of sixteen CM specimens with openings of different shapes, and while there were no confining members around the openings, slight reinforcement was used instead. It was concluded that diagonal struts can be developed in specimens with openings and work with the confining columns as virtual strut-and-tie models. Eshghi and Pourazin [11] used finite element models to study the behavior of CM walls with openings and compared it with experimental data, with the analytical model showing that compressive struts form between the opening corners and the specimen corners.

The characteristics of the wing-walls are close to those of CM walls with openings. Because of the lack of earlier research, the authors performed a series of static loading tests for the wing-walls to study their in-plane behavior [12][13]. Based on the results of these, an analytical model is proposed that can be used for seismic assessment of CM buildings with wing-walls. The model describes a bilinear relationship of the load-displacement envelope curve. This paper focuses on the cracking load and initial stiffness that define the elastic section.

BRIEF INTRODUCTION OF THE TEST

Seven full-scale CM wing-wall specimens were tested, five of them with monotonically increasing loading and the other two with cyclic loading. The lateral loading was applied by two hydraulic actuators through an L-shaped steel member to keep the top beams from rotating. An axial load of 313.6 kN, which represents the dead load in the base floor of a four-floor building was applied to each specimen with a jack fixed at the midline of each specimen top and connected to sliding reaction supports during the test. The test parameters include the position of the tie-column and the length of the wall panels. Figure 2 and Table 1 shows the basic details of the specimens, where f'_c is the concrete compressive strength, f_y and f_{yh} are the yielding strength of longitudinal and transverse reinforcement of the tie column. The compressive strength of masonry prism f'_m was obtained from double-wythe-eight-layer prisms, as shown in Figure 3. The shear strength of the brick-mortar interface f_{mbt} was obtained from triplet tests of three-layer prisms. All the masonry panels were 2700mm high and 200mm thick, made of 200mm × 95mm × 53mm solid clay bricks. Identical tie-column sections with a depth of 300mm and a width of 400mm were used in all specimens. The reinforcement in the tie-column included eight #6 reinforcing bars and #3 hoops with a spacing of 250 mm. Both the dimensions and reinforcement were based on typical low-rise school buildings in Taiwan.

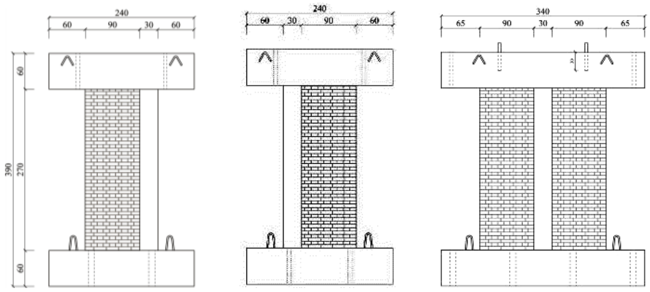


Figure 2: Types of the wing-wall specimen: a) Single Wing-Wall; b) Reversed Single Wing-Wall; c) Twin Wing-Wall

Figure 3: Masonry material test specimens: a) Prism; b) Triplet

Table 1: Details of the Wing-Wall Specimens

Specimen	Type	Masonry Panel (length* × number)	Loading	f'_c (MPa)	f_y (MPa)	f_{yh} (MPa)	f'_m (MPa)	f_{mbt} (MPa)
A	Single	900 × 1	Monotonic	29.9	439.2	318.9	17.4	1.54
B	Twin	900 × 2	Monotonic	30.9	439.2	318.9	17.4	1.54
C	Reversed single	900 × 1	Monotonic	30.7	490.5	400.3	19.4	1.35
AC	Single	900 × 1	Cyclic	32.2	490.5	400.3	19.4	1.35
BC	Twin	900 × 2	Cyclic	30.7	490.5	400.3	19.4	1.35
AL	Single	1200 × 1	Monotonic	36.2	490.5	400.3	19.4	1.35
BS	Twin	600 × 2	Monotonic	33.4	490.5	400.3	19.4	1.35

*: unit = mm

CRACKING BEHAVIOR OF THE TEST SPECIMENS

The first cracks that appeared on the specimens were the tensile cracks on the tie-columns. After this, inclined cracks showed on the masonry panel as the lateral load increased, and these lay approximately along the diagonal of the entire specimen but did not penetrate the tie-column. On most of the specimens, shear cracks were found on the tie-columns at the position where the diagonal cracks terminated. Small inclined cracks were observed along the vertical edge between the masonry panel and the tie column on specimens C and BS. Two of the specimens (A & BS) reached maximum strength when the diagonal cracks appeared. Specimens AC and AL had diagonal cracks that appeared at a relatively low loading, due to the presence of an unexpected vertical cracks close to the tie-column that formed before the test. The cause of the vertical cracks is not clear, although they were probably due to the uneven stress that was applied to the specimens during the setup of the test devices. The vertical cracks never show in the other specimens since we modified the test setup steps, so they are considered special cases. Figure 4 shows the cracking pattern of the specimens when the main diagonal cracks were observed.

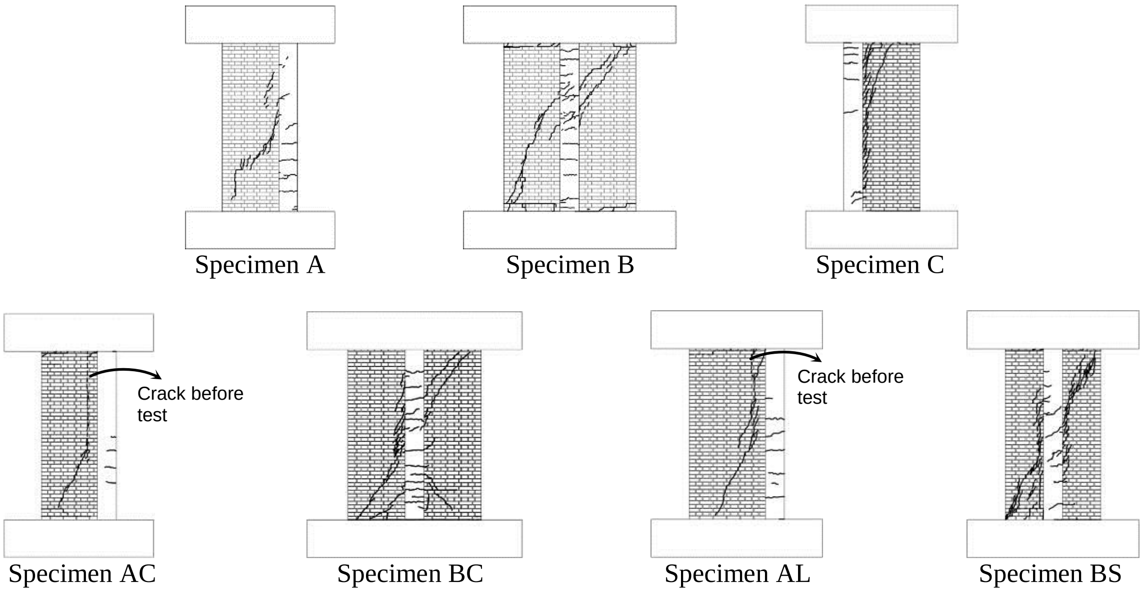


Figure 4: The cracking pattern when the main diagonal cracks were observed

The masonry panels and tie-columns behaved as integrated composite members until the masonry panels were severely damaged. The tensile cracks on the tie-columns indicate that they work as tension ties. The crushing of masonry panel corners also suggests the formation of a diagonal compression strut. According to the seismic design guide for low-rise confined masonry buildings, this "strut and tie" behavior corresponds with the behavior of CM panels with larger confining members [3]. However, the tensile cracks did not go across the whole depth of the tie-columns in all the specimens except for specimen AL. This indicates that part of the tie-column section was subjected to compression, and the readings of strain gauges on the reinforcing bars in tie-columns confirmed this. This phenomenon was more obvious in specimens with shorter panels. For the single wing-wall specimens, the strain readings and cracking patterns show that on the end where the panel is on the tension side, the tie-column carries the fixed-end moment alone. It also indicates that the inflection points of the single wing-walls are not at the mid-height but close to the end where the panel is on the tension side. The member ends become semi-rigid but not completely fixed ends after flexural cracking. The position change of inflection points might be caused by the asymmetric boundary conditions.

ANALYTICAL MODEL FOR LOAD-DISPLACEMENT RELATIONSHIP

Tomaževič and Klemenc [4] proposed a trilinear model for the load-displacement relationship of CM panels. The model is defined by three characteristic points: elastic (crack) limit, maximum resistance, and ultimate state. Bourzam et al. [14] adopted this concept, and developed an analytical method to calculate the shear capacity. Riahi et al. [15] proposed a similar backbone model defined by cracking, maximum strength, and ultimate deformation capacity. The trilinear and backbone models are easy to use in seismic assessment but they are both developed for regular CM panels with full confinement. Therefore, another model for wing-walls is proposed in this paper. This model follows the basic concept of the trilinear and backbone models, but the final section is omitted. The wing-walls are not supposed to provide enough deformation capacity due to the insufficient confinement. Figure 5 shows the proposed analytical model. A bilinear load-displacement relationship is determined by the cracking (C) and the maximum strength (U). The cracking is defined by the initial stiffness k_i and the cracking resistance Q_{cr}.

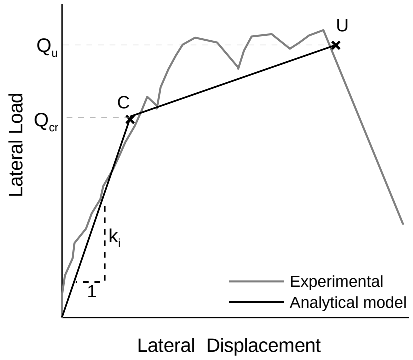


Figure 5: Analytical model for the load-displacement relationship of wing-walls

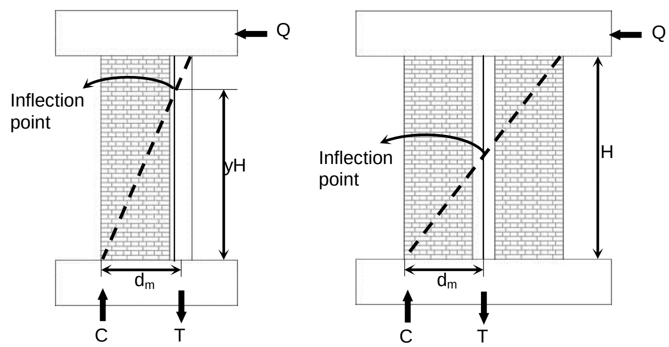


Figure 6: Relationship between the axial stress and the lateral force: a) Single Wing-Wall; b) Twin Wing-Wall

ADDITIONAL COMPRESSION IN THE CM WING-WALLS

The strength of masonry panels is highly affected by the axial stress. In accordance with Riahi et al. [15], most of the analytical equations for evaluating the cracking strength of a CM panel depend on the masonry shear strength and the axial stress. The axial stress is not only due to the dead load. As reported by other researchers [3][4][14] and observed in the presented test, additional compression in the masonry panel is induced due to the interaction between the panel and the tie-column. A simplified method to approximately evaluate the additional compression is proposed as follows.

Considering the CM panel as a composite member, the bending moment caused by the lateral load results in axial stress at the fixed ends. As shown in Figure 6 and Equation 1, the resultant axial compression *C* applied on the masonry panel can be approximately calculated by the equilibrium between the force couple and the fixed-end moment. In Equation 1, *Q* is the lateral load, d_m is the arm of the couple, and *yH* is the distance from the fixed end to the inflection point. In this paper, d_m is conservatively assumed to be the distance from the center of the tie-column to the outer edge of the masonry panel on the compression side. *yH* is assumed to be half of the clear height ($yH = 0.5H$) of the panel for the twin wing-walls, since they have identical boundary conditions on the top and bottom ends. For the single wing-walls, a compression strut is assumed to lie on the diagonal and *yH* is assumed to be the height of the intersection of the diagonal and the steel which is the closest to the masonry panel.

$$C = Q \times yH / d_m \quad (1)$$

The resultant axial compression *C* happens at the fixed end, but the observed diagonal cracks distribute between the inflection point and the member end, so a position factor $\gamma = 0.5$ is introduced to reduce the axial compression. Another factor β is used to consider the axial compression shared by the column sections when the panel is too short. As shown in Figure 7, the neutral axis of the CM wing-wall as a composite section might lie inside the column when the panel is short. The internal compression is therefore not only carried by the panel (C_m) but also shared by the concrete (C_c) and steel (C_s) in the tie-column. The axial compression share factor β means the ratio of compression carried by the panel to the total compression. β can be calculated by assuming that plane remains plane and linear stress-strain relationships theoretically. At first, we used $E_c = 4700\sqrt{f'_c}$ (MPa) and $E_m = 550 f'_m$ for the elastic modulus of concrete and masonry in accordance with ACI 318-08 [16] and FEMA 356 [17], respectively. However, the calculated position of neutral axis was within the panel for all the specimens. In order to make the calculated result consistent with the experimental observations, a lower E_m determined by trial and error is used. The relationship between the ratio of panel length W to the tie-column depth h and β is then developed as shown in Equation 2 and Figure 8. It should be noted that the masonry panel on tension side is assumed to be inefficient, so only a single side is considered for the twin wing-walls.

$$\beta = 0.03\left(\frac{W}{h}\right)^2 + 0.1\left(\frac{W}{h}\right) \quad , \quad 0 \le \beta \le 1$$
(2)

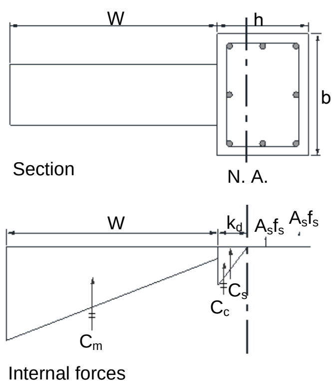


Figure 7: Internal forces in a CM wing-wall as a composite section

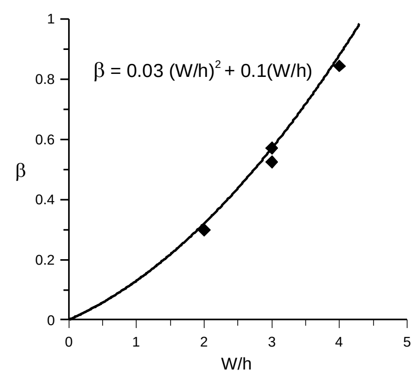


Figure 8: The axial compression share factor β

Summarizing the above, the additional axial compression ΔN in the masonry panel of CM wing-walls is calculated by Equation 3. The total axial force N_m applied to every single side of the masonry panels is the summation of ΔN and the original axial force N_{mEA}. The original axial force is the vertical force caused by the dead load and distributed to the masonry panel and the tie-column with the proportion of the product of the elastic modulus and section area (EA).

$$\Delta N = C \cdot \gamma \cdot \beta$$
(3)

CRACKING LOAD OF THE CM WING-WALL

The cracking load of a CM wing-wall Q_{cr} is proposed to be the summation of the diagonal cracking strength of the masonry panel Q_{mc} and the shear contribution of the tie-column Q_{cc}, as shown in Equation 4.

$$Q_{cr} = Q_{mc} + Q_{cc}$$
(4)

The diagonal cracking is caused by diagonal tension. Most earlier research suggests that the shear reinforcement in the tie-columns has no significant contribution to the diagonal cracking resistance in accordance with Riahi et al. [15]. Therefore, we use the equations for diagonal tension strength V_{dt}, that were proposed in FEMA 356 to estimate the cracking strength of the masonry panels, although they were originally used for unreinforced masonry (URM) structures. These are shown in Equations 5 and 6, and some modifications are suggested below to make them more suitable for use with CM wing-walls.

$$Q_{mc} = V_{dt} = f'_{dt} \times A_n \cdot \frac{W_s}{H_{eff}} \sqrt{1 + \frac{f_a}{v_{me}}}$$
(5)

$$f'_{dt} = v_{me} = 0.75 \times \left(0.75 v_{te} + P_{CE} / A_n\right) / 1.5$$
(6)

where A_n (mm^2) is the total section area of the masonry panels and H_{eff} (mm) is the clear height of the wing-wall. W_s (mm) was the panel length in the original equation, but it is suggested that this is replaced by the total length/width including the tie-column so the term W_s / H_{eff} represents the length-to-height ratio of the entire wing-wall. f_a (MPa) is the axial stress of the masonry panel, and it is recommended that this is N_m divided by the section area of single side of the panels. v_{te} (MPa) is the average bed-joint shear strength, and the default value suggested in section 7.3.2.10 of FEMA 356 is used here. P_{CE} is the vertical force applied to the panel, but it is recommended to replace the term P_{CE} / A_n with the axial stress f_a mentioned above.

Considering the CM wing-wall as a composite member, the tie-column is supposed to carry equal shear stress to the masonry panel before cracking. With equal shear stress, the shear forces in the tie-column and the masonry panel are proportional to the product of shear modulus *G* and section area *A*. The appropriate shear force distribution is studied by comparing the theoretically distributed shear force and the observed damage condition of the tie-columns in the specimens. As shown in Table 2, the theoretical shear force in the tie-column is calculated from the experimental cracking strength of each specimen with the *GA* proportion. This means the shear force shared by the tie-column when the diagonal cracking occurs. The shear modulus G is calculated by $G = E / 2(1+\nu)$, where $E_c = 4700\sqrt{f'_c}$ (MPa) and $E_m = 550 f'_m$ are used for the elastic modulus of concrete and masonry, respectively. $\nu = 0.2$ is used for the Poisson's ratio of both concrete and masonry. The theoretical shear cracking and ultimate strengths V_c and V_u are calculated in accordance with ACI 318-08. Table 2 shows that more than half of the specimens have a shear force in the tie-column larger than V_u, indicating the shear failure of the tie-columns when they had just cracked. This means that the theoretical shear force shared by the tie-column with the GA proportion is larger than the actual one. A reduction factor $i = 0.5$ determined by the comparison with experimental results is thus suggested for the use with the GA of the tie-columns. The shear contribution of the tie-column Q_{cc} to the cracking resistance of the CM wing-wall is then calculated by Equation 7.

Table 2: Comparison between the theoretically distributed shear force and the actual damage condition of the tie-columns

Specimen	Experimental cracking strength (kN)	Theoretical shear force in the tie-column (kN)	Shear cracking strength of the tie-column V_c (kN)	Shear ultimate strength of the tie-column V_u (kN)	Theoretical damage condition	Actual damage condition
A	283.3	184.4	102.0	147.47	Fail	Crack
B	475.4	231.3	100.8	146.28	Fail	Crack
C	235.4	148.7	103.4	160.43	Crack	Crack
AC	103.4	65.9	106.0	163.01	-	-
BC	413.5	190.9	100.4	157.44	Fail	Crack
AL	217.7	126.8	111.3	168.32	Crack	-
BS	304.1	174.2	106.7	163.77	Fail	Crack

$$Q_{cc} = i \times \frac{(GA)_c}{(GA)_m} \times Q_{mc} \leq V_c$$

(7)

where $(GA)_c$ and $(GA)_m$ is the product of the shear modulus and the section area of the tie-column and the masonry panel, respectively. It is suggested that the calculated Q_{cc} shall not exceed the shear cracking strength V_c calculated in accordance with ACI 318-08.

Since the additional axial force that is considered in the estimation of the cracking strength of a masonry panel is related to the lateral load, an iterative procedure is needed for the calculation of Q_{cr}. As shown in Figure 9, an additional axial force ΔN is assumed at the beginning, then Q_{mc}, Q_{cc}, and Q_{cr} are calculated based on the assumed ΔN. Another ΔN is obtained by substituting Q_{cr} into the Q in Equation 1, and then checking if this ΔN is close enough to the assumed ΔN. If not, the new ΔN is used for the next iteration of the calculation, and this continues until the calculated strength is convergent.

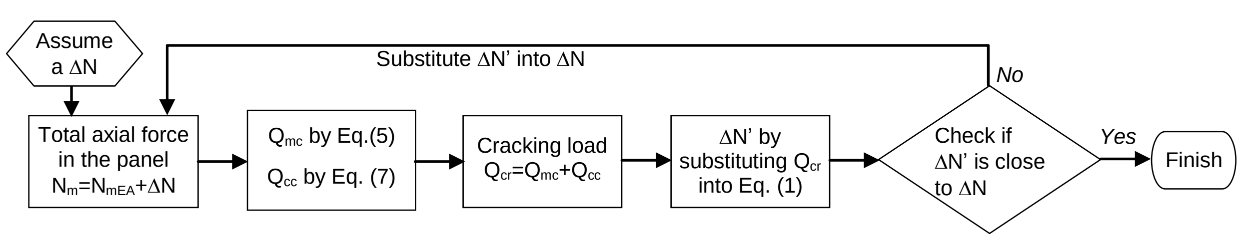


Figure 9: The iterative procedure for calculating the cracking load of the CM wing-walls

INITIAL STIFFNESS OF THE CM WING-WALL

The CM wing-wall is considered to be equivalent to a diagonal strut as shown in Figure 10. When the wing-wall has a lateral deflection Δ from A to A', the diagonal AB is compressed Δ_d to A'B and causes compression P_d in the strut. Through the force equilibrium between lateral load Q and P_d and the relationships between Δ_d and Δ, the initial stiffness is obtained as shown in Equation 8.

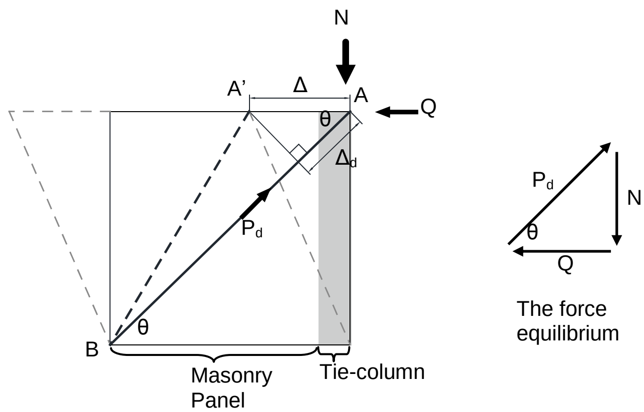


Figure 10: The equivalent diagonal strut in a CM wing-wall

$$k_i = \frac{Q}{\Delta} = \frac{E_m A_T}{L_d} \cdot cos^2\,\theta \qquad (8)$$

where E_m (MPa) is the elastic modulus, L_d (mm) is the length of the diagonal of the entire wing-wall, and θ is the angle from the horizontal to the diagonal. A_T (mm^2) is the section area of the equivalent strut and equal to the product of the thickness of the masonry panel t (mm) and the width of the equivalent strut a (mm). The width of the equivalent strut is calculated in accordance with FEMA 356, as shown in Equations 9 and 10.

$$a = 0.175 \times (\lambda_1 h_{col})^{-0.4} L_d$$

(9)

$$\lambda_1 = \left[\frac{E_m t \cdot sin\,2\theta}{4E_c I_{fe} h_{inf}}\right]^{\frac{1}{4}}$$

(10)

The following modifications are suggested for using for CM wing-walls. $h_{col} = h_{inf} = H_{eff}$ is the height of the wing-wall, and I_{fe} is the moment of inertia of the boundary frame, which is replaced by the moment of inertia of the top beam here.

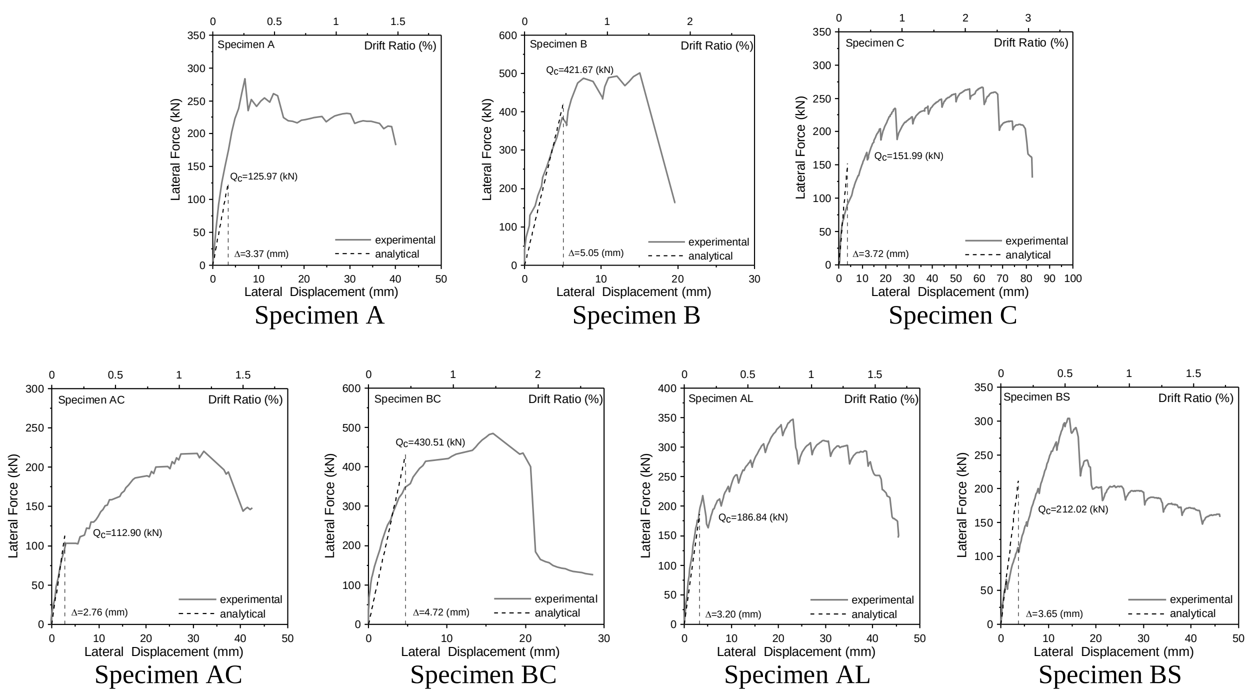
COMPARISON WITH THE EXPERIMENTAL RESULTS

The proposed analytical model is used to estimate the cracking resistance and initial stiffness of the seven specimens, and these are compared with the test results. The analysis is modified for specimens AC and AL to reflect the effects of the unexpected vertical cracks. For the calculation of W_s and A_n in Equation 5, the distance from the vertical crack to the outer edge of the tie-column is subtracted from the length of wing-wall/panel. The comparison between the analytical and experimental results is shown in Table 3 and Figure 11. Table 3 shows conservative estimation for both cracking resistance and initial stiffness for most of the specimens, while the comparison with experimental load-displacement curves shown in Figure 11 indicates a good fit.

Table 3. Comparison between the analytical and experimental results

Specimen	Cracking Load			Initial Stiffness		
	Analytical (kN)	Experimental (kN)	Error* (%)	Analytical (kN/mm)	Experimental (kN/mm)	Error* (%)
A	126.0	283.3	–55.54	37.34	72.09	–48.20
B	421.7	475.4	–11.30	83.50	119.77	–30.28
C	152.0	204.6	–25.72	40.87	29.76	37.33
AC	112.9	103.4	9.16	40.97	38.48	6.47
BC	430.5	413.5	4.12	91.27	196.49	–53.55
AL	186.8	217.7	–14.18	58.36	77.54	–24.74
BS	212.0	304.1	–30.29	58.13	35.38	64.30

* Error = (analytical value–experimental value)/experimental value



Specimen A — Specimen B — Specimen C

Specimen AC — Specimen BC — Specimen AL — Specimen BS

Figure 11: The comparison between analytical and experimental load-displacement curves

CONCLUSION

Analytical models for estimation of the cracking resistance and initial stiffness for the CM wing-walls are presented in this paper. The models are developed on the basis of existing equations and the failing behavior observed in the in-plane loading tests for seven CM wing-wall specimens. The failing behavior of wing-wall specimens indicates that the masonry panel and the tie-column work as a strut-and-tie system and caused additional axial compression in the masonry panel. The effect of the additional axial compression and the contribution of the tie-column are considered in the analytical model for cracking resistance of CM wing-walls. A comparison of the analytical results with the experimental ones shows conservative estimation for both cracking resistance and initial stiffness for most of the specimens. However, the comparison with the experimental load-displacement curves shows a good fit.

REFERENCES

1. EERI & IAEE (2012) the World Housing Encyclopedia, http://www.world-housing.net/
2. Brzev, S. (2007) *Earthquake-Resistant Confined Masonry Construction*, National Information Center of Earthquake Engineering, Kanpur, India.
3. Meli, R and Brzev, S. (2011) *Seismic Design Guide for Low-Rise Confined Masonry Buildings*, Confined Masonry Network, EERI & IAEE.
4. Tomaževič, M. and Klemenc, I. (1997) "Seismic Behaviour of Confined Masonry Walls" *Earthquake Engineering and Structural Dynamics*, Vol. 26, 1059-1071.
5. Tomaževič, M. and Klemenc, I. (1997) "Verification of Seismic Resistance of Confined Masonry Buildings" *Earthquake Engineering and Structural Dynamics*, Vol. 26, 1073-1088.
6. Kumazawa, F. and Ohkubo, M. (2000) "Nonlinear Characteristics of Confined Masonry Wall with Lateral Reinforcement in Mortar Joints" Proceedings of the 12th World Conference on Earthquake Engineering, No. 0743.
7. Yoshimura, K., Kikuchi, K., Kuroki, M., Nonaka, H., Kim, K. T., Matsumoto, Y., Itai, T., Reezang, W., and Ma, L. (2003) "Experimental Study on Reinforcing Methods for Confined Masonry Walls Subjected to Seismic Forces" Proceedings of the 9th North American Masonry Conference, 89-100.
8. Sivarama Sarma, B., Sreenath, H. G., Bhagavan, N. G., Ramachandra Murthy, A., and Vimalanandam, V. (2003) "Experimental Studies on In-Plane Ductility of Confined Masonry Panels" *ACI Structural Journal*, Vol. 100, No. 3, May-June, 330-336.
9. Marinilli, A. and Castilla, E. (2004) "Experimental Evaluation of Confined Masonry Walls with Several Confining-Columns" Proceedings of the 13th World Conference on Earthquake Engineering, No. 2129.
10. Yáñez, F., Astroza, M., Holmberg, A., and Ogaz, O. (2004) "Behavior of Confined Masonry Shear Walls with Large Openings" Proceedings of the 13th World Conference on Earthquake Engineering, No. 3438.
11. Eshghi, S. and Pourazin, K. (2009) "In-Plane Behavior of Confined Masonry Walls – with and without Opening" *International Journal of Civil Engineering*, Vol. 7, No. 1, 49-60.
12. Tu, Y. H., Lo, T. Y., Weng, P. W., Chuang, T. H., Chiou, T. C., and Weng, Y. T. (2009) "Experiment on Lateral Capacity of Slender Confined Masonry Walls" Proceedings of the 11th Taiwan-Korea-Japan Joint Seminar on Earthquake Engineering for Building Structures, 279-288.
13. Tu Y. H., Chuang, T. H., Lin, P. C., Weng, P. W., and Weng, Y. T. (2011) "Experiment of Slender Confined Masonry Panels under Monotonic and Cyclic Loading" Proceedings of the 2011 Structure Congress, 2730-2740.
14. Bourzami, A., Goto, T., and Miyajima, M. (2008) "Shear Capacity Prediction of Confined Masonry Walls Subjected to Cyclic Lateral Loading" *Doboku Gakkai Ronbunshuu A*, Vol. 64, No. 4, 692-704.

15. Riahi, Z., Elwood, K. J., and Alcocer, S. M. (2009) “Backbone Model for Confined Masonry Walls for Performance-Based Seismic Design” *Journal of Structural Engineering*, Vol. 135, No. 6, 644-654.
16. ACI Committee 318 (2008), Building *Code Requirement for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08)*, American Concrete Institute, Farmington Hill, U. S. A.
17. FEMA (2000), *Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA 356)*, Federal Emergency Management Agency, U. S. A.