EFFECT OF A MIDDLE TIE BEAM IN THE SEISMIC BEHAVIOR OF CONFINED MASONRY WALLS

Yuuki Alejandro Hosaka Ventura* **Supervisor: Shunsuke SUGANO****
MEE16716

ABSTRACT

The effect of adding a middle tie beam element to the seismic behavior of confined masonry walls is studied. Equations were proposed based on common failures in walls as a mean to quantify the contribution of such element. Utilizing the calculated strength, the failure process of the specimens is analyzed to understand the difference in behavior of the specimens with a middle tie beam.

Keywords: Confined masonry, middle tie beam, shear strength.

1. INTRODUCTION

In 2001, El Salvador was subjected to 2 seismic events, the January and February 13 earthquakes. Most of the damage caused by these events was concentrated in the housing sector (CEPAL, 2001). In regards to confined masonry in specific, Hasbun et al. (2008) reports that many of the damages on these structures were due to loss of confinement. As such, the objective of this study is to understand and quantify the effect that the middle tie beam brings to the wall (Figure 1). In order to do so, equations were proposed to model typical failure modes in confined masonry wall. Damage states were compared between specimens with and without this element. Finally, utilizing the calculated results the damage progression of the specimens is explained.

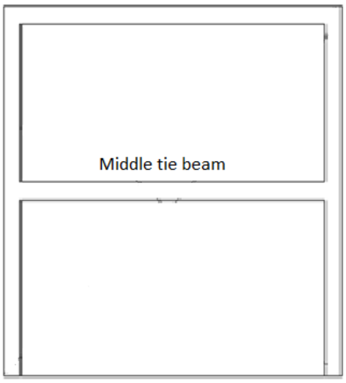

Figure 1. Wall with a middle tie beam.

2. EXPERIMENTAL DATA

2.1. TAISHIN PROJECT test data

Two specimens were tested, one control wall (SPC) and one wall with an additional beam at middle height (SPCI), to investigate the effect a middle tie beam would bring to the behavior of the wall. Both specimens are 3x3 m in length and height. All tie elements had a square cross section (15x15 cm) with the same main reinforcement (4 #3 bars). The material properties and results of test are presented in Table 1.

2.2. BRI study data

*Universidad Centroamericana José Simeón Cañas, El Salvador.
**Professor Emeritus, Hiroshima University, and visiting research fellow, BRI, Japan.

The goal of this study was to investigate the influence of a middle tie beam RC element experimentally. For this, three specimens were tested, one control wall and two walls with a middle tie beam. All specimens had a height and length of 1.33x1.18 m. The brick strength was 4.5 MPa and the mortar 44 MPa. Two diameters of bars were used, D8 and D4, with a yield strength of 312 and 434 MPa, respectively. The concrete used for the tie elements had a compressive strength of 26.9 MPa. The cross section of the tie elements and its main reinforcement is presented in Figure 2. All specimens are judged to have failed in diagonal tension.

Table 1. Material properties of the TAISHIN project specimens.

		SPC	SPCI
WALL	Masonry compressive strength (kg/cm^2)	17.4	17.4
	Mortar compressive strength (kg/cm^2)	229	186
TIE BEAM	Main rebar yield strength (kg/cm^2)	3629	3629
	Concrete strength (kg/cm^2)	247 (top)	174 (middle) 264 (top)
TIE COLUMN	Main rebar yield strength (kg/cm^2)	3629	3629
	Concrete strength (kg/cm^2)	275.5	218.5
TEST RESULTS	Maximum load (kN)	56.0	68.2
	Ultimate distortion (%)	0.2	1.043
	Judged Failure mode	Sliding	Bending

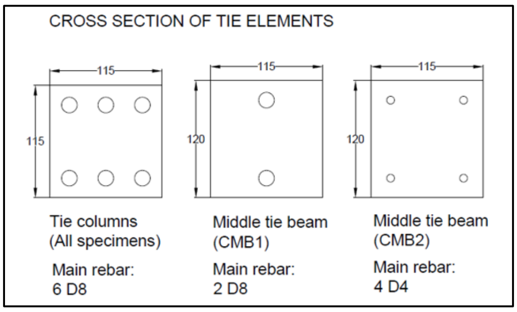

Figure 1. Specimens CMNB, CMB1 and CMB2 and cross sections of tie elements by Goto and Azuhata, 2004.

3. THEORY AND METHODOLOGY

The in plane lateral strength of the wall was calculated for the following failure modes: sliding, diagonal tension, diagonal compression, sliding and bending. The methodology employed consisted in separating each of the elements of the wall (brick panel, tie column and tie beam, if applies) and calculating its contribution; the lateral strength, then, would be the summation of those values.

3.1. Failure modes of confined masonry walls

- **Sliding failure:** This failure mode is characterized by cracks along the mortar bed (horizontal cracks). This failure mode occurs when the shear produced by the lateral load surpasses the strength of the mortar bed.

- **Diagonal tension:** This failure mode occurs when the tension induced by the combination of lateral and vertical loads surpasses the tensile strength of the masonry. It produces diagonal cracks that go from the loading corner to the opposite bottom corner.
- **Diagonal compression:** This failure mode happens when separation of the columns and brick panel occurs in the corners opposite to the loading ones. A compression strut is formed as a resisting mechanism, and crushing of masonry happens in the compression corners.
- **Bending:** It is more common on walls with high H/L ratio. It causes horizontal cracks along the mortar bed, as well as diagonal cracks.

3.2. Equations to evaluate the lateral strength of the wall.

Based on the previous failure modes, equations were proposed to evaluate the in-plane strength of the wall. The overall capacity will be the lowest value among all equations and its corresponding failure mode.

3.2.1. Brick panel

- For sliding, the brick panel contribution is the friction strength of the mortar joints (Crisafulli, 1997).
- The contribution in diagonal tension mode is derived when the principal tensile stress is equal to the tensile strength of the masonry (Crisafulli 1997).
- For diagonal compression, the masonry panel is modeled as a compression strut with the following properties.

3.2.2. Tie columns

For all failure modes these elements are modeled utilizing the dowel effect, which is a shear resisting mechanism. In the study the equation of Tomazevic (Tomazevic, 1999) is utilized.

3.2.3. Middle tie beam

This element is modeled as axial tension following a similar take as the one presented in the equations proposed by Kikuchi et al. (2010).

3.2.4. Bending strength

To evaluate the bending strength of the confined masonry walls, the AIJ equation (Eq. 11) is proposed. Although based on simple assumptions, it gives good results to estimate flexural strength (Sugano et al. 2014).

Table 2 presents a summary of the equations proposed.

Table 2. Summary of equations.

Failure mode	Equation
Sliding	$Q_{su} = 0.806nd^2\sqrt{f_c f_y} + \mu N + \tau_0 A_w$
	$Q_{su} = 0.806nd^2\sqrt{f_c f_y} + \mu N + \tau_0 A_w + A_h f_{hy}$
Diagonal Tension	$Q_{su} = \frac{f'_{mt} A_m}{2.3}\sqrt{\left(1 + \frac{f_a}{f'_{mt}}\right)} + 0.806nd^2\sqrt{f_y f_c}$
	$Q_{su} = \frac{f'_{mt} A_m}{2.3}\sqrt{\left(1 + \frac{f_a}{f'_{mt}}\right)} + 0.806nd^2\sqrt{f_y f_c} + A_h f_{hy}$
Diagonal Compression	$Q_{su} = 0.85\beta f'_m A_{st} \cos\emptyset + 0.806nd^2\sqrt{f_y f_c}$
	$Q_{su} = 0.85\beta f'_m A_{st} \cos\emptyset + 0.806nd^2\sqrt{f_y f_c} + A_h f_{hy}$
Bending	$Q_{su} = \left(A_t f_{ty} + 0.5A_w f_{wy} + 0.5N\right) l_w/h'$
	$Q_{su} = \left(A_t f_{ty} + 0.5A_w f_{wy} + 0.5N\right) l_w/h'$

Where:

- n: $Number\ of\ main\ reinfocement\ bars\ in\ both\ tie\ columns$
- d: $Diameter\ of\ main\ reinforcement\ bars\ in\ tie\ column$
- f_y: $Yieding\ strength\ of\ main\ reinforcement\ bars$
- f_c: $Compressive\ strength\ of\ the\ tie\ column\ concrete$
- N: $Axial\ load$
- μ: $Friction\ coefficient\ of\ the\ mortar$
- τ_0: $Inital\ bond\ strength\ of\ mortar$
- A_h: $Total\ area\ of\ the\ main\ reinforcement\ in\ the\ middle\ tie\ beam$
- f_{hy}: $Yielding\ strenght\ of\ the\ main\ reinforcement\ bars\ in\ middle\ tie\ beam$
- f'_{mt}: $Masonry\ tensile\ strength$
- A_m: $Cross\ sectional\ area\ of\ the\ wall.Calculated\ as\ length\ x\ thickness$
- f_a: $Axial\ stress$
- f'_m: $Masonry\ compressive\ strength$
- β: $Value\ of\ 1, assuming\ uniform\ cross\ sectional\ area\ along\ the\ length\ of\ the\ strut$
- A_{st}: $Strut\ area.Calculated\ as\ width\ x\ thickness$
- $\emptyset$: $Strut\ angle$
- A_t: $Area\ of\ the\ reinforcement\ in\ tension$
- f_{ty}: $Yielding\ strength\ of\ reinforcement\ in\ tension$
- A_w: $Area\ of\ reinforcement\ in\ the\ wall\ section$
- f_{wy}: $Yielding\ strength\ of\ the\ reinforcement\ in\ the\ wall\ section$
- N: $Axial\ load$
- l_w: $0.9L$
- h': $Loading\ point\ height\ (In\ this\ case\ it\ is\ equal\ to\ the\ wall\ height)$

4. RESULTS AND DISCUSSION

4.1. Results

Table 3. Calculated strength of the specimens.

	SPC	SPCI	CMNB	CMB1	CMB2
Sliding	**89.51**	186.03	127.17	170.80	142.86
Diagonal tension	107.10	203.62	**80.83**	**124.46**	**96.51**
Diagonal compression	116.23	207.67	128.18	168.00	140.05
Bending	92.60	**92.60**	125.68	125.68	125.68

4.2. Discussion

4.2.1. Equations

The proposed equations were able to predict correctly the failure mode of each of the specimens. In terms of strength, diagonal tension mode had good agreement, with CMNB specimen having the largest error (18.18%). The other two specimens had significant discrepancy in the calculated capacity. An error of 65.66% for SPC and 35.78% for SPCI. One of the possible reasons for this overestimation was the contribution of tie columns. It is likely that the reinforcement in these elements did not yield at the time the maximum strength was achieved. It is also possible that, if yielding occurred, it was not only due to shear, but also due to flexural action. In such case, modeling their contribution as dowel action would not be resemble completely the real resisting mechanism.

4.2.2. Damage progression of the specimens

Four of the specimens present a significant drop after achieving its maximum strength (Figure 2). It is likely that this reduction in strength and stiffness was due to the failure of the brick panel. The residual strength is attributed to the tie columns that held the wall together. It was observed that the introduction of the middle tie beam reduced vertical cracks in the brick panel-tie column interface, as well as the expansion of the brick panel due to cracking.

The deformation capacity of specimen CMB2 was affected due to insufficient reinforcement in the middle tie beam according to the 1997 technical norm of El Salvador (Figure 3). This element failed early in the test and affected the overall behavior of the wall.

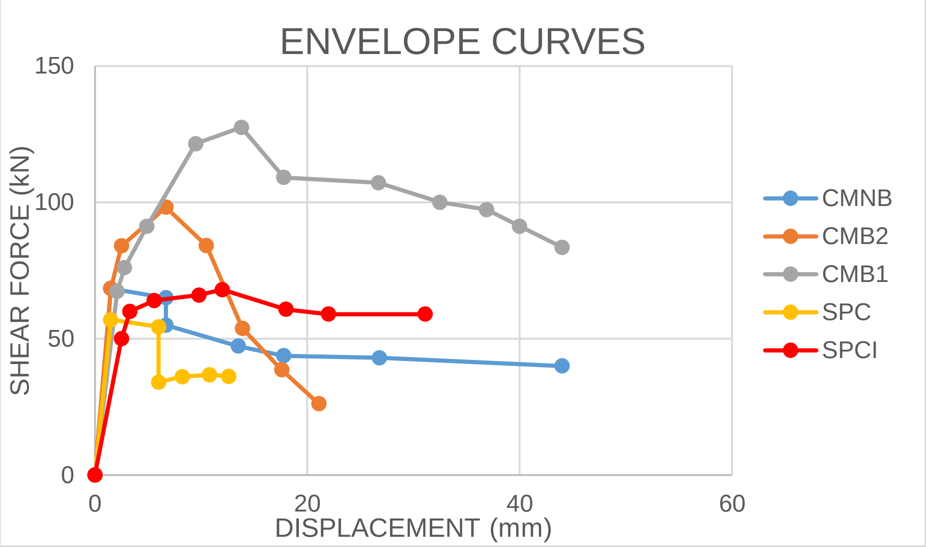

Figure 2. Envelope curves of specimens.

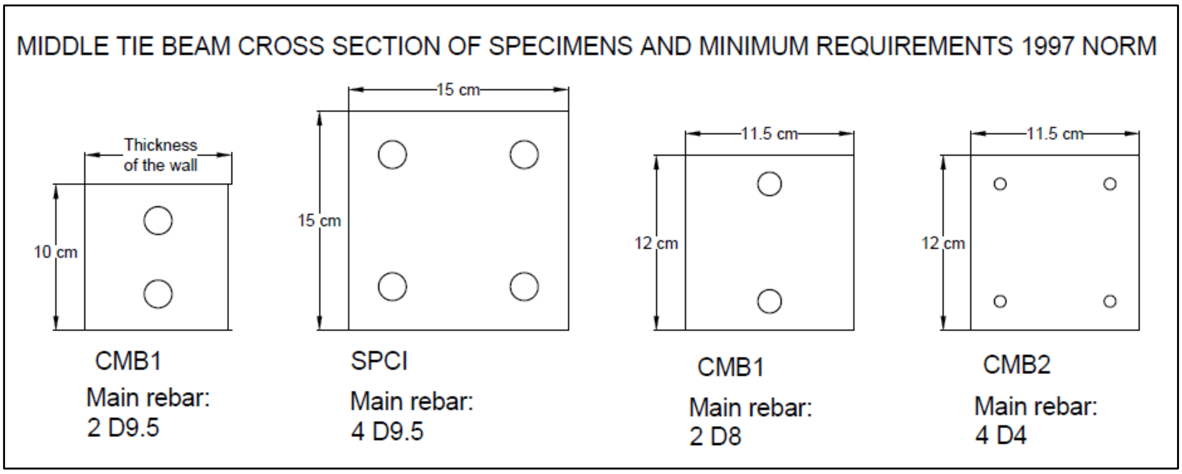

Figure 3. Comparison of tie beam cross sectional areas and technical norm requirements.

5. CONCLUSIONS

- The proposed equations were able to predict correctly the failure mode of the specimens.
- Calculated diagonal tension strength had good agreement with the experimental results.
- There was significant error in the calculated strength of the other failure modes, most likely due to an overestimation of the contribution of tie columns.

RECOMMENDATION

- Further studies are necessary to refine the proposed equations. There were significant errors in the calculation of the strength of the specimens. Most of them attributed to the reinforcement in the tie columns and middle tie beam not yielding when the maximum strength was achieved. Experimental tests are necessary to investigate the behavior of the reinforcement, which would allow more precision in the calculation of the capacity of the wall.

ACKNOWLEDGEMENTS

I would like to express my sincere gratitude to Dr. S. Sugano for his guidance in my individual study. I would also like to express my gratitude to Mr. M. Inukai and Dr. T. Azuhata, who, as well, guided me and offered their valuable advice and experience to me in this study. I would also like to thank Dr. M. Kuroki and Dr. K. Kikuchi, from Sojo University and Oita University for their valuable advice.

REFERENCES

American Concrete Institute Committee 318, 2008, Building Code Requirements for Structural Concrete (ACI 318-08).

CEPAL, 2001, EL Salvador: Evaluación del terremoto del martes 13 de febrero de 2001, United Nations. (In Spanish).

Crisafulli, F. J., 1997, Seismic behavior of reinforced concrete structures with masonry infills, University of Canterbury, Christchurch, New Zealand.

Goto, T., 2016-2017, Lecture notes on Masonry Structures 1, IISEE/BRI.

Goto, T. and Azuhata, T., 2004, An Experimental Study on Seismic Performance Improvement of Framed Masonry Wall, Proceedings of Annual Meeting of Japan Association for Earthquake Engineering, p. 300-301. (In Japanese)

Kuroki, M., Kikuchi, K., and Nonaka, H., 2010, Experimental Study on Reinforcing Methods for Window Openings in Confined Masonry Walls, Manuscript for the 35th Conference on Our World in Concrete & Structures.

Meli, R., 1974, Behavior of masonry walls under lateral loads, 5th World Conference of Earthquake Engineering, Vol. 1, p. 853-862.

Méndez de Hasbun, P., Gutiérrez, M., Ventura, E., Escobar, C., Ayala, N. Ortíz, A., Merlos, R. and López, M., 2008, Mampostería de Suelo Cemento Confinada Informe de Resultados, San Salvador, El Salvador, Viceministerio de vivienda y desarrollo urbano. (In Spanish)

Priestley, M. J. N. and Bridgeman, D. O., 1974, Seismic resistance of brick masonry walls, Bulletin of New Zealand and National Society for Earthquake Engineering, Vol. 7, No. 4.

Saito, T., STERA 3D Ver. 9.0 User Manual, Toyohashi University of Technology (TUT), Japan.

Sánchez Tizapa, S., 2009, Experimental and numerical study of confined masonry walls under in-plane loads: case: Guerrero state, Université Paris-Est.

Sugano, S., Saito, T., Zavala, C. and Cardenas, L., 2014, Strength and Deformation of Confined Brick Masonry Walls Subjected to Lateral Forces –Review of Existing Test Data in Japan and Peru-, Journal of Disaster Research, Vol. 9, No. 6, p. 984-992.

Tomazevic, M., 1999, Earthquake-resistant Design of Masonry Buildings, Imperial College Press, London, England.