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Experimental Study on Wall-Frame Connection of Confined Masonry Wall

Wira Wijaya[1], Dyah Kusumastuti[2], Made Suarjana[2], Rildova[2], and Krishna Pribadi[2]

[1]*Graduate student, Faculty of Civil and Environmental Engineering, Institute of Technology Bandung, Indonesia*
[2]*Faculty of Civil and Environmental Engineering, Institute of Technology Bandung, Indonesia*

Abstract

Four full-scale (3m × 3m) confined masonry wall specimens that represented simple house wall panels in Indonesia were subjected to cyclic in-plane lateral load. The construction of the specimens, including reinforcement assemblies, concreting, and brick-laying, followed the common construction practice in Indonesia. A specimen with no anchorage between the wall and the reinforced concrete frame was chosen as a benchmark model. The other specimens were varied in the details of wall-frame connection, i.e. zigzag (toothing) connection, short anchor between column and wall, and continuous anchorage from column to column. The models were then subjected to cyclic in-plane lateral loads, which represents earthquake loads, with increasing amplitude until collapsed. The behavior of these specimens was then evaluated and compared. The parameters evaluated were crack patterns and failure mechanism of the wall panel, loading capacity, and energy dissipation. The study revealed that zigzag connection and short anchor did not improve the performance of the confined masonry wall; instead they were more likely to reduce the performance of the wall. Cracks and failures of the two specimens were initiated by vertical crack on the face of the wall-frame connection, which then reduced the confinement of the wall. Therefore, the final failure mode followed sliding shear patterns on the bed joint of brick-mortar, which produced more brittle failure. Conversely, continuous anchorage strengthened the confinement, thus the diagonal crack patterns were observed on the wall and the strut and tie mechanism between the wall and the confining column was developed. Therefore, this specimen shows more ductile behavior as well as higher lateral load capacity. In conclusion, the study shows that installing proper wall-frame connection strategies is crucial in improving the structural performance.

Keywords: Confined Masonry Wall, Wall-frame Connections, Collapse Mechanism.

1. INTRODUCTION

The classifications of masonry wall are described in the Eurocode 6 (1996) and FEMA 306 (1998). Eurocode 6 classifies masonry wall into three categories: Unreinforced Masonry, Confined Masonry, and Reinforced Masonry. FEMA 306 categorizes masonry wall into three groups: Reinforced Masonry,

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Unreinforced Masonry, and Infilled Frame. Using both references, there are differences between confined masonry and infilled wall (infilled frame on FEMA) in terms of construction methods and lateral resistance mechanisms.

Infilled walls are usually constructed on buildings that have more than one story, in which frame construction preceded the construction of the infilled masonry wall. The lateral resistance mechanism of such structure is largely contributed by the frame, whereas the infilled wall provides additional stiffness and strength to the frame.

As for confined masonry wall, the construction is initiated by the construction of masonry wall and then followed by construction of the confining frame. This type of wall is commonly used as a panel wall in simple, one- to two-story housing in developing countries, such as Indonesia. For this particular type of wall, the lateral resistance mechanism is mostly contributed by the wall, provided that the wall has strong confinement. The confinement plays significant role in maintaining the wall integrity in order to develop optimum lateral resistance.

The installation of wall-frame connections on a confined masonry wall has been introduced to preserve the frame confinement to the wall and to improve the overall seismic performance of the structure. Although many numerical studies have been conducted on the subject, few experimental studies were carried out to study the effect of such connections on the structure. Therefore, an experimental study on wall-frame connections on a masonry wall confined by reinforced concrete frame was conducted.

2. EXPERIMENTAL PROGRAM

2.1. Material Properties

Brick materials used for specimens were clay bricks with moderate quality, and the dimension of the bricks used was 55x100x205 (mm). Mortar space in between bricks was 15mm thick with mortar mixture of 1:5 for cement and sand respectively, adding 100% water from the cement volume. The concrete frame mixture was of 1:2:3 for cement, sand and aggregate respectively with water being added as much as 100% from the cement volume. The frame reinforcements were plain rebars with ϕ8 for stirrups/hoops and ϕ10 for longitudinal reinforcement. Material properties obtained from testing of materials are presented in Table 1.

2.2. Test Specimens

The specimens used for the experiment were four full-scale (3m x 3m) confined masonry walls as described in Table 2 and illustrated in **Error! Reference source not found.** to **Error! Reference source not found.**. Model A represented the common practice without anchorage. Model B used short (32 cm) ϕ8 anchorage for every six layers of bricks. Model C utilized zigzag end surface of the brick wall for additional connection from the wall to the frame. Model D used two continuous column to column ϕ8 anchorages in addition to short anchorages as used in Model B. The continuous anchors were placed every 1 m height of the wall.

The beams and columns for all specimens were 150 mm x 150 mm. The reinforcement was plain bar ϕ10 for longitudinal reinforcement and ϕ8 for stirrups/hoops. The beam-column joints used 40d development length with hook

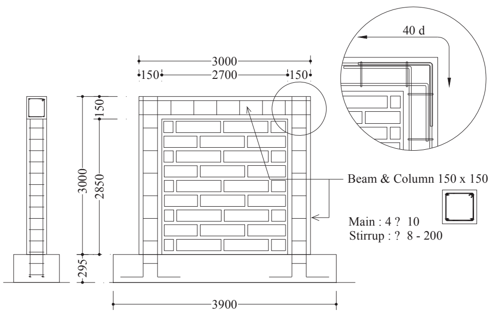
.

Table 1: Material properties

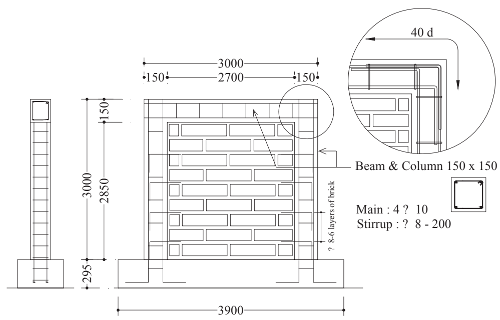
No	Material Test	Number of Samples	Average Compressive / Tensile Strength (MPa)	Standard Deviation (MPa)
1	Brick compressive strength	10	4.16	0.6
2	Concrete mortar compressive strength	8	8.74	2.88
3	Concrete compressive strength	14	18.08	2.71
4	Brick-mortar bonding	8	0.203	0.077
5	ϕ8 bar tension strength	3	350.97	23.54
6	ϕ10 bar tension strength	3	317.34	31.63

Table 2: Specimens details

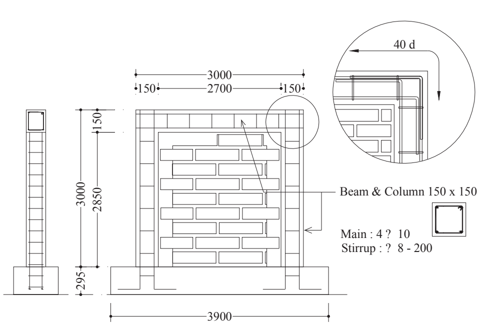
No	Model	Wall-Frame Connection
1	A (Common Practice)	-
2	B (Short Anchorage)	Brick-column anchor ϕ8 @ 6 brick layer, anchor length 32cm
3	C (Zigzag Connection)	-
4	D (Continuous Anchorage)	2 continuous column to column anchor ϕ8, and @6 brick layers ϕ8 - 32cm



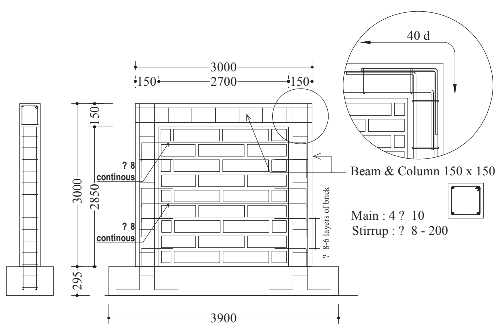
Model A (Common Practice)



Model B (Anchorage)



Model C (Zigzag Connection)



Model D (Continous Anchorage)

Figure 1: Test specimens

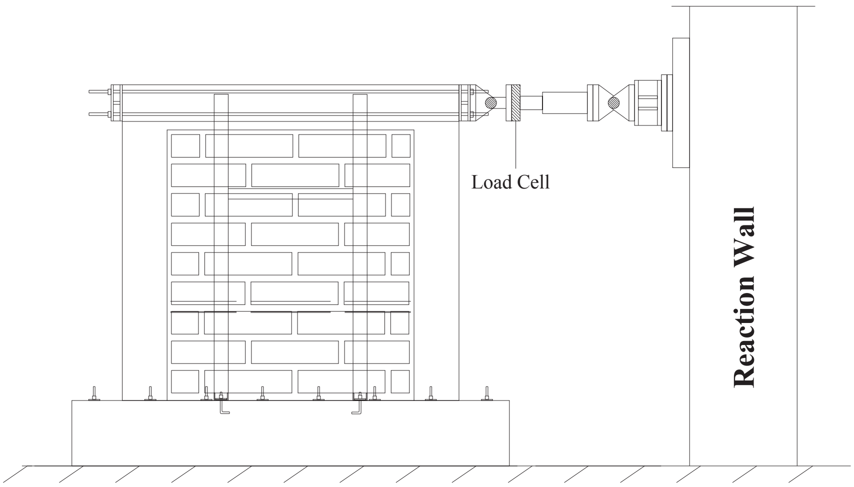


Figure 2: Test setup

Figure 2 presents the arrangement of the test set up. Strain gauges were installed on the reinforcement bars and LVDT (Linear Variable Displacement Transducers) were used to measure the displacement.

2.3. Testing Procedure

The experiment was conducted by applying in-plane quasi-static cyclic lateral loads. The lateral load was applied on the specimen's beam, using displacement control based on the measured deformation of the top of the specimens. Loading history was applied following ACI 374.1-05 recommendation as presented in Figure 3. The experiment was conducted until the structure collapsed or reached 5% drift. **Error! Reference source not found.** shows the testing of the specimen.

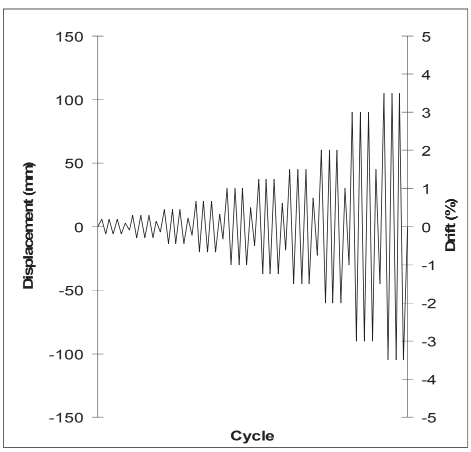


Figure 3: Loading scheme diagram

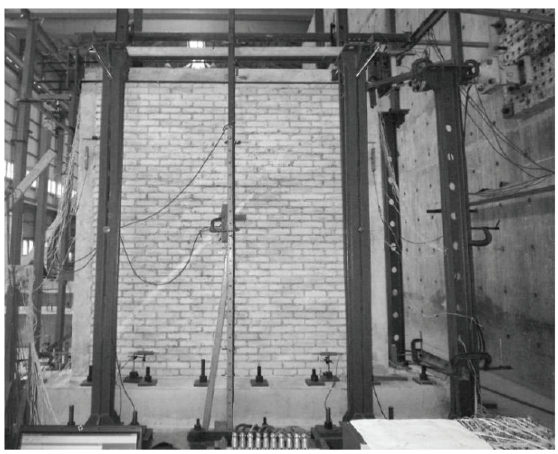


Figure 4: Testing of specimen

3. TEST RESULT AND DISCUSSION

3.1. Crack Patterns and Modes of Failure

In general, initial cracks on the wall panel appeared at 2 mm lateral displacement (drift 0.067%). The crack pattern showed that Model C (zigzag connection) developed mostly sliding shear crack at the upper half of the wall. Other models showed similar confining effect on the wall and were able to develop diagonal crack mechanism and formed compression strut (Figure 5).

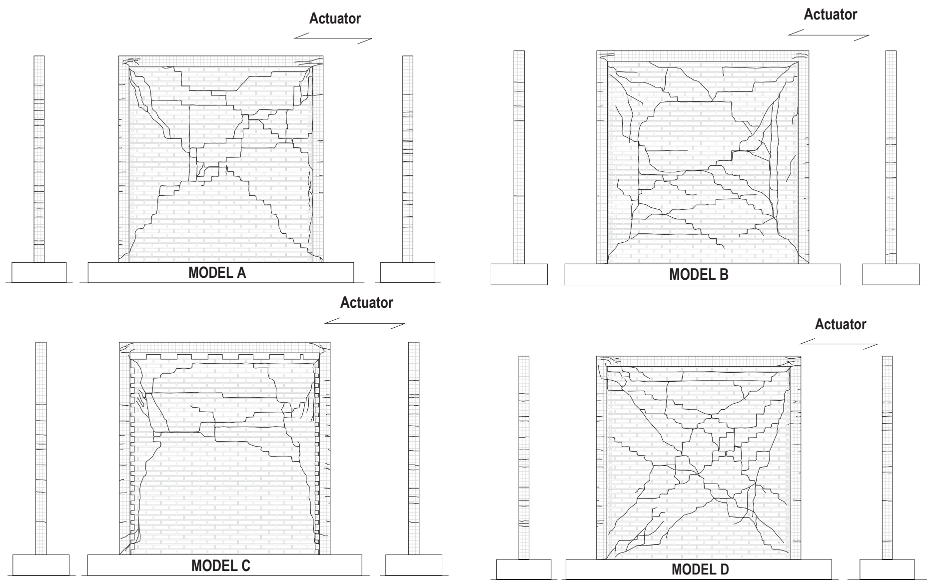


Figure 5: Crack patterns

Figure 8 shows that Models A and B have similar type of crack patterns. However, in Model B there is a shift in the weak point of the masonry from the column face to the front of the anchor.

The inability of the cracks on the masonry wall to close perfectly at zero drift added the volume of the wall panel, which then pushed the columns outward. The confining columns were then bent out in the wall plane (bulging effect). Bulging effect on the confining columns subsequently weakened the confinement and thus reduced the wall strength.

Model D (continuous anchorage) which has reinforcement on wall and columns succeeded in preventing large cracks on the wall and produced strong confinement during the experiment. The strong confinement has enable Model D to develop compression strut in both directions and therefore develop optimum lateral resistance (Figure 6).

3.2. Lateral Capacity

Figure 7 shows hysteretic loops of each specimen and Figure 8 shows the envelope of the hysteretic loop for all specimens. Due to brittle and unisotropic characteristics of the wall as mentioned previously, most of the specimens display asymmetrical form of envelope hysteretic curve, which indicates different behavior towards push and pull loading. The lateral capacity used will be defined by the minimum lateral capacity between push and pull loadings.

Summary of the hysteretic behavior for all specimens is presented in Table 3. Compared to Model A, additional short anchor on Model B slightly improve the lateral strength of the wall. Model D with continuous anchorage has the highest lateral load capacity due to strong confinement. However, Model C revealed that zigzag connection did not improve the lateral load capacity and this specimen has the lowest capacity of all models.

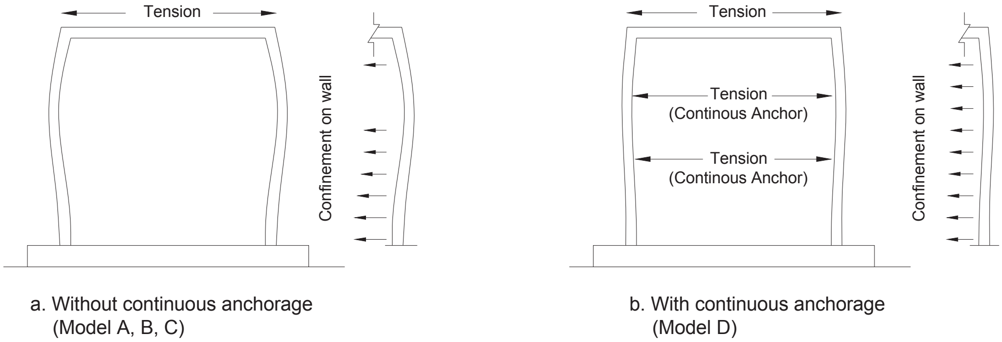


Figure 6: Bulging effect on the confining columns

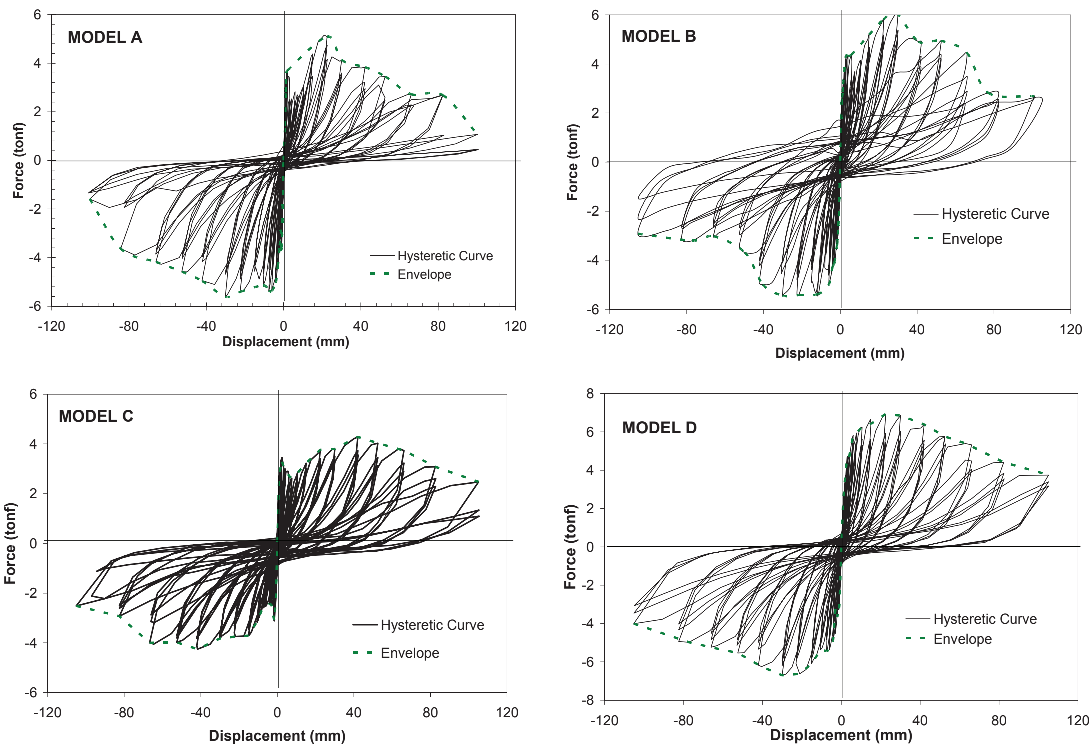


Figure 7: Specimens hysteretic loops

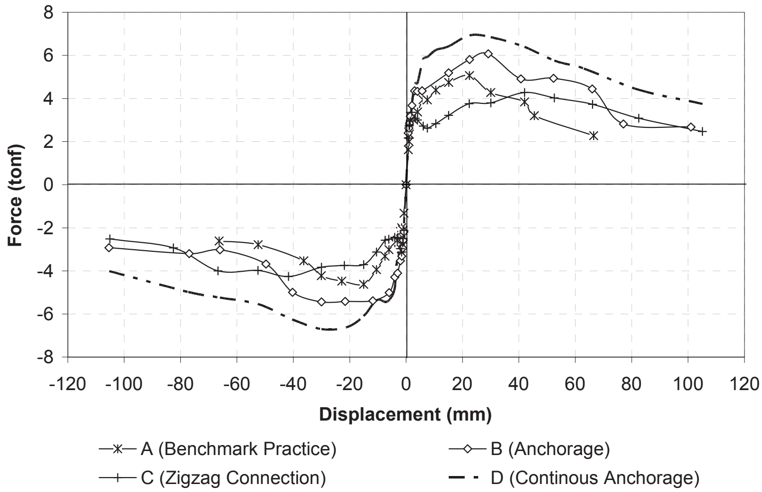


Figure 8: Envelope curve of hysteretic loops

Table 3: Hysteretic behavior of specimens

MODEL	Lateral Load Capacity (tonf)	Drift at Max Load (%)	Drift at Yield (%)	Drift at Ultimate (80% Max Load) (%)
A (Common Practice)	5.09	0.75	0.26	1.08
B (Anchorage)	5.44	0.97	0.34	1.82
C (Zigzag Connection)	4.26	1.40	0.54	2.46
D (Continuous Anchorage)	6.7	1.00	0.30	2.02

3.3. Input Energy and Energy Dissipation

The cumulative input energy E_{inp} is defined as the cumulative work of the actuator from the beginning of the test to the final displacement amplitude (destination drift ratio). Work of the actuator in one loading cycle ΔE_{inp} is calculated as the area under the positive and negative part of hysteretic loop (Figure 9). The amount of dissipated energy in one cycle of loading was calculated as the area inside a single hysteretic loop.

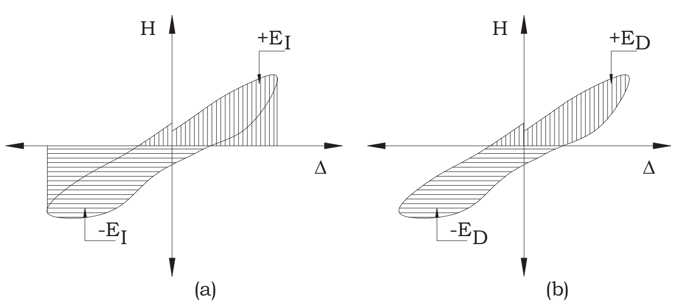


Figure 9: (a) Input energy in one loading cycle; (b) Dissipated hysteretic energy in one loading cycle

Table 4: Cumulative input energy and dissipated energy

MODEL	Drift 0 - 3.5%		
	Input Energy (kN-mm)	Dissipated Energy (kN-mm)	% Dissipated
A (Common Practice)	196294.40	142274.09	72.48
B (Anchorage)	231386.35	172393.32	74.50
C (Zigzag Connection)	178407.42	128681.59	72.13
D (Continuous Anchorage)	283188.49	211173.52	74.57

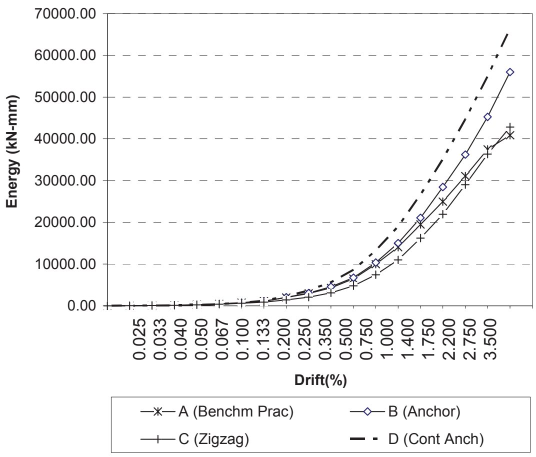


Figure 10: Cumulative input energy

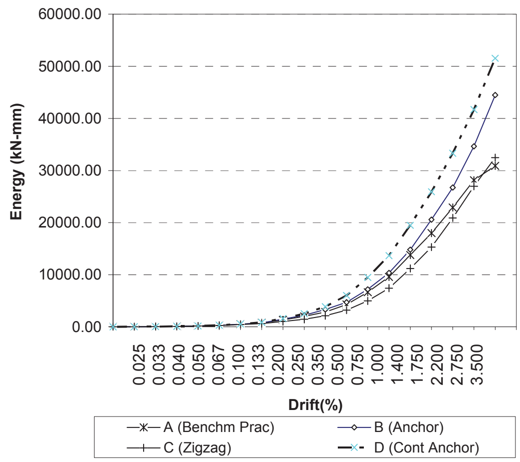


Figure 11: Cumulative dissipated energy

Table 4 and Figure 10 and 15 show that up to drift 3.5%, the highest input energy was produced by Model D with 283.2 kN-m. Model D also shows the highest dissipated energy, with 74.6% energy dissipation. Although Model B developed somewhat lower input energy and dissipated energy, the percentage of energy dissipation is comparable to Model D. However, additional zigzag connection on Model C seems to have no significant influence on its input and dissipated energy, compared to Model A. It appears that the presence of anchorage (short and continuous) requires more input energy to cause damage to the specimens.

4. CONCLUSION

The behavior of four confined reinforced masonry walls with variations of wall-frame connection details in resisting in-plane lateral cyclic loads was investigated experimentally.

The study revealed that zigzag connection and short anchor did not improve the performance of the confined masonry wall; instead it is more likely to reduce the performance of the wall. Crack patterns and failures on the brick wall were initiated by vertical crack on the face of wall-frame connections, which then reduced the confinement of the wall. Therefore, the final failure mode followed sliding shear patterns on the bed joint of bricks-mortar, thus reducing the structural performance. Conversely, continuous anchorage strengthened the confinement of the wall and allowed the development of diagonal crack patterns. As a result, the strut and tie mechanism between the wall and the confining column was able to develop as lateral load resistance mechanism. Therefore, better structural performance was observed for this specimen. The study shows that installing proper wall-frame connection strategies is crucial in improving the structural performance.

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REFERENCES

[1] ACI Committee 374 (2005). "Acceptance Criteria for Moment Frames Based on Structural Testing and Commentary" (ACI 374.1-05), American Concrete Institute.

[2] BSSC. (2003). "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures" (FEMA-450) Part 1: Provisions, Building Seismic Savety Council, Washinton DC.

[3] European Committee of standardization (CEN) (1996). "Design of Masonry Structures. Part 1-1: General Rules for Buildings-Reinforced and unreinforced Masonry". ENV 1996 1-1 Eurocode 6, UK

[4] FEMA 306 (1998), "Evaluation of Earthquake Damage Concrete and Masonry Wall Buildings, Basic Procedures Manual", Federal Emergency Management Agency, Washington, D.C.

[5] Tomaževič, M. (1999), "Earthquake-Resistant Design of Masonry Buildings", Imperial Collage Press.

[6] Tomaževič, M., Lutman, M., and Petkovic, L., (1996), "Seismic Behavior of Masonry Walls: Experimental Simulation", J. Struct. Eng,.ASCE, 122(9).1048–1054.