Analytical Approach to Estimate Equivalent Strut Width for Wire Meshed Masonry Infilled RC Frame under Dynamic Loading

S. V. S. Jebadurai1, D. Tensing1, C. Daniel1,2,*, E. Arunraj1, G. Hemalatha1

1Department of Civil Engineering, Karunya Institute of Technology and Sciences, 641114, Coimbatore, India
2Department of Civil Engineering, Mohamed Sathak A J College of Engineering, 603103, Chennai, India

Received April 25, 2021; Revised May 20, 2021; Accepted June 27, 2021

Cite This Paper in the following Citation Styles

(a): [1] S. V. S. Jebadurai, D. Tensing, C. Daniel, E. Arunraj, G. Hemalatha, "Analytical Approach to Estimate Equivalent Strut Width for Wire Meshed Masonry Infilled RC Frame under Dynamic Loading," Civil Engineering and Architecture, Vol. 9, No. 4, pp. 1198-1205, 2021. DOI: 10.13189/cea.2021.090421.

(b): S. V. S. Jebadurai, D. Tensing, C. Daniel, E. Arunraj, G. Hemalatha (2021). Analytical Approach to Estimate Equivalent Strut Width for Wire Meshed Masonry Infilled RC Frame under Dynamic Loading. Civil Engineering and Architecture, 9(4), 1198-1205. DOI: 10.13189/cea.2021.090421.

Copyright©2021 by authors, all rights reserved. Authors agree that this article remains permanently open access under the terms of the Creative Commons Attribution License 4.0 International License

Abstract In this paper, an analytical investigation is carried out using an open-source FEM software SEISMOSTRUT to analyze infill RC frame with and without chicken wire mesh along with experimental verification. To estimate the equivalent strut width, six models proposed by various researchers are considered to find various failure modes of infills such as in tension, compression and shear. The theoretical model had the same dimensions and load pattern as compared to experimental investigation. For studying the infill wall's lateral load capacity, two specimens were cast, namely infill wall without mesh (B1) and infill wall with mesh (B2). From the experimental investigations, yield displacement (\(\Delta y\)), Initial stiffness (Ki), Ultimate loads (Pu), Ultimate displacement (\(\Delta u\)), Ductility (\(\mu\)) and Cumulative energy dissipation capacity were estimated. The proposed model is found to be in close agreement with the experimental model results in terms of ultimate load and displacement. The failure mode observed for the infill walls was diagonal tension in the experimental investigation. Based on diagonal tension and corner crushing mode, an equation is derived which is suitable for estimating the equivalent strut width for walls with mesh and the failure loads in compression and tension. The failure loads calculated from the proposed empirical relations are compared with the experimental investigations for verification.

Keywords FEM, SEISMOSTRUT, Infill Masonry, Chicken Wire Mesh, Failure Modes

1. Introduction

Experimental investigation of infill walls is not feasible in many situations due to the cost involved in constructing the model and time required for testing. Also, since the infill wall's behavior is a complex phenomenon, the study of various parameters is a difficult task [1]. Hence, researchers have worked on numerical models to estimate the behavior’s parameters and predict the failure patterns. The models are classified under two broad categories as Macro-models and Micro-models. Macro-models are more simplified than Micro-models as the Macro-models represent infill masonry panels' global behavior and their impact in building response. A method as an equivalent strut to be representing the masonry infill which carries out a portion of the horizontal loads applied and relieves stresses on other structural components. This leads to redistribution of stresses and can lead to localized stress resulting in localized cracking. Macro-models can be designed as a continuum macro element or equivalent strut model. The contact length between infill and frame can be single or multiple. A finite element package can
predict how realistic structures behave if the difference between experimental and analytical results is less than 20 percent [2]. Mallick and Severn carried out the first implementation of the finite element package in 1967, and later, other researchers successfully used various finite element packages to investigate the interaction of the infilled frame [3]. Simulation of brick masonry is difficult because several determining parameters have to be considered, including dimensions and anisotropy of bricks, joint width, and bed joints arrangement. The models' level of complexity depends on the various phase material of refinement of masonry models that are evaluated [4].

A wide variety of software packages are developed to perform a nonlinear, static, and dynamic study of 2D or 3D ANSYS, ABACUS, ADINA, DIANA and ATHENA that are few widely used finite element packages. Over the years, with the increase in sophistication, the modelling and interfacing have been done conveniently using this software. However, mesh responsiveness is difficult in these models. Quite dense meshes often yield better performance, but this may not be true when modelling brittle materials. Alternatively, OpenSees software specially formulated for masonry is available to design the masonry infill with ease. SEISMOSTRUT is one such software designed exclusively for masonry walls, and the results are found to be reliable. Several researchers have developed models to predict the behavior of infill walls and its interaction with the surrounding frames [5-8]. In this context, Holmes first suggested the strut width, the model based on infill walls' shear failure [9]. An average of 20-30% of the infill panel perimeter was found to contact the frame after the bond between the infill panel, and the frame was lost [10]. A contact length in the range of 5 and 50% of the frame height was proposed by Smith [11]. For formulating the models for the infill walls, researchers have considered various failure modes that are predominant. The modes of failure of infill walls depend on the frame's relative strength and the infill [12].

The failure modes considered by various researchers for formulating the models are summarized. Smith considered the diagonal tension and diagonal compression failure as the predominant failure mode when the infill is weak compared to the frame [13]. Later, Smith and Haldar revised the formula, including the infilled frame's sliding shear failure [14]. Mainstone derived his equation based on diagonal tension and corner crushing [15]. Wood also studied the probability of shear failure for a strong frame and weak infill and proposed equations considering sliding shear failure, diagonal tension, and corner crushing [16]. Liauw and Kwan proposed empirical relationships based on diagonal compression and infill corner crushing [17]. Smith and Coull suggested a single-corner crushing model [18]. Priestley and Haldar articulated bed-joint sliding shear strength as a function of the compressive strength of infill material and infill and frame geometry, as was adopted [19]. Saneinejad and Hobbs defined shear failure, diagonal stress, diagonal compression and corner crushing for infill frame failure mode [20]. Flanagan and Bennett found their model's corner-crushing [21].

Kappos suggested a basic design method for infilled frames based on a dual stiffness [22]. Anić has provided a study of work on the out-of-plane behaviour of infilled masonry frames [23]. Diana Wang addressed previous research on reinforcement/refitting strategies for seismic-loaded URM buildings. Various researchers have given their opinions through analytical expressions to ease the complications. Thus, an analytical approach is chosen, which has been compared with an experimental work to verify its effectiveness and derive the model equation for the infill wall with wire mesh in open-source software [24].

The models described above cannot accurately predict frame members’ shear force and bending moment diagrams. This is because a strut model cannot reflect the actual contact length/area between the frame and the infill panel. To address this issue, researchers proposed multi-strut models for masonry infill.

2. Model Set Up

2.1. Experimental Setup

For the experimental investigation, specimens of single-bay reinforced concrete frames scaled down to 1:3 ratio were considered. The thickness of the wall is 77mm, and the thickness of the wire mesh is 1mm. The reinforcement details on the frames are shown in figure 1.
The infill bricks were also scaled down to cater to the thickness of the infill wall. Specimens were cast namely infill wall without mesh (B1), and infill wall with mesh (B2) and the average results are taken into consideration. Experimental investigations were done on B1 and B2 to study the wall's in-plane behavior subjected to lateral load, which is of dynamic nature of push and pull. The schematic diagram of placing of wire mesh on the infill wall's surface is shown in figure 2. 4 numbers of 8mm diameter rod of grade Fe 250 provided for each column which is laterally tied on all four corners. The chicken mesh was attached vertically on both the infill wall's sides to reduce the internal stresses. The plaster's thickness is 10mm, with the cement sand mortar mix ratio of 1:3.

The lateral displacements at the top and bottom of the test specimens were measured using two LVDT’s (linearly varying differential transformers) with a stroke length of ±50mm. Typical instrumentation scheme and the loading sequence for the experiment are presented in figure 3. The response of the model B1 and B2 such as the Pre-yield stiffness (Ki), Yield displacement (Δy), Ultimate displacement (Δu), Ultimate load (Pu), Ductility (μ) and energy dissipation are calculated as mentioned in Table 1. The failure pattern of the specimens observed is the diagonal tension.

![Figure 1](image1.png) Reinforcement detail of the frame [25]

![Figure 2](image2.png) The schematic diagram for fixing Chicken mesh

![Figure 3](image3.png) Experimental setup

### Table 1. Comparison of responses of the frames

| Specimen | Description          | K_i (kN/mm) | Δy (mm) | Δu (mm) | P_u (kN) | μ   | Energy dissipation kNm |
|----------|----------------------|-------------|---------|---------|---------|-----|------------------------|
| B1       | Infill frame without mesh | 1.65        | 2.09    | 30.61   | 16.20   | 14.64 | 189                    |
| B2       | Infill frame with mesh   | 2.71        | 1.21    | 21.36   | 19.30   | 17.60 | 198                    |

2.2. Analytical Investigation on Infill Wall

For the analytical investigation of the infill wall, the
following two models are considered for modelling using SEISMOSTRUT:
I) Infill wall without mesh (A1)
II) Infill wall with mesh (A2)

![Figure 4. Cyclic load applied in SEISMOSTRUT](image-url)

For analytical investigations, the beams and columns were modelled using beam element and the infill wall using the panel element in SEISMOSTRUT software. The Cyclic load applied to an infill wall as per the loading pattern is depicted in figures 4 and 5. The load vs displacement behavior of A1 and A2 specimen is investigated.

![Figure 5. Loading pattern for cyclic loading](image-url)

Table 2. Strut width ($w_{ds}$) proposed by six researchers

| Models                   | Strut Width ($w_{ds}$)                        | $W_{ds}$ (m) | Failure type                                                                 | Eqn No.               |
|--------------------------|-----------------------------------------------|--------------|------------------------------------------------------------------------------|-----------------------|
| Holmes (1961)            | $w_{ds} = \frac{d}{3}$                         | 0.471        | shear failure                                                                | (1)                   |
| Stafford Smith & Carter (1969) | $w_{ds} = 0.58(l/h)^{0.445} \lambda H^{0.335} (l/h)^{0.064}$ | 0.513        | Sliding shear failure, diagonal tension, diagonal compression, Tension failure of columns, shear failure of beam/column. | (2)                   |
| Mainstone (1971)         | $w_{ds} = 0.175(\lambda H)^{-0.4} d$          | 0.278        | Diagonal tension and corner crushing                                         | (3)                   |
| Liauw TC & Kwan KH (1984)| $w_{ds} = \frac{0.95h \cos \theta }{\lambda H}$ | 0.567        | Diagonal compression and corner crushing                                    | (4)                   |
| Paulay & Pristley (1992) | $w_{ds} = \frac{d}{4}$                        | 0.35         | Sliding shear failure, diagonal compression                                 | (5)                   |
| Chrysostomou (2012)      | $w_{ds} = 0.27(\lambda h)^{-0.4} d$           | 0.423        | Diagonal tension and corner crushing                                        | (6)                   |
3. Results and Discussions

3.1. Infill Wall without Mesh (A1)

To compare the experimental and analytical results and derive a model equation for infill wall with chicken mesh as skin reinforcement, an open-source software SEISMOSTRUT was used. This is a Finite Element software, and it is designed based on the equivalent strut and tie model. Six models proposed by researchers were considered for the study. The models chosen are shown in table 2.

\[ \lambda H = \left[ \frac{E_{\text{wall}}^H}{E_{\text{columns}}} \right] \left[ \frac{h_{\text{wall}}^H}{h_{\text{columns}}} \sin 2\theta \right] \]  

(7)

The model width was calculated based on the equations proposed by the researchers, as mentioned earlier and given in the SEISMOSTRUT. A single bay RCC frame was taken into for the analysis. The model was dimensioned and designed similar to the experimental model for comparison. The lateral load was applied in 1kN steps at the top node, and the displacement response was calculated. The parameters like yield load, yield displacement, maximum load and maximum displacement were found from the analytical investigation. The results are compiled in table 3.

The results from the experimental investigations were compared with the analytical results using the expression of various researchers. From the comparison, it can be seen that the percentage variation of displacement, when comparing with the experimental study is observed as 3.14%, which is the lowest difference obtained for Mainstone.

3.1.1. Failure Modes and load-carrying capacity of A1

Further, to define failure modes using mathematical modelling, the equation suggested by Dawe and McBride was considered [26-27]. Equations 8-10 are based on the plastic moment of beams, columns, joints, infill strength.

\[ \min \left\{ \frac{2(M_{pj} + M_{ps})}{\gamma_p f_{\text{m}} t h^2}, \tan \theta \frac{2(M_{pj} + M_{ps})}{\gamma_p f_{\text{m}} t h^2} + \frac{4M_{aj}}{\gamma_p f_{\text{m}} t h^2} \right\} \]  

(8)

\[ \gamma_p = 2.663m^2 - 1.37m + 0.406 \leq 0.45 \]  

(9)

\[ m = \frac{8M_{pj}}{f_{\text{m}} t L^2} \]  

(10)

W, where,

- \( M_{pc} \) is the plastic moment capacity of the column
- \( M_{pb} \) is the plastic moment capacity of the beam
- \( M_{aj} \) is the plastic moment capacity of the joint (smaller of \( M_{pc} \) and \( M_{pb} \))
- \( H \) is the column height between centre lines of beams
- \( h \) is the height of the column (Chrysostom)
- \( f'\text{m} \) is the compressive strength of infill
- \( t \) is the thickness of infill
- \( \theta \) is the angle between the diagonal and the horizontal
- \( L \) is the length of the infill

To calculate the beams and columns' moment capacity, push over analysis was carried out on the single bay RC frame using SAP2000. The ultimate moments in beams and columns were estimated as those when the plastic hinges were formed. It was observed that the plastic hinge was formed in the beam first and subsequently in the column.

| Models          | Yield Load (kN) | Yield displacement | Variation wrt Experiment (%) | Ult. Load (kN) | Max. Displacement (mm) | Variation wrt Experiment (%) |
|-----------------|-----------------|--------------------|------------------------------|----------------|------------------------|------------------------------|
| Smith           | 8.80            | 1.36               | 26.49                        | 10.41          | 6.61                   | 25.98                        |
| Mainstone       | **8.70**        | **1.35**           | **27.02**                    | **10.18**      | **8.65**               | **3.14**                     |
| Paulay & Priestly | 8.80            | 1.30               | 29.73                        | 10.31          | 7.51                   | 15.90                        |
| Holmes          | 8.80            | 1.37               | 25.94                        | 10.40          | 6.68                   | 25.20                        |
| Liauw TC & Kwan KH | 8.60            | 1.31               | 29.19                        | 10.30          | 8.08                   | 9.52                         |
| Chrysostom      | 8.80            | 1.38               | 25.40                        | 10.30          | 8.19                   | 8.29                         |
| Experimental    | **10.50**       | **1.85**           | -                            | **16.20**      | **8.93**               | -                            |

Table 3. Comparison for Infill wall without mesh (A1) Models
Table 4. Push over-analysis of the frame

| Plastic Moment | Ultimate Moment capacity |
|----------------|--------------------------|
| $M_{pb}$       | 0.412 kN-m               |
| $M_{pc}$       | 0.597 kN-m               |
| $M_{aj}$ (minimum of $M_{pb}$ and $M_{pc}$) | 0.412 kN-m |

Table 5. Comparison for Infill wall with mesh (A2)

| Models                  | Yield Load (kN) | Yield displacement | Variation wrt Experiment (%) | Ult. Load (kN) | Maximum Displacement (mm) | Variation wrt Experiment (%) |
|-------------------------|-----------------|--------------------|-------------------------------|----------------|--------------------------|-------------------------------|
| Smith                   | 8.90            | 1.28               | 39.05                         | 10.71          | 3.28                     | 47.10                         |
| Mainstone               | 8.90            | 1.38               | 34.29                         | 10.37          | 5.85                     | 5.65                          |
| Paulay&Pristley         | 8.90            | 1.33               | 36.66                         | 10.61          | 4.19                     | 32.42                         |
| Holmes                  | 9.00            | 1.32               | 37.14                         | 10.59          | 3.35                     | 45.97                         |
| Liauw TC& Kwan KH       | 8.80            | 1.32               | 37.14                         | 10.51          | 4.87                     | 21.45                         |
| Chrysostom              | 8.90            | 1.36               | 35.24                         | 10.50          | 5.08                     | 18.06                         |
| Experimental            | 12.50           | 2.10               | -                             | 19.30          | 6.20                     | -                             |

Solving equation 8, it was found that the equation for mode 3 resulted in the least value of 0.209. This mode indicates failure due to diagonal tension. The pushover analysis of the frame results is tabulated in table 4.

The equations proposed by Mainstone for estimating the compressive load and Saneinejad and Hobbs for tension load were considered for specimen A1. They were taken as the reference and compared with the experimental results [15,20].

3.2. Infill wall with Chicken Wire Mesh (A2)

The empirical equation for the infill wall with chicken mesh was determined by approaching the strut's equivalent width inclusive of the mesh. To estimate the width of the strut, the equation proposed by Akin was taken to calculate the capacity $V_{cr}$ of the infill given as [28],

$$A_{cm} = w_{cm} t_{tie}$$  \hspace{1cm} (11)

$$t_{tie} = t_{cm} + t_{in}$$  \hspace{1cm} (12)

$$f_{cm} = \frac{V_{cr}}{A_{cm}}$$  \hspace{1cm} (13)

Where,
- $f_{cm}$ - tensile strength of the chicken mesh
- $W_{cm}$ - width of the chicken mesh
- $t_{in}$ - thickness of infill
- $t_{cm}$ - thickness of the chicken mesh

Based on the equations 11-13, the infill wall's capacity with chicken mesh is obtained as 24.44kN. Table 3 and Table 4 show that the percentage variation in the ultimate load was closely correlated with the model proposed by Mainstone. The equivalent strut area factor was found to be 2.5, i.e., the strut area for all the models mentioned in table 2 is multiplied by the factor 2.5. This value of strut area is given as input in the SEISMOSTRUT software. The yield displacement results, ultimate displacement, yield, and ultimate load are presented in table 5. It can be observed that the percentage of variation was a minimum of 3.6% for the model proposed by Mainstone and maximum of 47.10 % for the model proposed by Smith.

The equation proposed by Mainstone resulted in values in close correlation with the experimental results, as mentioned in table 5.

Hence, equation 3 was modified by increasing the width as mesh increases the wall stiffness. The proposed equation is given in equation no. 14, which includes 2.5 as the modification factor

$$w_{ds} = 0.175(\lambda h)^{-0.4}d \left(\frac{E_{cm}}{E_m}\right)^{2.5}$$  \hspace{1cm} (14)

For estimating the failure load, the equation proposed by Mainstone (1971) was modified by a factor of 0.875. The maximum compression load is given by,

$$V_c = 0.50(\lambda h)^{-0.875} f_{cm} h(t_{in} + t_{cm}) \cot \theta$$  \hspace{1cm} (15)

Where $f_{cm}$, Compressive strength of infill with mesh

The tensile cracking load is proposed as,

$$V_{cr} = \frac{1.75 \sqrt{2} L (t_{in} + t_{cm}) \sigma_{cr}}{L + (t_{in} + t_{cm})}$$  \hspace{1cm} (16)

For shear failure, the equation proposed is,

$$V_{ms} = A_{vh} 0.15 \sqrt{f_m}$$  \hspace{1cm} (17)

The validation of the proposed equations was carried out by comparing the results with the experimental
investigations. The variation between the numerical and experimental results is shown in figure 6 for specimens A1 and A2. The percentage variation for the specimens is less than 15%. Hence, the equation holds good for infill wall with chicken mesh as skin reinforcement [29]. The failure loads for model is tabulated in table 6.

The experimental investigation's failure pattern was diagonal tension, and the same is inferred in the analytical investigations as the values are in close agreement with the experimental as shown in table 7.

| Model | $V_c$ (kN) | $V_v$ (kN) | VMII (kN) | $V_{mf}$ (kN) |
|-------|------------|------------|-----------|---------------|
| A1    | 18.78      | 5.97       | 15.321    | 4.596         |
| A2    | 22.51      | 8.06       | 17.03     | 5.11          |

The experimental investigation's failure pattern was diagonal tension, and the same is inferred in the analytical investigations as the values are in close agreement with the experimental as shown in table 7.

The percentage variation of displacement, when comparing with the experimental investigation was minimum for the equation proposed by Mainstone, and it was 3.14%. Also, the failure pattern observed during the experimental investigation was in the form of diagonal tension. Mainstone's equation is also derived based on the phenomenon of diagonal tension. Hence, Mainstone proposed the equation for further modifications, for infill wall with skin reinforcement.

**Acknowledgments**

We are very grateful to the reviewers for their appropriate and constructive suggestions to improve for our research work and article.

**REFERENCES**

[1] Pradhan, Prachand Man, Ramesh Kumar Maskey, and Prajwal Lal Pradhan. "Experimental study of partially masonry infilled reinforced concrete frame under lateral loading." In Advanced Engineering Forum, vol. 21, pp. 22-32. Trans Tech Publications Ltd, 2017.

[2] Asteris, Panagiotis G., S. T. Antoniou, Dimitris S. Sophianopoulos, and Ch Z. Chrysostomou. "Mathematical macromodeling of infilled frames: state of the art." Journal of Structural Engineering, vol. 137, no. 12, pp. 1508-1517, 2011.

[3] Mallick, D. V., and R. T. Severn. "The behaviour of infilled frames under static loading." Proceedings of the Institution of Civil Engineers, vol. 38, no. 4, pp. 639-656, 1967.

[4] Asteris, Panagiotis G., Christis Z. Chrysostomou, Ioannis Giannopoulos, and Paolo Ricci. "Modeling of infilled framed structures." In Computational Methods in Earthquake Engineering, pp. 197-224. Springer, Dordrecht, 2013.

[5] Pradhan, Prachand Man. "Equivalent strut width for partial infilled frames." Journal of Civil Engineering Research, vol. 2, no. 5, pp. 42-48, 2012.

[6] Pradhan, P. M., P. L. Pradhan, and R. K. Maskey. "A review on partial infilled frames under lateral loads." Kathmandu University Journal of Science, Engineering and Technology, vol. 8, no. 1, pp. 142-152, 2012.

[7] Vincent Sam Jebadurai, S., D. Tensing, and C. Freeda Christy. "Enhancing performance of infill masonry with skin reinforcement subjected to cyclic load." International Journal of Engineering, vol. 32, no. 2, pp. 223-228, 2019.

[8] Jebadurai, S. V. S., D. Tensing, P. M. Pradhan, and G. Hemalatha. "Enhancing performance of infill masonry with latex modified mortar subjected to cyclic load." In Structures, vol. 23, pp. 551-557. Elsevier, 2020.

[9] Holmes, Malcolm. "Steel frames with brickwork and concrete infilling." proceedings of the Institution of civil Engineers, vol. 19, no. 4, pp. 473-478, 1961.

[10] Polyakov, S. V. "Masonry in framed buildings: An investigation into the strength and stiffness of masonry infilling. Translation into English by GL Cairns." Gosdarstvenoeizdatel'stvoliteraturypostroitels'tvuiarkhiteckture: Moscow, 1956.
[11] Smith, Bryan Stafford. "Lateral stiffness of infilled frames." Journal of the Structural Division, vol. 88, no. 6, pp. 183-199, 1962.
[12] Asteris, Panagiotis G., Demetrios M. Cotsovos, C. Z. Chrysostomou, A. Mohebkhah, and G. K. Al-Chaar. "Mathematical micromodeling of infilled frames: state of the art." Engineering Structures, vol. 56, pp.1905-1921, 2013
[13] Stafford Smith, Bryan, and C. Carter. "A method of analysis for infilled frames." Proceedings of the Institution of Civil Engineers, vol. 44, no. 1, pp. 31-48, 1969.
[14] Haldar, Putul, Yogendra Singh, Dominik H. Lang, and D. K. Paul. "Comparison of seismic risk assessment based on macroseismic intensity and spectrum approaches using ‘SeisVARA’." Soil Dynamics and Earthquake Engineering vol. 48, pp. 267-281, 2013
[15] Mainstone, Rowland J. "Summary of Paper 7360. On the Stiffness and Strengths of Infilled Frames." Proceedings of the Institution of Civil Engineers vol. 49, no. 2, pp. 230, 1971
[16] Wood, Randal Herbert, MR Horne, W. Merchant, E. Lightfoot, BG Neal, HSL Harris, AJS Pippard Et Al. "Discussion. The Stability of Tall Buildings." Proceedings of the Institution of Civil Engineers, Vol. 12, no. 4, PP. 502-522, 1959
[17] Liauw, T. C., and K. H. Kwan. "Plastic Theory of Non-Integral Infilled Frames." Proceedings of the Institution of Civil Engineers, vol. 75, no. 3, pp. 379-396, 1983
[18] Smith, B.S. and Coull, A., 1991. Infilled-frame structures, Tall Building Structures: Analysis and Design. New York: Wiley, pp.168-183, 1991.
[19] Paulay, Thomas, and MJ Nigel Priestley. "Seismic design of reinforced concrete and masonry buildings."., pp. 135-146, 1992
[20] Saneinejad, Abolghasem, and Brian Hobbs. "Inelastic design of infilled frames." Journal of Structural Engineering, vol. 121, no. 4, pp. 634-650, 1995.
[21] Flanagan, Roger D., and Richard M. Bennett. "Bidirectional behavior of structural clay tile infilled frames." Journal of structural engineering, vol. 125, no. 3, pp. 236-244, 1999
[22] Kappos, Andreas J., Gregory G. Penelis, and Christos G. Drakopoulos. "Evaluation of simplified models for lateral load analysis of unreinforced masonry buildings." Journal of structural engineering, vol. 128, no. 7, 890-897, 2002
[23] Anić, Filip, Davorin Penava, Lars Abrahamczyk, and Vasilis Sarhosis. "A review of experimental and analytical studies on the out-of-plane behaviour of masonry infilled frames." Bulletin of Earthquake Engineering, vol. 18, no. 5, 2191-2246, 2020
[24] Wang, Chuanlin, Vasilis Sarhosis, and Nikolaos Nikitas. "Strengthening/retrofitting techniques on unreinforced masonry structure/element subjected to seismic loads: A literature review." The Open Construction and Building Technology Journal, vol. 12, no. 1, 2018.
[25] Cruze, Daniel, Hemalatha Gladston, Ehsan Noroozinejad Farsangi, Arnab Banerjee, Sarala Loganathan, and Sundar Manoharan Solomon. "Seismic Performance Evaluation of a Recently Developed Magnetorheological Damper: Experimental Investigation." Practice Periodical on Structural Design and Construction, vol. 26, no. 1, pp. 04020061, 2021
[26] McBride, Roy Thomas. "The behavior of masonry infilled steel frames subjected to racking." PhD diss., University of New Brunswick, Department of Civil Engineering, 1984.
[27] Dawe, J. L., and C. K. Seah. "Out-of-plane resistance of concrete masonry infilled panels." Canadian Journal of Civil Engineering, vol. 16, no. 6, pp. 854-864, 1989
[28] Akın, Emre, Erdem Canbay, Barış Binici, and Güney Özcebe. "Scale Effect on CFRP Strengthening of Infilled Reinforced Concrete Frames." Journal of Advanced Concrete Technology, vol. 13, no. 7, pp. 355-366, 2015
[29] Alireza Bahrami, Ali Mahmoudi Kouhi , "Compressive Behaviour of Circular, Square, and Rectangular Concrete-Filled Steel Tube Stub Columns," Civil Engineering and Architecture, Vol. 8, No. 5, pp. 1119 -1126, 2020. DOI: 10.13189/cea.2020.080538.