Civil Engineering and Architecture 9(4): 1198-1205, 2021 http://www.hrpub.org
DOI: 10.13189/cea.2021.090421

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

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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.*

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 (Δy), Initial stiffness (Ki), Ultimate loads (Pu), Ultimate displacement (Δu), Ductility (μ) 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 3DANSYS, 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.

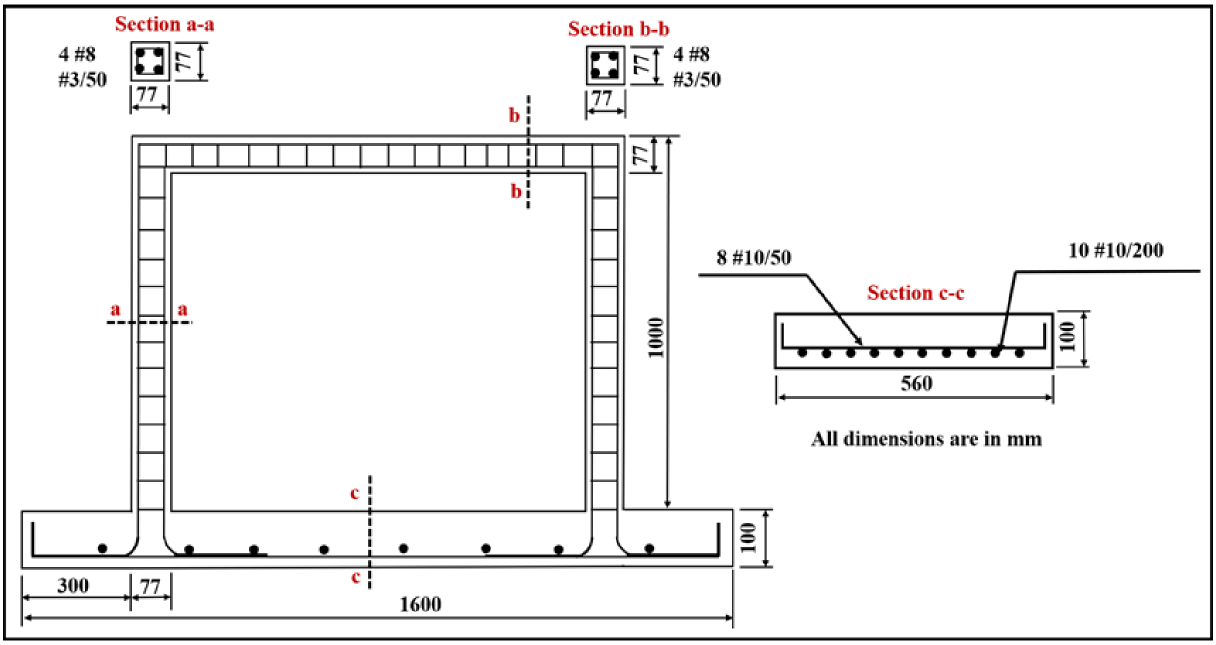

Figure 1. Reinforcement detail of the frame [25]

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

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.

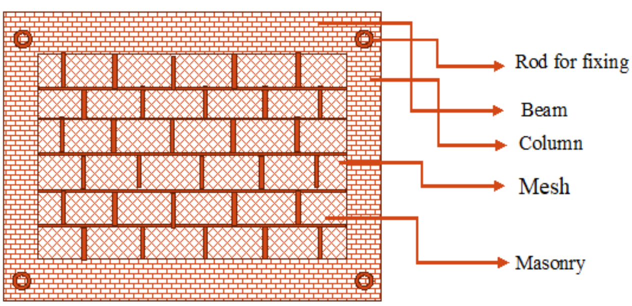

Figure 2. The schematic diagram for fixing Chicken mesh

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.

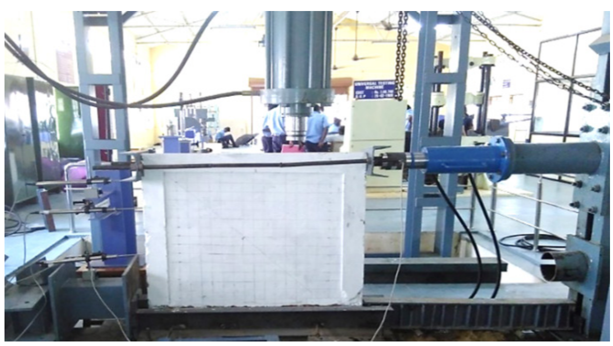

Figure 3. Experimental setup

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)

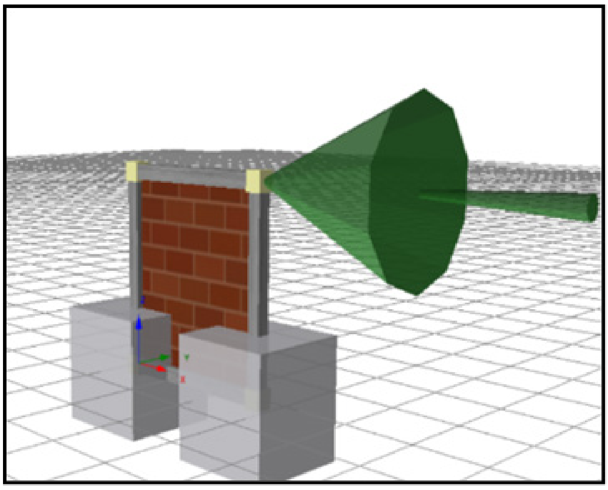

Figure 4. Cyclic load applied in SEISMOSTRUT

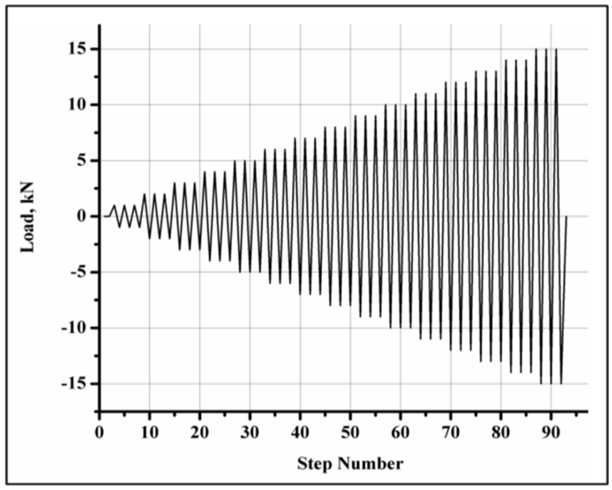

Figure 5. Loading pattern for cyclic loading

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.

Table 2. Strut width (wds) proposed by six researchers

Models	Strut Width(W_{ds})	W_{ds} (m)	Failure type		Eqn No.
			Infill	Frame	
Holmes (1961)	$w_{ds}=\frac{d}{3}$	0.471	shear failure		(1)
Stafford Smith & Carter (1969)	$w_{ds}=0.58(l/h)^{-0.445}\lambda H^{0.335d}(l/h)^{0.064}$	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)	$w_{ds}=\frac{0.95h\cos\theta}{\sqrt{\lambda H}}$	0.567	Diagonal compression and corner crushing		(4)
Paulay & Pristley (1992)	$w_{ds}=\frac{d}{4}$	0.35	Sliding shear failure, diagonal compression	Tension failure of columns, shear failure of beam/column	(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.

$$\lambda H = \left[h_{column} \sqrt[4]{\frac{E_{wall} t_{wall} \sin 2\theta}{4 E_{column} I_{column} h_{wall}}} \right] \quad (7)$$

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.

$$\min \begin{cases} \sqrt{\dfrac{2(M_{pj} + M_{pc})}{\gamma_p f'_m t h^2}} \\ \dfrac{1}{tan\theta} \sqrt{\dfrac{2(M_{pj} + M_{pc})}{\gamma_p f'_m t h^2}} \\ \dfrac{4M_{pj}}{\gamma_p f'_m t h^2} + \dfrac{1}{6\max\{1, tan\theta^2\}} \end{cases} \quad (8)$$

$$\gamma_p = 2.663m^2 - 1.37m + 0.406 \leq 0.45 \quad (9)$$

$$m = \frac{8M_{pj}}{f'_m t L^2} \quad (10)$$

Were,
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'm is the compressive strength of infill
t is the thickness of infill
h 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.

Table 3. Comparison for Infill wall without mesh (A1) Models

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 & Priestley	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 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 Vcr of the infill given as [28],

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

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

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

Were,
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 5.6% 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

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

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,

$$V_c = 0.50(\lambda h)^{-0.875} f'_m h(t_{in} + t_{cm}) cot\theta \quad (15)$$

Where f'_m - Compressive strength of infill with mesh

The tensile cracking load is proposed as,

$$V_{cr} = \frac{1.75\sqrt{2}L(t_{in} + t_{cm})\sigma_{cr}}{\left[\frac{L}{(t_{in} + t_{cm})} + \frac{(t_{in} + t_{cm})}{L}\right]} \quad (16)$$

For shear failure, the equation proposed is,

$$V_{mi} = A_{vh} 0.15\sqrt{f'_m} \quad (17)$$

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.

Table 6. Failure loads for the model

Model	V_c (kN)	V_{cr} (kN)	VMI (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.

Table 7. Comparison of analytical and experimental results

Model	Analytical (kN)	Experimental (kN)	% Variation
A1	18.78	16.20	13.7
A2	22.51	19.30	14.26

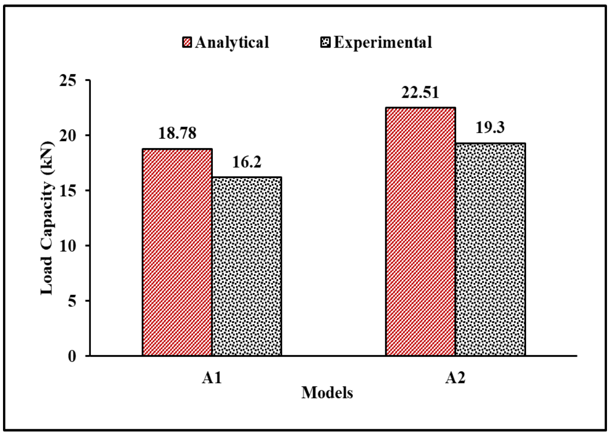

Figure 6. Comparison of experimental and numerical load capacity for A1 and A2

4. Conclusions

The experimental and analytical investigations carried out for the implementation of chicken mesh as skin reinforcement to enhance the diagonal tension, and shear capacity of the masonry infill wall have been obtained from this study. The values of failure load calculated numerically were compared with experimental results, and it can be observed that the variation between experimental and analytical result is 13.7% for A1 and 14.26% for A2 specimen. From the Comparison, it can be seen that 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.

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