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DESIGN OF CONCRETE FRAMES WITH MASONRY INFILL WALLS

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Abstract

Based on recent experimental evidence, a design procedure of concrete frames infilled with masonry walls is proposed. The strength of the system is estimated as the sum of strength of the masonry walls and the strength of the frame. The strength of the masonry walls is estimated using Canadian code while the frame strength is equal to the force required for the bare frame to undergo a prescribed drift. The frame strength is calculated with an elastic analysis. Nonlinear behavior is taken into account through reduction of inertia for the columns. Using this procedure, estimations for cracking and maximum strengths are obtained. Experimental results show that the contribution of horizontal reinforcement to the strength of the system is dependent on the relative stiffness of wall and frame. Current estimations of strength do not consider that parameter leading to overestimation of strength when flexible frames are used.

Keywords: infilled frames, infill walls, masonry, shear strength, cracking strength

1. Introduction

Infilled reinforced concrete frames with masonry walls are designed considering the frame as the main load-resisting system. It is argued that the contribution of the infill walls is a reserve of resistance. However, this consideration does not always lead to safer designs. Passed seismic experiences have left the learning that the infill walls lead beneficial and adverse effects to the behavior of the structure, depending on the physical and mechanical characteristics of its components [1, 2].

Several researchers have studied the behavior of infilled frames. The main observations are: at the global level, infill walls significantly increase the stiffness and strength of the structure and improve the energy dissipation capacity of the system; at the local level, interaction between panel and frame produce highly concentrated normal stresses at two diagonally opposed corners, at the other corners, separation of the wall from the frame occurs. Therefore, the presence of infill walls transforms the internal force diagrams obtained from an analysis of the structure without walls.

In order to evaluate the stiffness and the strength of infilled frames, there is a wide variety of analytical techniques. Different models based on the concept of equivalent diagonal strut have been proposed [3, 4] that vary on the way to obtain the width of the strut. Canadian Standard [5], New Zealand Standard [6] and Mexican Code, NTCM-2004 [8], allow the use of the above-mentioned models. However, there is not a simplified analysis model universally accepted, able to simulate local effects generated by the wall-frame interaction.

In this investigation, a simple procedure to estimate the strength of the system, concrete frame infilled with masonry walls, is described. The study is based on the results from a recent experimental investigation [7] that included tests of six specimens scaled 1:2. A summary of that investigation is presented in the next section.

The Canadian diagonal strut method and the proposed method were applied to the specimens tested and compared.

2. Experimental Investigation

To ease the presentation a brief summary of the experimental investigation used in this paper [7] is given next. Six specimens, scaled 1:2, were tested in pairs, each specimen in the pair having identical characteristics except for the size of the frame columns that in one case represented the columns of the six floor building and the other with columns of the three story one (Fig. 1).

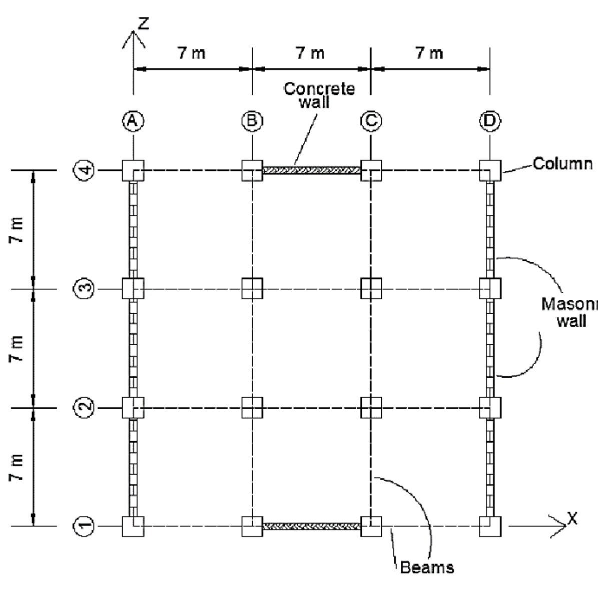

Fig. 1 - Floor plan of the prototype building

In the first pair, the infill walls did not have any confining elements nor horizontal reinforcement, in the second pair tie-columns and tie-beams were added to the infill walls and finally for the third pair, horizontal reinforcements anchored in the tie-columns were added to the infill walls considered in the second pair. The specimens were subjected to pseudo-static increasing cycles of lateral deformation.

Fig. 2 and 3 illustrate details, dimensions and materials properties of the specimen's frame corresponding to the six and three story buildings, respectively.

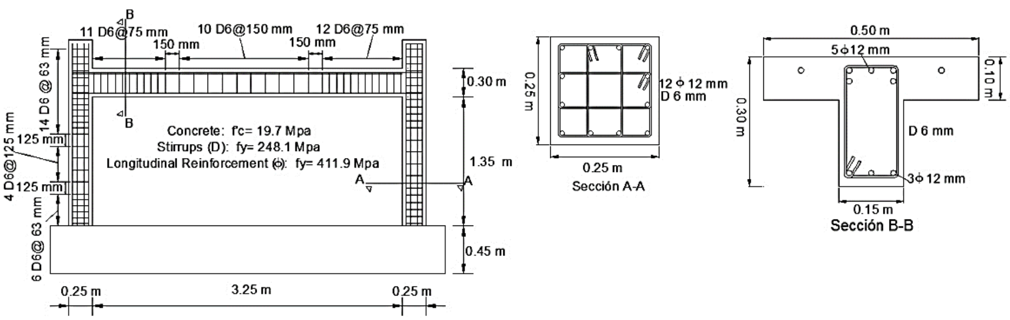

Fig. 2 - Reinforcing details and dimensions of frame corresponding to six story building

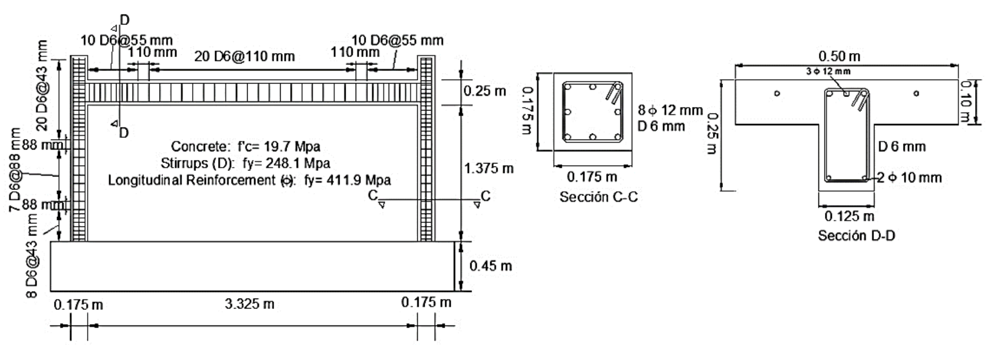

Fig. 3 - Reinforcing details and dimensions of frame corresponding to three story building

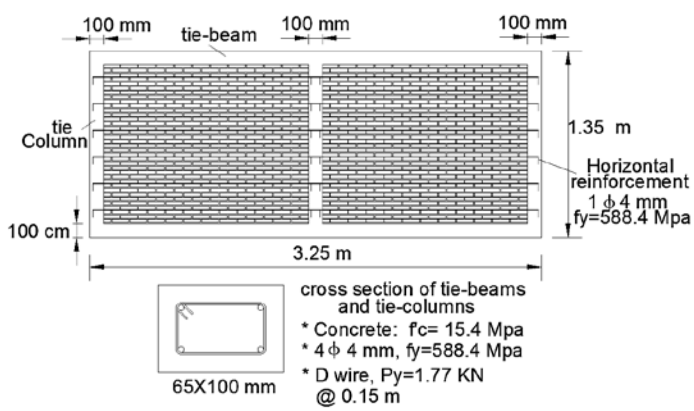

Fig. 4 - Reinforcing details and dimensions of infill wall corresponding to the six story buildings

Table 1 Specimen characteristics

Specimen	Col Size (mm)	Tie-col (cm)	Tie-Beam	Horizontal Reinforcement
MD6NSR	250×250	No	No	No
MD6N	250×250	65x100	65x100	No
MD6NRH	250×250	65x100	65x100	1 ϕ 4 mm @6 c
MD3NSR	175×175	No	No	No
MD3N	175×175	65x100	65x100	No
MD3NRH	175×175	65x100	65x100	1 ϕ 4 mm @6 c

In the columns, reinforcement percent ratios were 2.4% and 3.3%. Plastic hinges could develop at the end of the frame members so that the separation of the stirrups was reduced in these zones.

The overall dimensions of the wall corresponding to the six story building were 1.35 m height and 3.25 m long, which represents an aspect ratio H/L equal to 0.415. The global dimensions of the panel corresponding to the six story building were 1.375 m height and 3.325 long, having an aspect ratio equal to 0.414. Reinforcement details and dimensions of the infill walls are shown in Fig. 4 (Frame and wall are shown separately in Figures 2 to 4, however it is clear that every specimen consisted on a infill wall as the of Figure 4 enclosed by a frame as the of Figure 2 or 3).

2.1 Test Results

The loading-drift curves presented similar properties and they can be characterized in four stages. At low levels of lateral displacements, masonry and frame acted as a monolithic composite structural system. The initial stiffness of the specimens was calculated as the slope of the line connecting the maximum point of the first cycle and the origin. The initial stiffness of each specimen is shown in Table 2.

When separation between wall and frame occurred, the wall acted as a diagonal strut that stiffened the flexible frame. However, compared with the first stage of the tests, the stiffness was considerably smaller. The average stiffness in the second stage of the specimens with smaller and larger size columns were 18.52 kN/mm and 26.81 kN/mm, respectively. In this second stage of the tests, first inclined cracking on the wall occurred. However, the first cracking cannot be observed in the drift-lateral load envelope curve. The average drift at the first pronounced slope change of the enveloped curves were 0.0047 and 0.0060, in the specimens with smaller and larger size columns, respectively.

In most specimens, inclined crack appear first followed by horizontal cracks. The lateral stiffness of the system reduced gradually until the maximum shear strength was attained. The last stage corresponded to descending branch produced due the softening of the material, characterized by gradual crushing of the masonry.

The envelope curve of specimen MD6N is shown in Fig. 6; and idealized curve showing the four stages of the test is shown with a dashed line.

The Table 2 summarizes the values of the initial stiffness (K_i) and second stage stiffness (K_s), drift at the first inclined cracking (Δ_{cr}) and pronounced slope change (Δ_{pb}) and the corresponding lateral loads, maximum strength (V_{max}) and drift at maximum lateral load (Δ_{max}) developed for different specimens.

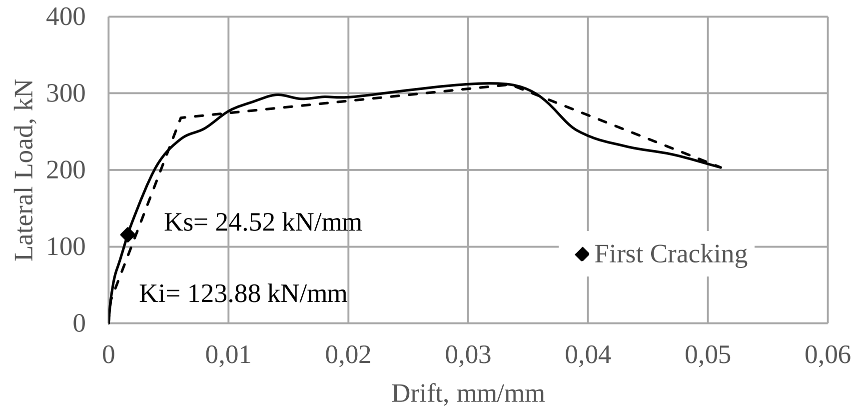

Fig. 6 - Positive envelope curve of specimen MD6N.

Table 2 - Summary of main parameters of envelope curves

Specimen	K_i (kN/mm)	K_s (kN/mm)	First Cracking		Pronounced slope change		Maximum Strength	
			Δ_{cr}	V_{cr} (kN)	Δ_{pb}	V_{pb} (kN)	Δ_{max}	V_{max} (kN)
MD6NSR	117.75	25.04	0.0014	98.88	0.0060	233.38	0.0242	267.71
MD6N	123.88	24.52	0.0016	115.36	0.0060	240.44	0.0335	311.37
MD6NRH	115.36	30.87	0.0014	114.38	0.0060	264.67	0.0240	313.72
MD3NSR	19.68	16.67	0.0028	77.99	0.0040	108.89	0.0100	162.55
MD3N	88.68	18.36	0.0014	76.91	0.0040	119.38	0.0241	148.23
MD3NRH	103.73	20.53	0.0014	68.57	0.0060	150.88	0.0241	194.04

Envelopes for specimens corresponding to six and three story buildings are shown in Figs. 7 and 8, respectively. It is observed that the horizontal reinforcement had a greater contribution to maximum load in specimens with a greater wall/frame stiffness ratio. In both figures, it is observed that the confinement elements do not increase significantly the lateral strength of the system; however, the maximum load is reached at higher drift than those without confining elements, maintaining the integrity of the wall.

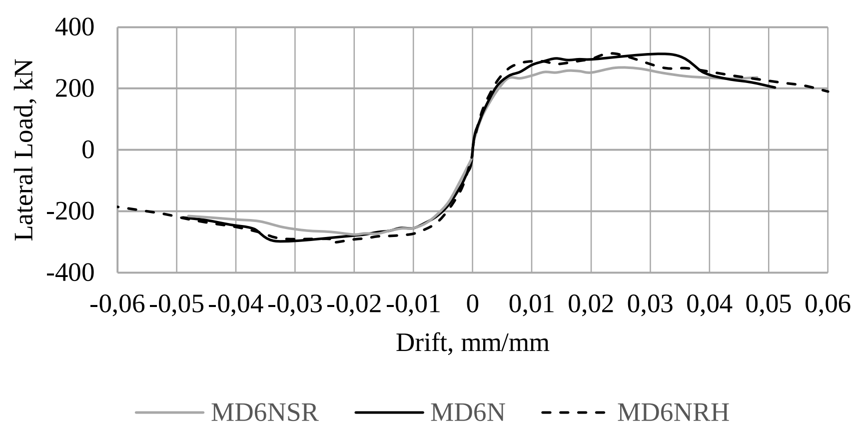

Fig. 7 - Envelope curves of specimen corresponding to six story building

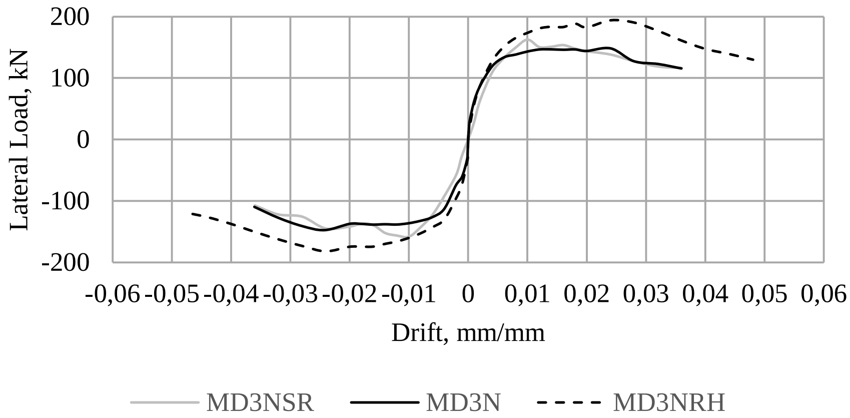

Fig. 8 - Envelope curves of specimen corresponding to three story building

Similar crack patterns and crack sequences were developed by the six tested specimens. At earlier stage of the tests, separation between wall and frame was manifested through cracks in the interface of these elements. It was very evident that according to the sense of the lateral load, wall and frame was in contact in opposite corners, while separation took place at the others opposite corners.

The first diagonal cracking in each specimens was detected by visual inspection and, in all cases, it was developed on central zone of the wall. The specimens without reinforcing elements developed well-defined first inclined cracks. The specimens with tie-beams and tie-columns developed cracking on the wall more distributed than the specimens without confining elements. The specimens with horizontal reinforcement also had more distributed cracking. In this pair of specimens was difficult to identify the first crack. Yield in the load-drift curve did not happen when the first diagonal cracking was detected, but rather yield strength was reached until the first diagonal cracking was extended toward the ends of the wall. The values of the lateral load and drift at, both, the first diagonal cracking and the yield strength of the specimens are shown in Table 2.

After the first inclined cracking, a combination of inclined cracks and horizontal sliding was developed on the wall. As the tests progressed, horizontal sliding gained greater importance. It was notorious to observe, in the case of the specimens with confinement elements, sliding planes were developed uniformly each six courses. In the specimens MD6NSR y MD3NSR, only two main sliding planes were formed at the middle height of the wall.

In the specimens with confining members, the shear strength of the central tie-column had an important role. In these specimens, the ultimate lateral load strength was reached when the central tie-column failed when a crack crossed it. In all specimens, combination of sliding of the infill wall and diagonal tension in the connection of beam-column of the frame was the dominant failure mode of the system. The final crack pattern of specimen MD3N is shown in Figure 9.

Figure 9. Crack pattern in specimen MD3N

3. Proposed Design Procedure

3.1 Resistance of infill walls.

Canadian Standard provide a complete set of recommendations to estimate masonry shear strength; it considers different failure modes and the aspect ratio of the wall. Three failing modes mechanism are considered, strut compression failure, diagonal failure and sliding failure. Strut compression failure is calculated with Eq. (1),

$$P_r = (0.85\,\chi\,\phi_m\,f'_m)\cdot A_e \tag{1}$$

where χ is a factor used to account for direction of compressive stress in a masonry member, which was taken as 0.50; f'_m is the compressive strength of masonry normal to the bed joint; and A_e is the effective cross-sectional area of masonry ($A_e = t_e \cdot w_e$).

Diagonal tension resistance is obtained through Eq. (2).

$$V_m = \phi_m(v_m b_w d_v + 0.25P_d)\cdot\gamma_g \tag{2}$$

In Eq. (2), v_m is the shear strength of masonry, which was calculated equal to 0.16 · $(f'_m)^{1/2}$; b_w is the overall web width; d_v is the effective depth ($d_v = 0.8l_w$); P_d is the axial compressive load on the section under consideration, which was considerated equal to 0; and γ_g is a factor to account for partially grouted walls or ungrouted walls, which for the cases considered is 1.0 since solid brick masonry was used.

When reinforcement in the mortar joins is included, Canadian standard proposes the Eq. (3) for diagonal tension resistance,

$$V_m = \phi_m(v_m b_w d_V + 0.25P_d)\gamma_g + 0.6\phi_s A_v f_y(d_v/s) \tag{3}$$

where A_v and f_y are cross-sectional area of shear reinforcement and yield strength of reinforcement, respectively and s is spacing of reinforcement.

Sliding shear resistance is estimated using Eq. (4).

$$V_{sr} = 0.16\phi_m(\sqrt{f'_m}A_{uc} + \mu P_1) \tag{4}$$

In Eq. (4), A_{uc} is the uncracked area of the cross-section ($t_e \cdot d_v$); μ is the friction coefficient, which is equal to 1.0 for masonry to masonry planes; and P_1 is the compressive force acting normal to the sliding plane. P_1 is normally taken as P_d plus 90% of the factored vertical component of the compressive forces resulting from the diagonal strut action.

Table 2 shows the resistance of the infill wall of the tested specimens at the first inclined cracking and the maximum resistance. The resistance at the first inclined cracking was related with the diagonal tension resistance. It was considered that at the first cracking, horizontal reinforcement did not contribute at the resistance of the wall, which can be verified in Table 1, where similar values for this parameter on the specimens with and without horizontal reinforcement can be observed.

Table 3 - Resistance of the infill walls

Specimen	Resistance at First inclined cracking (kN)	Maximum Resistance (kN)
MD6NSR	47.8	72.7
MD6N and MD6NRH		76.5
MD3NSR	49.0	74.3
MD3N and MD3NRH		78.1

Confinement elements increased the strut compression resistance of the wall, which it's not considered in Eq. (1). On specimens with tie-columns and tie- beam the maximum resistance was reached by sliding shear resistance.

3.2 Resistance of the frames.

To estimate the maximum strength of the system, the frame's contribution is calculated equal to the required force to deform the bare frame up to the drift of failure, considered to be 1%. Stiffness of the beam that transfers lateral load is taken into count in the analysis of bare frame. It was observed in all tests that at a drift equal to 0.01, specimens reached between 90% and 100% of their maximum resistance. A similar drift level at maximum strength was reported in reference [9] for walls with and aspect ratio of 1 and different amounts of horizontal reinforcement. A reduced inertia moment was considered in the analysis, which was equal to 0.50 of gross inertia moment.

To estimate the cracking strength, the contribution of the frame is calculated equal to the required force to deform the bare frame at drift equal to 0.0014 that correspond to the first inclined cracking in the most specimens (see Table 2). At such drift level, no cracks were detected on the specimens's frames, so that moments of inertia of uncracked sections were considered in the analysis.

Table 4 – Resistance of the frames

Specimen	Resistance at First inclined cracking (kN)	Maximum Resistance (kN)
MD6NSR, MD6N and MD6NRH	73.2	261.5
MD3NSR, MD3N and MD3NRH	21.0	75.1

3.3 Resistance of the system.

The strength of the system is calculated equal to the sum of the contributions of the masonry and the frame, in Table 5 the results were summarized. A comparison between analytical and experimental values was carried out; a relative error was calculated which is also shown in Table 5.

In general, the estimated strengths are acceptable, most specimens presented a percentage of error lower than 10% for both, resistance at the first inclined cracking and maximum resistance.

From the tests, it was observed that the contribution of the horizontal reinforcement and the confinement elements on lateral strength of the system depend of the stiffness wall/frame ratio, i.e. a very small contribution of the reinforcement was observed when the frame had small columns and a considerably larger contribution was observed when larger columns were used.

Table 5 – Resistance of the system

Specimen	First inclined cracking		Maximum Resistance	
	Resistance (kN)	% error	Resistance (kN)	% error
MD6NSR	121.0	22.3	334.2	24.8
MD6N		4.9	338.0	8.6
MD6NRH		5.7		7.7
MD3NSR	70.0	-10.2	149.4	-8.1
MD3N		-8.9	153.2	3.3

MD3NRH		2.1		-21.0

This parameter is not considered for the calculation of the maximum strength leading to and over estimation of the strength particularly n frames with small columns. This observation is not included in the Canadian Standard's recommendations for infill wall resistance, so that a higher percentage of error was presented on maximum resistance of MD6NSR and MD3NRH.

4. Diagonal Strut Method

Analytical models based on the concept of the equivalent diagonal strut that considers the structure as an equivalent braced frame system with a diagonal compression strut replacing the infill wall, provide an accurate prediction of the global behavior of the system [10]. In these analytical models, numerous empirical equations are employed, through which, researchers have tried to relate the mechanical and geometrical properties of infilled frames with some structural parameters, such as stiffness, lateral strength and the contact length between the frame and the infill panel. The structural parameters above mentioned are obtained as a function of the dimensionless parameters proposed in reference [3] and [4], which express the relative stiffness between the wall and the frame.

The diagonal strut model of Stafford-Smith [3] is described in Figure 10 and Eq. (5), (6) and (7). The mechanical properties for the strut are the same to the masonry wall. In this model, the width of the diagonal strut depends on several factors as

α_h: Contact length between the panel and the column

α_L: Contact length between the panel and the beam

E_C: Modulus of elasticity of the concrete frame

E_m: Modulus of elasticity of the masonry

I_C: Moment of inertia of the column

I_b: Moment of inertia of the beam

t_e: Thickness of the panel

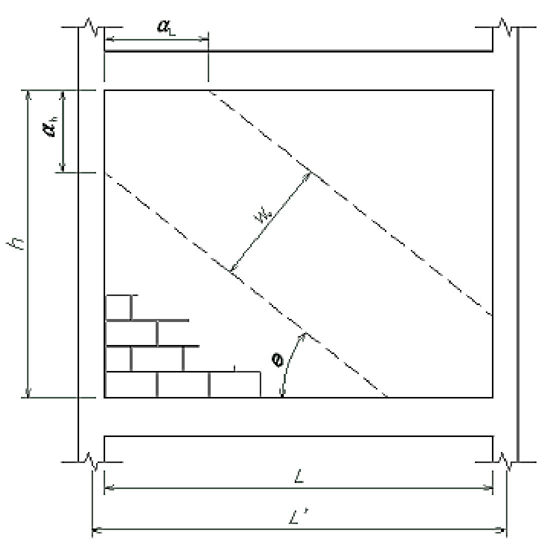

Figure 10.Diagonal strut model

$$\alpha_h = \frac{\pi}{2} \cdot \sqrt[4]{\frac{4 \cdot E_c \cdot I_c \cdot h}{E_m \cdot t_e \cdot \sin(2\theta)}} \tag{5}$$

$$\alpha_L = \frac{\pi}{2} \cdot \sqrt[4]{\frac{4 \cdot E_c \cdot I_b \cdot L}{E_m \cdot t_e \cdot \sin(2\theta)}} \tag{6}$$

$$W_0 = \sqrt{\alpha_h^2 + \alpha_L^2} \tag{7}$$

Canadian Standard considers a diagonal strut model for infilled frames, in which the width of the diagonal strut corresponds to the lower of half the width proposed by Stafford-Smith [3] or one quarter of the diagonal length.

Elastic analysis of the specimens through diagonal strut method was carried out. The lateral resistance was obtained through the analysis of the braced frame system. The resistance of the system was equal to the required force so that the infill wall reaches first inclined cracking and maximum resistance. The provisions of the Canadian Standard for infill wall's resistance were considered. The results are presented in Table 6.

Comparison between diagonal strut method and experimental results are presented too. It is observed that for the resistance at the first inclined cracking, diagonal strut method did not have a good agreement with the experimental results. For maximum resistance, the most specimens had good agreement with experimental results, however, the corresponding drift was around 0.0059 in all the cases, which is smaller than the observed experimental values, as expected from an elastic analysis.

Table 6 – Resistance of the braced frame system

<table>
<tr><th rowspan="2">Specimen</th><th colspan="2">First inclined cracking</th><th colspan="2">Maximum Resistance</th></tr>
<tr><th>Resistance (kN)</th><th>% error</th><th>Resistance (kN)</th><th>% error</th></tr>
<tr><td>MD6NSR</td><td rowspan="3">205.5</td><td>100.8</td><td>312.4</td><td>16.7</td></tr>
<tr><td>MD6N</td><td>78.1</td><td rowspan="2">328.5</td><td>0.3</td></tr>
<tr><td>MD6NRH</td><td>79.7</td><td>-0.4</td></tr>
<tr><td>MD3NSR</td><td rowspan="3">91.0</td><td>16.7</td><td>140.2</td><td>-13.7</td></tr>
<tr><td>MD3N</td><td>18.3</td><td rowspan="2">147.3</td><td>-0.6</td></tr>
<tr><td>MD3NRH</td><td>32.7</td><td>-24.1</td></tr>
</table>

5. Conclusions

In general, the proposed design procedure provide a good estimation of the strength of the system. Most specimens presented a percentage of error lower than 10% for both, resistance at the first inclined cracking and maximum resistance.

The strut model gave a good prediction of the maximum strength but a very large overestimation of the cracking strength. This may have a considerable impact when analyzing the serviceability limit state (performance level).

In order to improve the proposed design procedure, contribution of the horizontal reinforcement and the confinement elements, as well as, their relationship with the wall/frame relative stiffness, must be studied further. In addition, for the analysis of the bare frame, It is necessary include a model of evolution of the moment of inertia.

The proposed procedure is akin to a future development of a full performance level design as it is driven by displacement rather than strength.

5. References

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