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LATERAL RESPONSE EVALUATION OF STEEL FRAME STRUCTURES WITH MASONRY INFILLS

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Abstract. *An established practice for the design of steel frame structures is to ignore the contribution of non-structural elements, such as masonry infilled walls. Even though this is considered a pro-safety simplification, it removes the opportunity to predict more realistically the actual structural response, on one hand, and to evaluate non-structural damage, particularly under less severe earthquakes, on the other. This study attempts to evaluate the contribution of masonry infills to the response of typical steel frames. In particular, a selection of multi-storey frames designed according to Eurocode 3 and 8, ignoring the presence of infills, is examined. The infills are then modeled, using an equivalent diagonal strut scheme. Their contribution to the structural response is evaluated through both linear and nonlinear analyses, for variable ratio of wall openings and number of storeys. The response characteristics are compared with the respective ones of the bare steel frames. The results reveal a significant increase in stiffness, as well as improved lateral resistance. Moreover, in regard to damage limitation requirements, the effectiveness of the infilled frames is significantly enhanced.*

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1 INTRODUCTION

The contribution of non-structural elements in the analysis and design of steel frame structures is normally ignored, partly due to the inherent uncertainties on masonry materials and construction. However, it has been indicated by experimental and analytical studies [1, 2] that the impact of masonry infill in the seismic response of frame structures, can become significant both in terms of stiffness and resistance and therefore necessary to be accounted for.

For the modeling of masonry infills under global frame analysis, two practical approaches have emerged: micro-modeling and macro-modeling. With micro-modeling, the interaction between every brick unit and the mortar is included in a discrete form, with characteristics that are directly connected to the mechanical properties of the materials. This approach is considered the most accurate one, but unfortunately can prove time consuming and hard to implement. With macro-modeling on the other hand, the influence of the infill is represented with sets of equivalent struts, reflecting infill failure modes and characteristics that are normally estimated through analytical or empirical methods. Macro-modeling has evolved from single diagonal strut implementations [3] to multi-strut assemblies that are capable to capture the corner interactions between masonry and frame members [4-8]. Even though multi-strut models are more accurate than single ones, they are also less intuitive and require much more parameters to be properly setup. Detailed and in-depth state-of-the art works can be found in Asteris et al. 2011 & 2013 [9-10], and Mohyeddin et al. 2017 [11].

An aspect of the infill geometry that greatly affects in-plane behavior is the presence of openings. Mosalam et al. (1997) [12] studied experimentally the response of infilled frames with openings under cyclic loading. Also, Kakaletsis and Karayannis (2008) [13] examined experimentally the earthquake response of RC infilled frames with openings and suggested an equivalent strut scheme. Tasnimi and Mohebkhah (2011) [14] studied experimentally steel frames and suggested analytical models for infills with window and door openings. Asteris et al. (2012) [15] have proposed a reduction factor for the stiffness of infills with openings.

In this work the performance of steel moment-resisting frames, with variable ratio of openings and number of storeys, that were designed as usual, ignoring the influence of masonry infills, is evaluated under lateral loading conditions, using diagonal strut infill modeling.

2 DESCRIPTION OF EXAMINED STEEL FRAMES

A parametric analysis has been undertaken in order to investigate the in-plane influence of masonry infills in the response of 2D moment-resisting steel frames. Specifically, four regular frame designs were considered, with 4, 8, 10 and 12 storeys, each 3m high. The number of bays has been kept constant to three, with each bay 5m long. The frames were designed for both gravity and seismic loading according to Eurocodes 3 and 8 [16,17], ignoring masonry infills completely, as typically is done in practice. For lateral stability, moment resisting framing was selected, which relies on the rotational stiffness and the bending resistance of the beam-to-column joints to successfully transfer lateral loads to the ground.

The design process has been performed through the commercially available program SAP2000 [18], with the following design parameters: steel grade: S355, uniform dead load: 15 KN/m, uniform live load: 20 KN/m, seismic ground acceleration: 0.24g, soil type: B, seismic behaviour factor: 5.0.

In Figure 1, the four designed frames are displayed. Selection of the beam cross sections was dictated by the gravity loading. Column profiles on the other hand, were determined by the joint resistance requirements and more specifically, by the requirement of the joint component column web panel in shear. This component can be reinforced through a supplementary web plate that is welded upon the column web, but according to EN1993-1-8 [19], the thickness of this plate cannot be bigger than the thickness of the column web itself. Therefore, the capacity of the joint to transfer gravity or seismic forces is directly linked to column size.

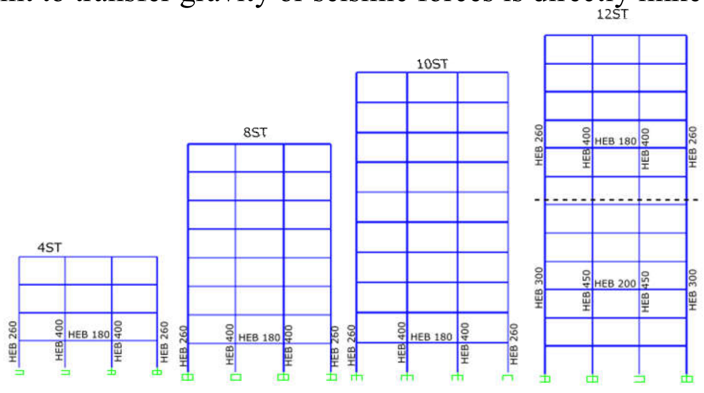


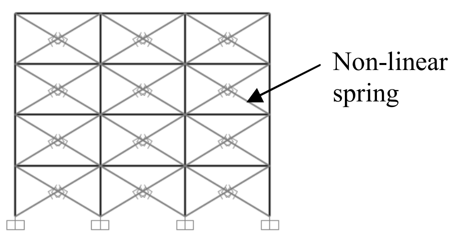
Figure 1: Steel frames that were examined.

3 EVALUATION OF FRAME BEHAVIOR WITH MASONRY INFILLS

3.1 General

In this work, the modeling of the infills was accomplished using single diagonal compression struts, as a balanced compromise between accuracy and required effort to prepare a full scale model, with commonly available frame analysis software. In particular, the main shortcoming of single strut modeling is the representation of shear and bending moment interactions between the infill and the surrounding members. However, it requires significantly less parameters for input and its performance against experimental pseudo-dynamic tests [15] has been found less accurate than multi-strut models, but in satisfactory agreement with them.

In Figure 2, the 4-storey frame is displayed, with diagonal compressive springs applied to every panel. The member elements are positioned along their centerlines and the length of each spring element is defined by the nodal distance of diagonally opposite beam-to-column joints. The additional time for the preparation of such a model was minimal and comparable to modeling with bracings, provided that the force-deformation characteristic of the springs is previously established. This is considered a decisive factor for a more wide-spread implementation of such an approach by the steel construction industry.



Masonry properties	
Thickness t_w	180 mm
Compressive strength f_m	4.1 MPa
Young's Modulus E_w	2.3 GPa

Figure 2: Steel frame model with diagonal struts for the simulation of infills and masonry properties

The force-deformation characteristic of the springs is nonlinear, with only the compressive path being active, as illustrated in Figure 3. The initial segment of the characteristic corresponds to the elastic linear range with initial stiffness S_{ini}. The firstly encountered limit corresponds to the first crack on the masonry that occurs for a strut force F_{cr}, and deformation δ_{cr}. Subsequently, the ultimate load limit, occurs at force F_m and deformation δ_m, after which the response deteriorates, up to a residual force level, denoted with force F_r and deformation δ_u.

The minimum required parameters for the complete description of the force-displacement characteristic are S_{ini}, F_m, k, a, b and m, according to the following relationships:

- $S_{\sec} = kS_{ini}$
- $F_{cr} = aF_m$
- $F_r = bF_m$
- $\delta_u = m\delta_m$

Parameter $a = F_{cr} / F_m$ is taken equal to 0.8, similar to existing literature practices [7,20], while parameter $b = F_r / F_m$ usually ranges between 0.02-0.4 [4,7,20] and a value equal to 0.2 is adopted here. The parameter k that relates secant stiffness towards the ultimate load, $S_{\sec}$, with the initial stiffness, S_{ini}, is taken equal to 0.5, based on the characterization of experimental data [21] and available analytical modeling [4]. For the needs of estimating the ultimate displacement δ_u, the negative stiffness of the third branch of the force-deformation law has been approximated in a range of 0.01-0.1 of initial stiffness [22], with value 0.1 corresponding to quite brittle masonry with weak framing. Conservatively, a value of m equal to 5 is considered here. The determination of S_{ini} and F_m is described in the next two sections.

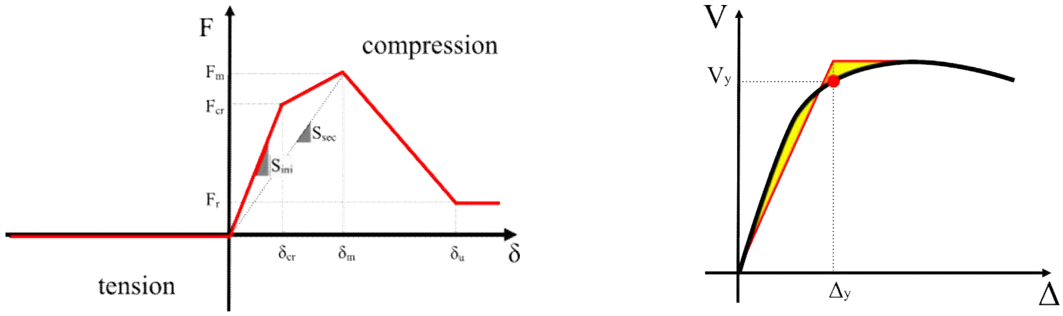


Figure 3: Force-deformation law of the diagonal struts (left) and capacity curve yield point definition (right).

3.2 Initial Stiffness

The width w of an equivalent diagonal strut may be found according to equation 1, which was first proposed by Mainstone and Weeks (1971) [3] and was included in FEMA-306 [2]:

$$w = 0.175\lambda_h^{-0.4} d \tag{1}$$

where, d is the diagonal length of the masonry infill and λ_h given by the following equation:

$$\lambda_h = \sqrt[4]{\frac{E_w t_w \sin 2\theta}{4EIh_w}} \cdot h \tag{2}$$

where E_w is the modulus of elasticity of the masonry infill, t_w its thickness, h_w the net height of infill panel, EI the flexural rigidity of the columns, h their height, measured between beam centrelines, and θ the angle, so that, $\tan\theta = h_w / L_w$, where L_w the net length of infill.

When openings exist in the infills, the thickness of the equivalent diagonal strut is modified, by multiplying the value found from equation 1, with a reduction factor λ, that has been proposed by Asteris et al. (2012) [15], and is shown in the following equation:

$$\lambda = 1 - 2a_w^{0.54} + a_w^{1.14} \tag{3}$$

where α_w is the ratio of the openings relative to the total infill panel area.

3.3 Strength

The resistance of masonry infills is rather complicated to predict analytically. Several failure mechanisms can be realized and the ultimate load that can be sustained for each one is not only dependent on the infill characteristics, but also on detailing and construction quality. Typical in-plane failure mechanisms include [2] compression failure, diagonal tension failure, shear sliding failure and general shear failure. Only the compression strength has been considered for our evaluation, which, in the diagonal direction, can be expressed as:

$$V_c = wt_w f'_{m90} \tag{4}$$

where f'_{m90} the masonry compressive strength in the horizontal direction, that can be assumed 50% of the respective stacked prism strength, and the parameters *w, t* as before.

3.4 Modeling of members of the steel frame

Steel columns and beams members were modeled taking into account material and geometric nonlinearity. For material nonlinearity, a lumbed plasticity approach has been adopted, with the properties of individual plastic hinges determined through discretization of the cross-sections to fiber layers. Offset distance from centerline joints, equal to 5% of the member length were designated for the location of plastic hinges.

4 RESULTS AND DISCUSSION

The four test frames were analysed in SAP2000[18], using static pushover analysis with uniform lateral loading, for six cases of infill opening ratios a_w: 0, 0.1, 0.25, 0.5, 0.75 and 1. The case a_w=1 corresponds to the bare frame while a_w=0 to solid infills without any opening. The normalized capacity curves (base shear to building weight ratio *V/W* vs. roof drift to building height ratio *Δ/H*) are shown in Figure 4. The red dot on each curve, marks the yield point, which is calculated from the yield displacement of the bilinear approximation (Figure 3), which is regressed until the areas between the bilinear and the actual curve are balanced out. For this calculation, the actual curve is considered up to its maximum capacity or its capacity at 10% drift, whichever is bigger.

The masonry infills significantly influence the response both in terms of initial stiffness *K* and capacity. Specifically, the increase of initial stiffness, compared to the bare frames, is depicted in Figure 5 and proves quite drastic, reaching levels of 16x magnification for a_w=0, while for a_w=0.25~0.5 the increase ranges between 1.8 and 4.4, which is still considered remarkable. For a_w=0.75 on the other hand, the change is insignificant. The described behavior is uniform for all different number of storeys. Also, similar increase, but slightly smaller, can be observed for the secant stiffness targeting the yield point.

In terms of capacity, a substantial increase is also observed for the frames with infills. As shown in Figure 5, the yield base shear for a_w=0 becomes 2-3 times larger compared to the bare frames, and for a_w=0.25 the increase lies between 1.15 and 1.40, while for larger a_w it becomes rather insignificant. This increase however is accompanied with a notable change of the shape of the capacity curve, the appearance of limit point, after which the capacity drops to a rather stable residual capacity, which for small a_w remains quite higher, than the bare frame. This post-critical capacity drop proves more severe for frames with more storeys.

The drifts on the other hand, typically decrease due to masonry infills. The comparison of the roof drifts at yield of the infilled frames relative to the bare frame ones, reveal a close to linear increase with the infill opening ratio a_w, as can be seen in Figure 5. For a_w=0, the yield roof drift becomes approximately five times smaller than the drift of the bare frame. The behavior remains uniform for all different frames. The reduced drifts can prove valuable for the verification against damage limitation seismic requirements [17]. Specifically, damage limitation is satisfied when design intestorey drift ratios are smaller than 1%, in buildings with non-structural brittle materials attached to the structure. In this context, the ratios of required base shear to induce a 1% roof drift ratio (RDR), relative to the bare frame, are depicted in Figure 5, for all different frames and infill openings. It is evident that substantially bigger base shear is needed (approximately 4 times if a_w=0, and 2 times if a_w=0.25), to exceed the 1% RDR in infilled frames. The relationship proves quite close for all different frames.

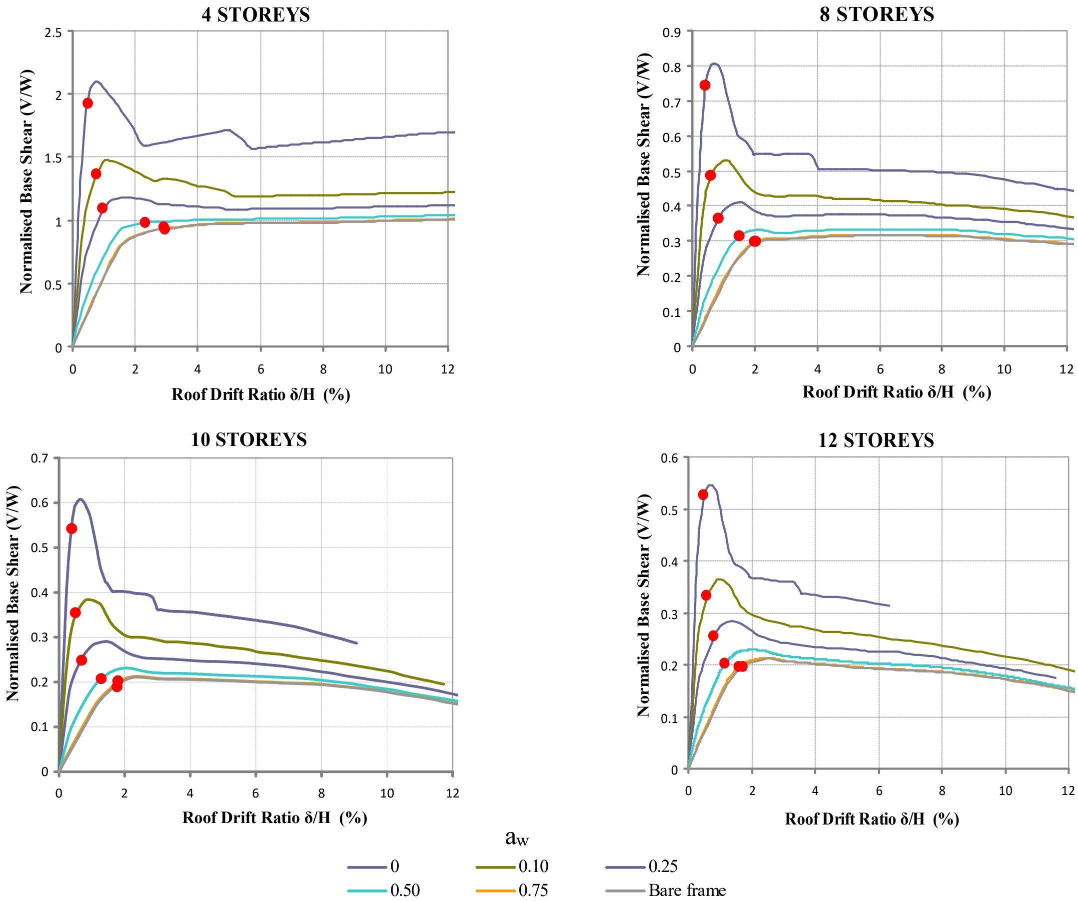


Figure 4: Normalized capacity curves of the examined frames for various ratios of openings a_w.

5 CONCLUSIONS

An investigation of the response of typical moment-resisting steel frames with masonry infills and variable ratio of openings has been performed, using nonlinear static analyses. The results revealed an improved behavior for the frames with infills, however some points of concern have been observed as outlined below.

- Up to yielding, the response of the frames with infills, improves drastically, both in terms of initial stiffness and drifts. In particular, for performance level of 1% roof drift ratio, the increased stiffness offers the possibility to surpass the damage limitation requirements for the seismic design of the structure.
- The post-yield behavior seems also improved, in terms of yield and maximum capacity, but a critical point of the response can emerge, after which strength degradation is observed that is quite severe for infills with small or no openings.
- When the opening ratio exceeds 50% in every frame panel, no significant improvement of the infilled frame response has been detected.

- All infilled frames managed to reach high drift ratios (above 5%), while at the same time exceeding the sustained base shear of the bare frame. However, the improvement of the capacity at these levels of drift ratios is not equally impressive as in a pre-yield state. Specifically, at 5% roof drift ratio, which can be considered appropriate for no collapse requirements, a 1.7x magnification of capacity has been recorded for infills with no openings and 1.15x for infills with 25% openings.

The drastically increased pre-yield stiffness of the infilled frames should raise awareness, for the potential consequences when using a fundamental period under seismic design, based on the bare frame alone. For the frames examined in this work, the fundamental periods of the bare frames were found more than triple of those of the infilled frames with a_w=0. Using the same seismic design parameters, the errors in the calculation of the design seismic base shear, from EN1998 [17] spectrum, using the bare frame period instead of the infilled frame one, have been calculated and are shown in Figure 6. An extreme underestimation of the design base shear, up to -77%, has been found, for the higher storey frames with fewer openings.

Before practical design recommendation can be made, further research is required, towards the impact of irregularities and post-yield masonry-frame interactions, particularly under dynamic loading conditions.

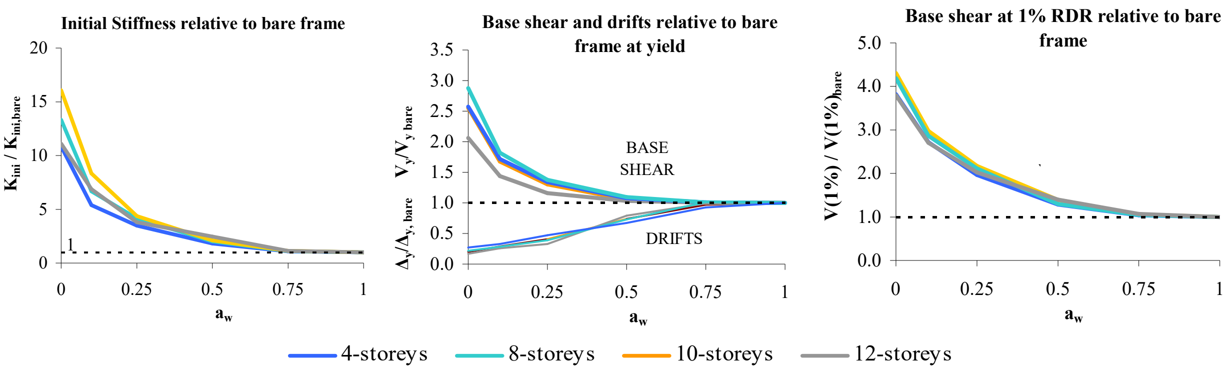


Figure 5: Response characteristics of the examined frames versus the infill openings ratio a_w.

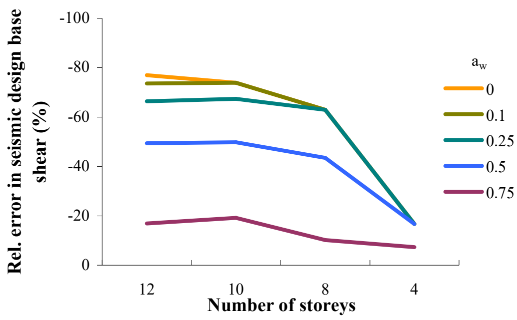


Figure 6: Relative error in the estimation of seismic base shear according to EN1998 when ignoring infills

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