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MECHANICAL-BASED SEISMIC VULNERABILITY ANALYSIS OF INDUSTRIAL STEEL STRUCTURES WITH MASONRY INFILLS

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Abstract. *This work presents a numerical procedure for constructing mechanical-based fragility curves for industrial steel structures. In particular, the structures under considerations are characterised by the absence of anti-seismic details and by the presence of masonry infills. This structural typology has been identified in southern Italy and represents a widespread construction technology. The exposure information are obtained by an in-situ survey of the industrial area in the municipality of Spezzano Albanese (Italy) using the CARTIS-GL form. Fragility curves are constructed using Monte Carlo simulations and the industrial structures are modelled using mixed frame finite elements accounting for geometrical and material non-linearities.*

Keywords: seismic vulnerability, steel structures, industrial structures, masonry infills.

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1 INTRODUCTION

As demonstrated by the recent earthquakes occurred in different areas of the Italian territory, industrial structures represent a structural typology at high seismic risk [1]. In particular, steel structures usually exhibit a better response to seismic action than precast concrete ones [2], as damages produced by Emilia earthquake in 2012 have highlighted [3]. However, if designed without any seismic criteria, even steel structures can undergo severe damages and collapses under seismic actions [4]. For this reason, recent works are focusing on developing strategies for estimating the seismic vulnerability of steel structures in residential [5, 6, 7] and industrial areas [8, 9]. This interest is further remarked by the long-span (GL) CARTIS database, developed by Italian Civil Protection and many Italian universities [10], which collects data on industrial structures in the Italian territory with the final purpose of providing a reliable seismic risk assessment.

Within the framework of urban scale seismic vulnerability estimate, empirical methods, based on observed damages due to earthquakes, are usually adopted [11]. However, the low number of post-earthquake damage data and the high variability of the typological features of industrial steel structures prevent empirical methods to be used and mechanical-based methods [12, 13] should be adopted.

This work focuses on the seismic vulnerability of industrial steel structures using a mechanical-based approach. In particular, we focus on structures designed without any seismic criteria and characterised by the total absence of bracings, but with masonry infills that provide a certain resistance to horizontal actions [14]. In fact, secondary elements, as masonry infills, can play a significant role on the fragility of civil [15] and industrial buildings [16].

This typology has been recognised in many Italian areas, such as that of Spezzano Albanese, in southern Italy. The starting point is the exposure analysis conducted by using CARTIS-GL form [9]. Then, Finite Element (FE) models are constructed, considering both geometrical and material nonlinearities. Sensitivities analysis are conducted to recognise the influence of geometrical and mechanical features on the seismic behaviour. Finally, mechanical-based fragility curves are constructed and the role played by masonry infills is highlighted. Fragility curves are evaluated using a full-blown Monte Carlo method, considering the variability of the structural features observed in the studied area [17].

The work is organised as follows. The description of the industrial area os Spezzano Albanese is given in Section 2. The FE model of nonlinear steel structures is presented in Section 3. Sensitivity analyses and the construction of the mechanical-based fragility curves is shown in Section 4. Finally, conclusions are drawn in Section 5

2 EXPOSURE ANALYSIS

The area under consideration, indicated in Fig. 1, is located in the municipality of Spezzano Albanese, in Calabria region, southern Italy. It is an industrial area built before 1980 and mainly devoted to agricultural purpose which is the principal economic activity of the zone. The total number of industrial buildings is 48, of which 14 made of steel. The structural data for 11 structures is collected using CARTIS-GL form.

The structures are composed by a sequence of parallel frames, realised by steel columns and a truss arch. A typical plan view of the industrial structure is shown in Fig. 2. Figure 3 shows some pictures of the industrial steel structures. It is possible to observe how masonry walls are present along the perimeter. In particular, three configurations are recognised, depending on the walls height, as shown in Figure 4.

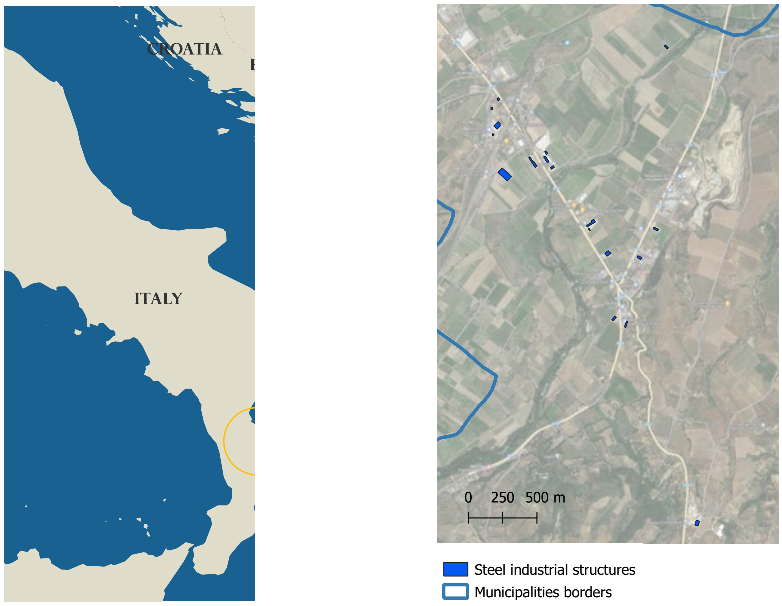


Figure 1: Localisation of the studied area.

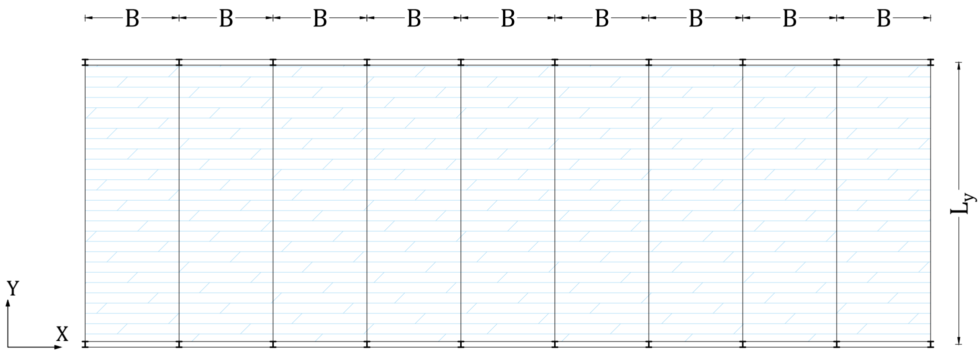


Figure 2: Plan of an industrial steel structure under consideration.

Each surveyed structure is characterised by the group of data listed in Table 1 in which their ranges of variations are also given.

The masonry infills are made of concrete blocks.

3 NUMERICAL MODEL

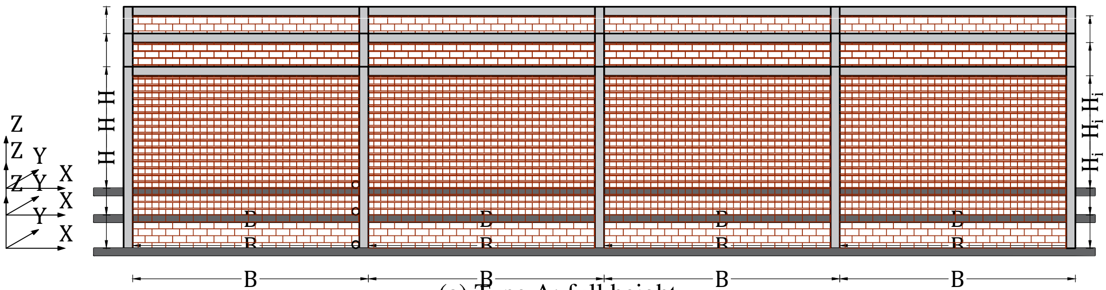
3.1 Finite Element

The structure is modelled using plane beam FE. In particular, we make use of a shear deformable mixed assumed stress FE [18] in which displacements and generalised stresses are considered as primary variables. The assumed stress interpolation satisfies equilibrium equations for zero bulk loads [19]. An elastic-perfectly plastic behaviour is assumed for steel members and the yield surface is represented by an ellipsoid to efficiently consider axial-flexural interaction. The accuracy of the representation of the yield surface can be readily improved by adopting techniques based on the Minkowsky sum of ellipsoids [20]. Plasticity is checked at 3 Gauss-Lobatto points on each FE and the integration of the constitutive equation is performed at the element level in order to preserve the assumed stress interpolation [21, 22].

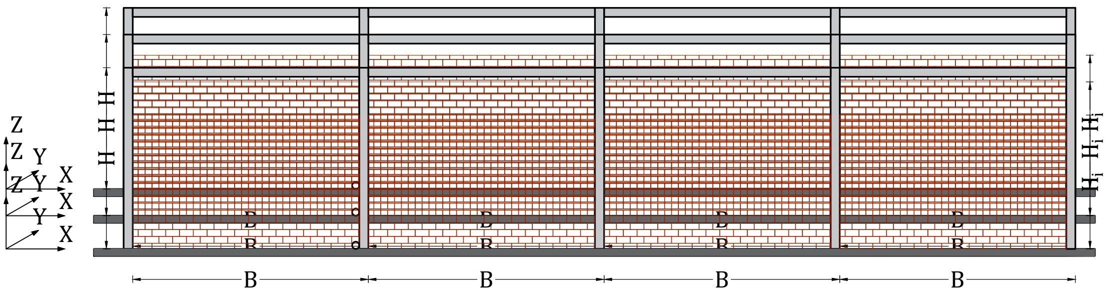
Figure 3: Some pictures of the industrial structures in the area under consideration.

Table 1: Variability of geometrical and mechanical parameters of the recognised industrial buildings

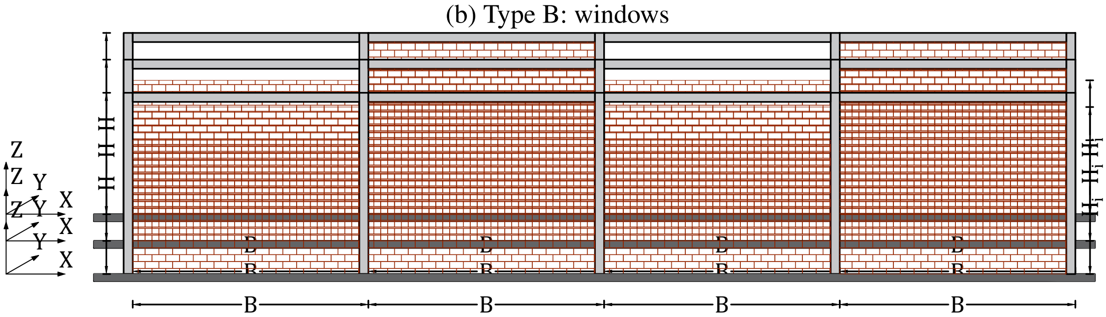
parameter	description	minimum value	maximum value
L_y	Length	11 m	15 m
H	Column height	4.5 m	7.3 m
H_m	Arc height	0.4 m	2.8 m
H_w	Windows height	0.7 m	2.0 m
n_t	Number of frames	4	16
B	Frame distance	5 m	5 m
C_s	Column section	IPE270	HE300A
B_s	Beam section	IPE 140	IPE 240



(a) Type A: full height



(b) Type B: windows



(c) Type C: alternated

Figure 4: Configurations of infilled frames along x direction.

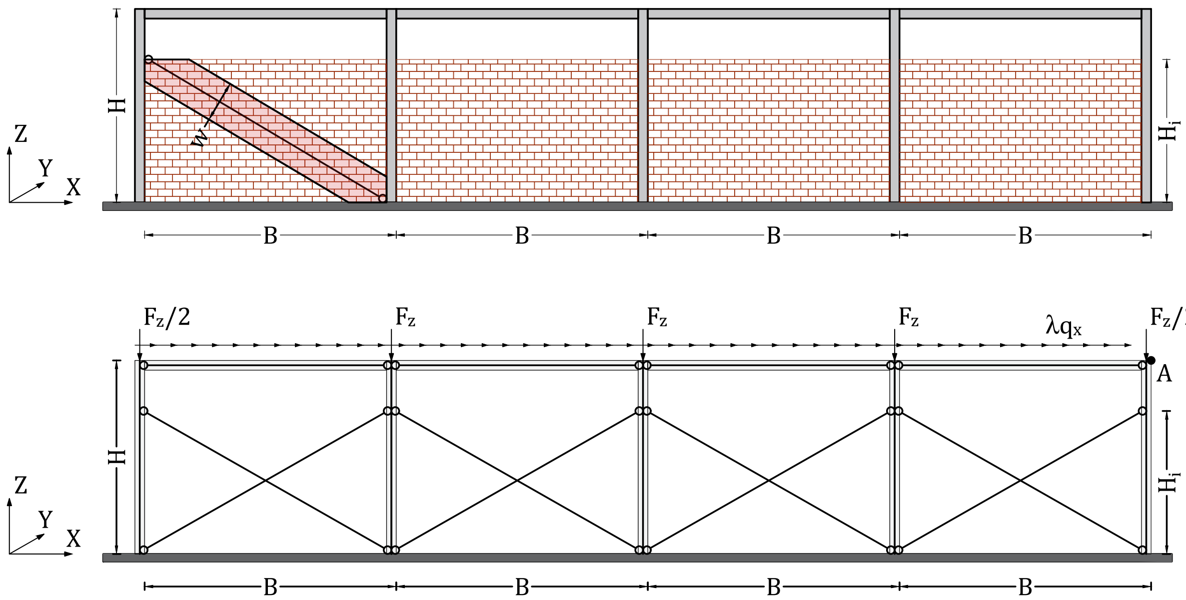


Figure 5: Model of an infilled frame along x direction.

Masonry infills are modelled by adopting a simple single equivalent strut description (see Fig. 5), even if more complex model can be used to improve accuracy [23, 24]. A backbone uniaxial response is adopted [25, 26].

Columns are supposed to be fully clamped on foundations.

The model has been validated by comparing the results with those obtained using the commercial FE software SAP 2000 [27] and Abaqus [28].

3.2 Capacity curve

Nonlinear static analyses are performed on the FE model to simulate the seismic response. An arc-length algorithm is employed to trace the equilibrium path [22].

Figure 6 shows the capacity curves obtained both for the infilled structure and the bare one. It is possible to observe that the presence of the masonry infill increases the initial stiffness and the peak shear force (T). After peak, a strong degradation is observed and the infills contribution vanishes, so that the structural behaviour coincides with that of the bare frame [29].

Three damage levels are considered, namely immediate occupancy (IO), Life Safety (LS) and Near Collapse (NC) [30, 4, 9]. The damage state is defined in terms of inter-storey drift, on the basis of the FEMA [30] legislative provisions, which refers to the following limits: 0.0075 for IO, 0.025 for LS and 0.05 for NC.

3.3 Inelastic spectra

The seismic action is represented by the idealised spectra of Newmark–Hall type defined by Italian seismic code [31] for the municipality of Spezzano Albanese, considering a soil of class C and a topography type T1. In order to obtain the displacement demand, we make use of the inelastic spectra method. In particular, we adopt the modified N2 method [32, 33, 34], specifically devised to be used in presence of masonry infills.

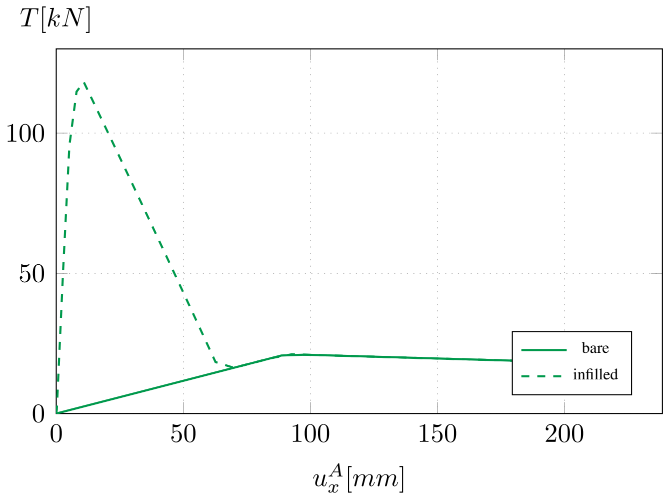


Figure 6: Capacity curves for bare and infilled frames.

Table 2: Variability of additional parameters

parameter	description	minimum value	maximum value
σ_y	Steel yield stress	200 MPa	350 MPa
f_{vt}	Infill shear resistance	0.1 MPa	0.3 MPa
q	Permanent roof load	0.3 kN/m^2	2 kN/m^2

4 FRAGILITY CURVES

In this work we focus on the structural behaviour along x direction only (See Fig. 5). First, a sensitivity analysis is presented. Then, the fragility curves are obtained using Monte Carlo simulations.

4.1 Sensitivity analyses

In this Section, a sensitivity analysis is presented, with the aim of identifying the influence that each structural parameter has on the seismic capacity. We consider the variation ranges obtained by in-situ survey presented in Table 1, plus the variations of other parameters given in Table 2.

A reference structure is considered, characterised by the parameter values given in Table 3, and, on its basis, sensitivity curves are constructed by varying each parameter and obtaining the seismic peak ground acceleration a_g which produces the damage levels. For columns and beams, the section and the related index are given in Tables 4 and 5.

Figure 7 shows the sensitivity curves for the case of bare frames. It is possible to observe that high significance is given by the column type, which influence both the elastic and plastic response. Additionally, the permanent load has a high relevance, especially on the damage levels LS and NC, since it influence the total mass of the dynamic system.

Figures 8, 9 and 10 show the sensitivity curves for the infilled frames for the infill types A, B and C, respectively. In general, one can observe how the a_g values that produce each damage level significantly increase with respect to the bare frame case.

Table 3: Parameters of the reference structure

parameter	L_y	H	H_w	n_t	B	C_s	B_s	σ_y	f_{vt}	q
value	15 m	4 m	1.5 m	6	5 m	HE240A	IPE200	275 MPa	0.2 MPa	1.5 kN/m^2

Table 4: Section types used for columns in the sensitivity analysis.

index	1	2	3	4	5	6	7	8	9	10	11	12	13
section - IPE	270	300											
section - HE			200A	220A	200B	240A	220B	260A	240B	280A	260B	300A	280B

4.2 Fragility curves

Fragility curves are constructed for the structural typology under consideration. As usual [13, 17, 9], fragility curves are expressed by means of cumulative lognormal probability functions, as

$$p(d \geq d_i) = \Phi\left(\frac{log\left(\frac{\lambda_i}{a_g}\right)}{\beta_i}\right), \quad i = 1, \ldots, 3, \tag{1}$$

where d is the structural damage (IO, LS, NC), Φ is a normal cumulative probability function, while λ_i and β_i are the mean value and the logarithmic standard deviation of the a_g values that cause the d_i damage grade, respectively. For each damage grade d_i, the definition of a fragility curve requires the evaluation of the two parameters λ_i and β_i. To this end, it is necessary to obtain a population of buildings reflecting the in-situ variation of the significant features of the masonry buildings under consideration and to model the uncertainties involved in the seismic vulnerability.

This population is built using Monte Carlo simulation, based on a random generation considering the variability ranges in Tables 1 and 2. For each parameter, a uniform distribution is assumed.

Fragility curves are given in Fig 11 for the bare and type A infilled frames. It is possible to observe how the bare frame shows significantly higher vulnerability than the infilled structure. This aspects highlights the fact that for this type of structure modelling the infill is of crucial importance and suggests that it does not play a secondary structural role, but represents the main seismic-resistant component.

Table 5: Section types used for beams in the sensitivity analysis.

index	1	2	3	4	5	6	7	8	9	10	11
section - IPE	140	160 A	160	180 A	180	200 A	200	220 A	220	240 A	240

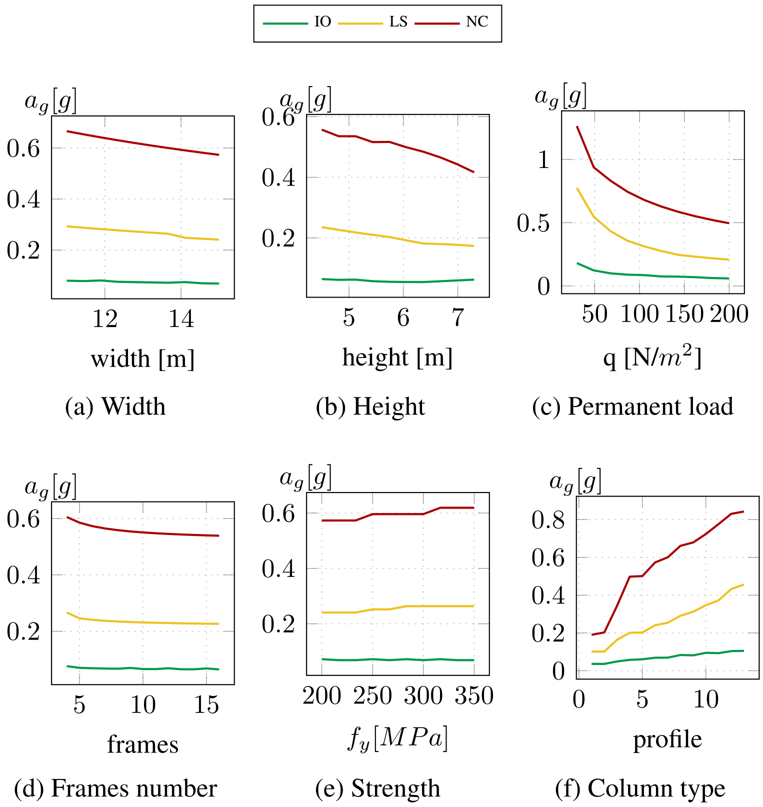


Figure 7: Sensitivity analysis for some geometrical and mechanical attributes, bare frame.

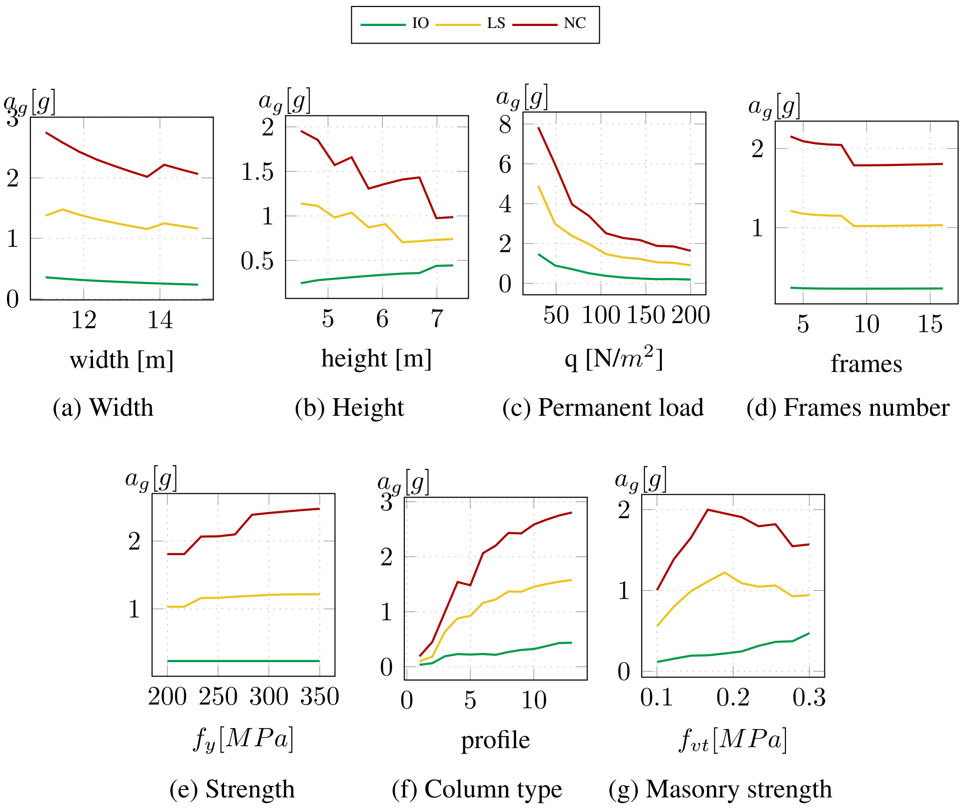
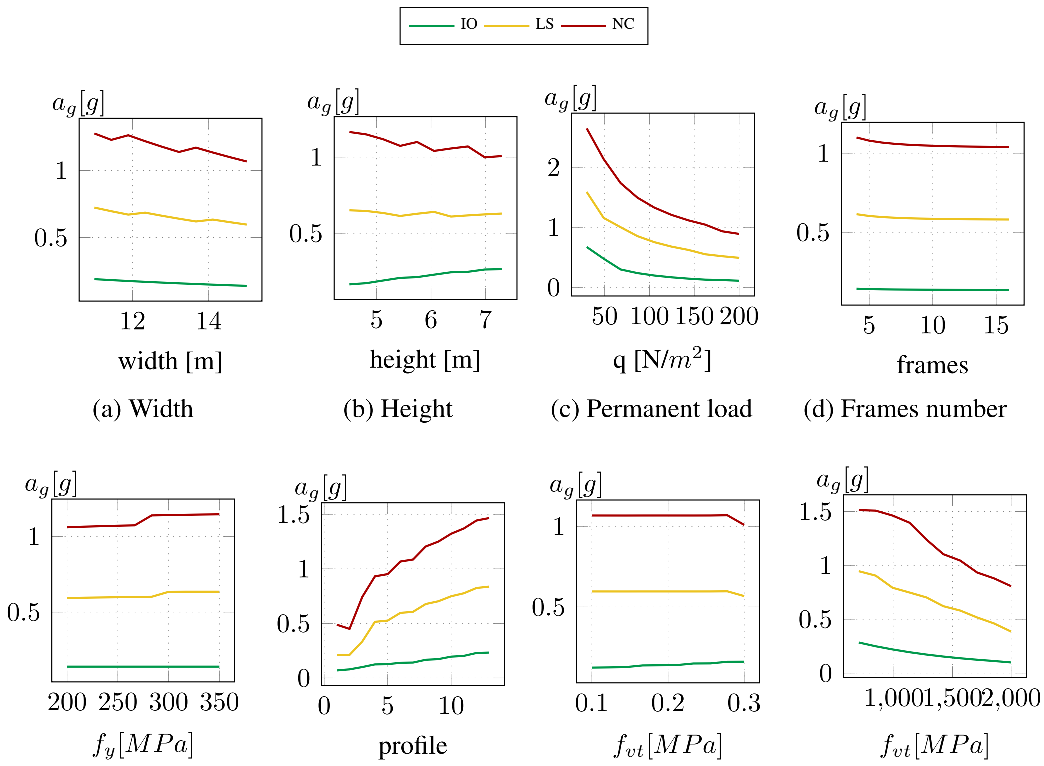


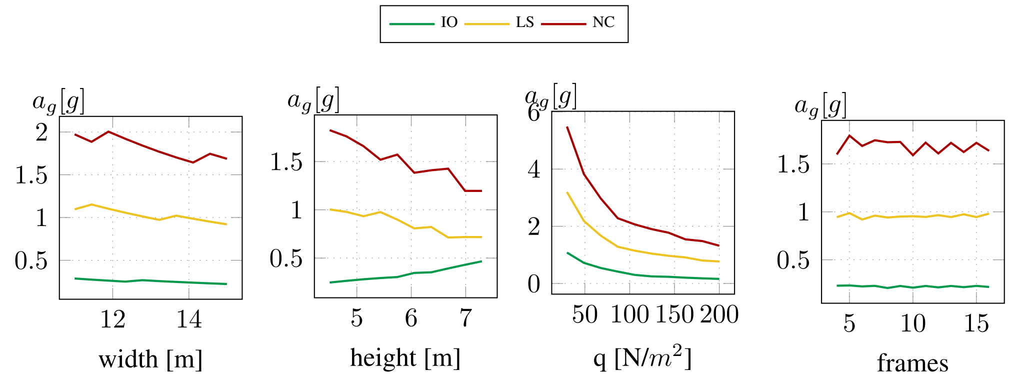
Figure 8: Sensitivity analysis for some geometrical and mechanical attributes, type A infills, X direction.



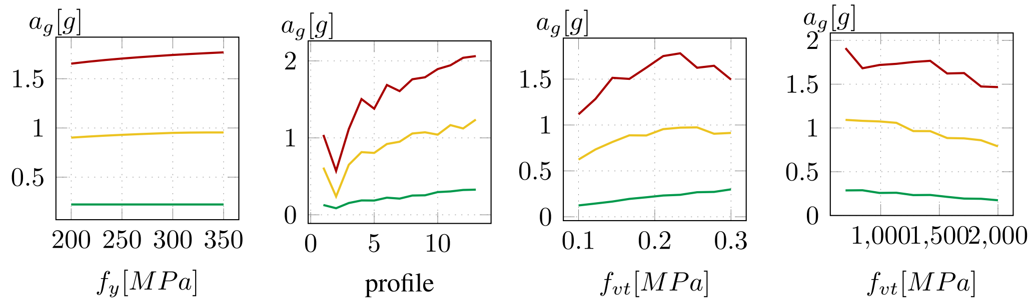
(a) Width (b) Height (c) Permanent load (d) Frames number

(e) Strength (f) Column type (g) Masonry strength (h) Windows height

Figure 9: Sensitivity analysis for some geometrical and mechanical attributes, type B infills, X direction.

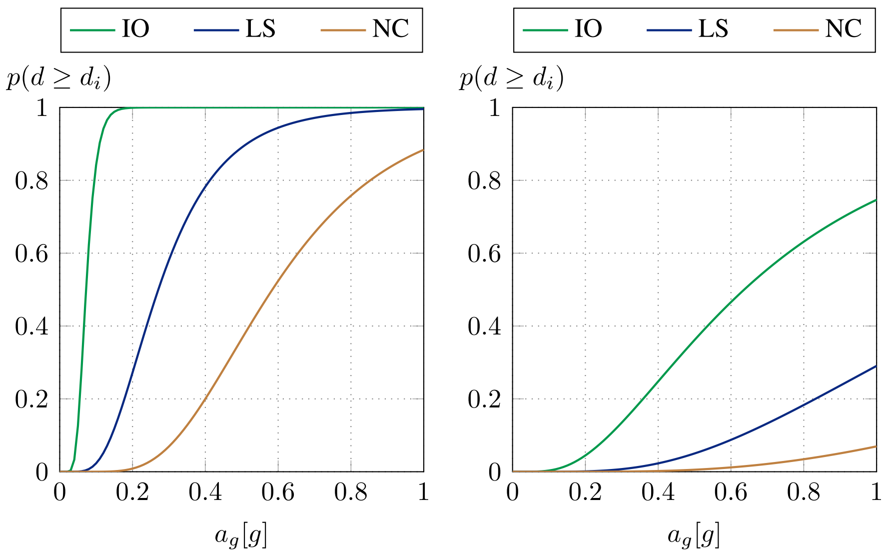


(a) Width (b) Height (c) Permanent load (d) Frames number



(e) Strength (f) Column type (g) Masonry strength (h) Windows height

Figure 10: Sensitivity analysis for some geometrical and mechanical attributes, type C infills, X direction.



(a) Bare frame (b) Infilled type A

Figure 11: Fragility curves.

5 CONCLUSIONS

In this work, the problem of the seismic vulnerability of steel industrial structures designed without anti-seismic criteria and with masonry infills has been addressed. An exposure analysis of an industrial area in the municipality of Spezzano Albanese (Italy) has been conducted using CARTIS-GL form. A numerical model, based on beam FE having geometrical and mechanical nonlinearities has been constructed to analyse the industrial structures. On the basis of the surveyed data, sensitivity analysis has been conducted to identify the most relevant parameters. Results have shown that the mass of the construction has a remarkable influence on the seismic vulnerability, thereby suggesting to reduce the weight of the roof to alleviate the seismic risk. By comparing fragility curves for bare and infilled frames, it has been shown that the presence of masonry infills has a positive influence on the seismic vulnerability.

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