Seismic Vulnerability and Fragility of Industrial Steel Buildings Affected by the Emilia-Romagna Earthquake

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Abstract. The recent L'Aquila (2009) and Emilia-Romagna (2012) Italian earthquakes have highlighted the vulnerability of recently erected buildings, with particular reference to industrial ones. Although steel buildings have usually demonstrated a good behaviour under earthquakes, with limited damages and rare cases of collapse, they still represent a structural typology at risk due to the significant exposure connected to the importance of the resources they host. In the paper a parametric study on several industrial steel buildings, different for typology, geometrical dimensions, seismic zone and snow geographic area, has been done through pushover analyses. The results have allowed to plot vulnerability curves, which have been compared to seismic fragility curves derived from literature studies. The comparison among curves have allowed to estimate the effectiveness of the theoretical relationships, as well as to evaluate the seismic damages suffered by investigated structures under dissimilar grade earthquakes.

1. INTRODUCTION

The most of the Italian built heritage consists of buildings constructed without appropriate anti-seismic design rules, as they were erected in those regions that, before of the new technical legislative measures, were not considered as seismic zones. The recent L'Aquila (2009) and Emilia-Romagna (2012) Italian earthquakes have highlighted the vulnerability of the Italian built heritage, with reference not only to the historical buildings, but also to the recently erected industrial constructions. In this direction, an example is the town of Mirandola (district of Modena), where 80% of industrial buildings, mostly made of pre-stressed reinforced concrete, was destroyed or considered to be unfit for use after seismic events occurred on 2009 May 20th and 29th.

The constructive peculiarity of these buildings, that are much widespread all over the Italian country, is the easy erection process based on hinged beam-to-column joints. Contrary, this building typology is particularly sensible to horizontal actions, especially when additional structural systems, such as cranes and pallet racks, are placed inside them. In this paper the attention is dedicated to the industrial steel buildings that, despite they suffered limited damages compared to those of pre-stressed r.c. structures, were designed without suitable seismic rules introduced in Italy only since 2003. Generally, the industrial steel frame buildings assure high levels of reliability in case of earthquake, considering that many of them, although they were not designed to resist seismic actions, remained either unharmed or suffered limited damage. The only cases of collapse are mostly conditioned by the failure of pallet racks that, often reaching considerable heights, represent real structures inside the industrial building and, therefore, they need an appropriate seismic design. Therefore, in the case of industrial buildings, the issue of the life safety is associated to the theme of the safeguard of the values exposed at risk and, above all, to the continuity of business activities after the earthquake. In the paper the seismic behaviour of some industrial steel buildings has been assessed through non-linear static analyses which allowed to plot, starting from pushover curves, their vulnerability curves, used to know exhaustively the expected seismic damages suffered under earthquakes of different intensity with reference to different limit states. Considering the difficulty to investigate all varieties of existing industrial buildings, only some of the most common types detected in Italy, representative of the industrial steel buildings heritage, have been examined. In particular, identification and analysis of a number of typical buildings, different each other for geometric dimensions and constraint conditions, have been done, as it will be shown in the next Sections.

2. SELECTION OF STRUCTURAL TYPOLOGIES

Generally, one-story buildings for industrial use are characterised by regular plan layouts having large spans with minimum encumbrance of structural elements. Usually, the longitudinal distance among columns ranges from 5 to 15 m, while the transversal one varies from 15 to 30m. The inner height between the work plane (about 1 m far from the floor) and the lowest point of the roof intrados is often contained between 5 and 15m. Obviously, these dimensional values are only for guidance and they are usually variable depending on the types and structural elements employed. In order to be able to assess more accurately the variability fields of these buildings, a significant number of projects and real case studies, from which the most recurrent in plan and in elevation average sizes for each type are derived, have been collected. Subsequently, the selected types have been divided into classes depending on both the type and the slope of roof beams. Finally, for each of the case studies selected, lattice girders or full web beams have been considered. The structural schemes adopted have been designed on the basis of the regulations at the time of their realization through a simulated design process. After defining the individual sub-models (*geometrical, mechanical and loading*), which together contribute to define the structural numerical model, the simulated design has been carried out considering the variability of various parameters associated to the *constraint conditions*, the *dimensional aspects* and the *geographical area* where structures are located.

The geometrical model has been defined considering the variability of the most common structural schemes symbolising the industrial steel buildings. After identifying the more representative model of each investigated structural type, having given average dimensions, different schemes of the same structural system, but with different sizes(columns (h) and roof beams (h') depths) and constraint conditions (hinge(H) or encastre(E)), have been numerically examined. All schemes subjected to the seismic vulnerability assessment are shown in Figure 1, where the investigated frames are identified with acronyms according to both the type of structural elements used for roofing systems (plane lattice beams *(PLB),* plane beams *(PB),*double slope lattice beams *(DSLB)* and double slope beams *(DSB))*and the constraint conditions (*H -E).*

Then, such schemes have been respectively identified by the letters A, A', B and B' and, for each of them, two different constraint conditions have been contemplated, leading to the definition of the following eight structural systems: A_H, A_E, A'_H, A'_E, B_H, B_E, B'_H and B'_E. For the first two structural patterns (A_H and A_E), it has been also hypothesised a variability of the dimensions h and h' for the execution of a more wide parametric analysis. In such a case, starting from a reference case study having assigned average dimensions (l =20m, h=9m and h'= 2m), it has been expanded the field of investigation considering possible variations of the geometric parameters h and h', which gave rise to thirty cases of analysis (Figure 1). Therefore, considering also other six cases where any parametric analysis has been performed, a total of 36 structural models have been analysed.

The *mechanical model* has been implemented by defining the nature of the materials used. In the case in question it has been used a S275 steel type with characteristics intermediate between the mild steels commonly used in the constructive practice. Finally, with regard to the loads acting on the structures and, therefore, to the *loading model*, gravitational actions linked to the masses of structural and non-structural elements have been defined and the characteristic values of the variable actions due to snow, wind and earthquake have been calculated. Actually, all the parameters to be analysed for an extensive parametric analysis would be significantly numerous, but this paper refers to a particular selection of some typological, dimensional and geographical variables only. Geographic variability is linked to the location of the structural system and it influences the entity of seismic and variable (wind and snow) loads considered. With reference to these former loads, by taking into account the three climatic zones representative of the Northern, Central and Southern regions of the Italian country, the number of analyses have been increased from 36 to108.

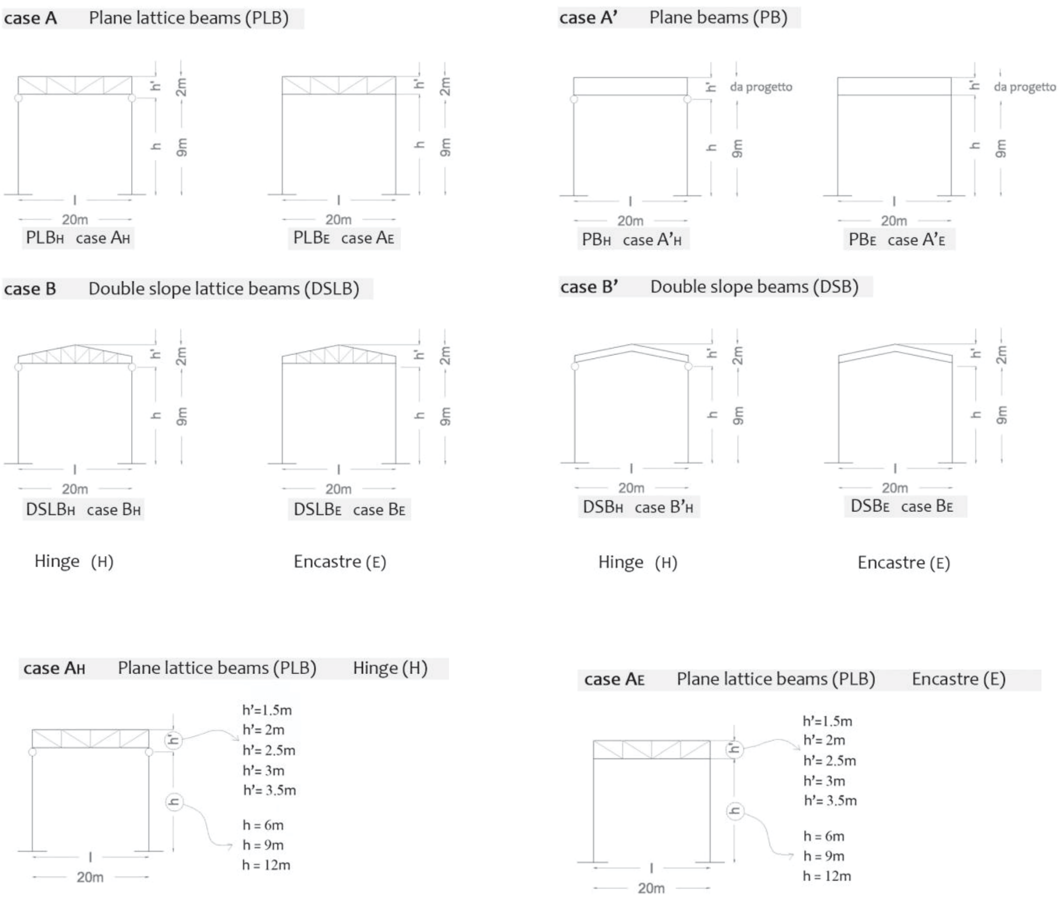


Fig 1: Geometric schemes and variations of dimensional and constraint conditions of structural systems analysed

After defining the geometry and global dimensions of the various structural systems to be investigated, considering all possible loads applied, the design of individual profiles to be employed have been done. The structures of the lattice girders have been obtained by coupling two profiles with *C* or *L* cross-sections: the former (UPN) has been assumed for the upper (U.B.) and lower (L.B.) beams, while the latter has been chosen for the diagonal members (D.M.). For the structural type with plane beam (P.B.) HEB700/800/900profiles, with a variability essentially conditioned by the geographical location assumed for each structural scheme, have been used.

As a first step, the values of the more heavy stresses related to the different load combinations for each structural element have been collected, so to identify the most suitable profiles to be adopted for each individual component of the analysed systems. This design phase has been performed through a simulated design according to the prevailing regulations at the construction time of these structures, essentially based on the Allowable Stress method (Table 1). The structure check has been done by means of the actual code, allowing to confirm or not the dimensions originally assigned to frame members (Table 2). This check phase has been performed through the finite element analysis program SAP 2000.

From the comparison between the frames dimensioned according to the two different code approaches, a minimum difference of weight, with an average percentage difference equal to 1.26%, emerges. This means that the seismic actions do not affect the design of these structures, whose design is essentially dictated by the wind loads only. As a consequence, high levels of structural reliability of these structural types also according to the new seismic code are assured.

Table 1: Weights of AH structures designed according to the old Italian technical code (CNR 10011)

	Frames	L.B.	U.B.	D.M.	Columns	Weights[KN]
AH_1	$AH_{13}z1$ (l=20, h=6, h'=2.5)	UPN 140	UPN 200	90x10	HEB 220	80.1
	$AH_{13}z2$ (l=20, h=6, h'=2.5)	UPN 140	UPN 220	90x12	HEB 260	85.3
	$AH_{13}z3$(l=20, h=6, h'=2.5)	UPN 100	UPN 180	90x7	HEB 280	78.5
AH_2	$AH_{23}z1$ (l=20, h=9, h'=2.5)	UPN 140	UPN 200	90x10	HEB 280	91.4
	$AH_{23}z2$ (l=20, h=9, h'=2.5)	UPN 140	UPN 220	90x12	HEB 300	98.4
	$AH_{23}z3$(l=20, h=9, h'=2.5)	UPN 100	UPN 180	90x7	HEB 360	93.3
AH_3	$AH_{33}z1$(l=20, h=12, h'=2.5)	UPN 140	UPN 200	90x10	HEB 320	104.1
	$AH_{33}z2$(l=20, h=12, h'=2.5)	UPN 140	UPN 220	90x12	HEB360	112.5
	$AH_{33}z3$(l=20, h=12, h'=2.5)	UPN 100	UPN 180	90x7	HEM320	130.9
		Total				874.5

Table 2: Weights of AH structures designed according to the actual Italian technical code (M.D. 08)

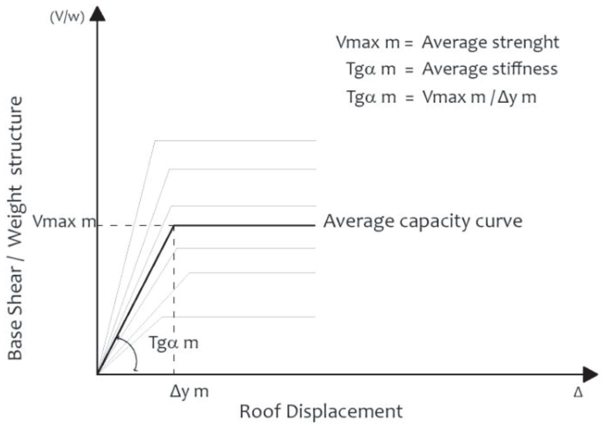
	Frames	L.B.	U.B.	D.M.	Columns	Weights[KN]
AH_1	$AH_{13}z1$ (l=20, h=6, h'=2.5)	UPN 140	UPN 180	90x9	HEB 220	77.6
	$AH_{13}z2$ (l=20, h=6, h'=2.5)	UPN 160	UPN 200	90x10	HEB 260	84.7
	$AH_{13}z3$(l=20, h=6, h'=2.5)	UPN 100	UPN 140	80x10	HEB 280	78.1
AH_2	$AH_{23}z1$ (l=20, h=9, h'=2.5)	UPN 140	UPN 180	90x9	HEB 280	89.9
	$AH_{23}z2$ (l=20, h=9, h'=2.5)	UPN 160	UPN 200	90x10	HEB 300	95.7
	$AH_{23}z3$(l=20, h=9, h'=2.5)	UPN 100	UPN 140	80x10	HEB 400	96.1
AH_3	$AH_{33}z1$(l=20, h=12, h'=2.5)	UPN 140	UPN 180	90x9	HEB 400	109.9
	$AH_{33}z2$(l=20, h=12, h'=2.5)	UPN 160	UPN 200	90x10	HEM 280	122.9
	$AH_{33}z3$(l=20, h=12, h'=2.5)	UPN 100	UPN 140	80x10	HEM320	130.6
		Total				889.5

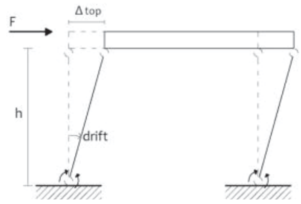
3. NON-LINEAR ANALYSIS AND CAPACITY CURVES

The seismic response of the structures under study has been evaluated by non-linear static analyses carried out by using the calculation program SAP2000. A lumped plasticity modelling for structural elements has been adopted by identifying areas of plasticity and defining the behaviour of the plastic hinges in terms of generalized force-displacement curves. The non linear behaviour of beams has been taken of elastic-plastic type with an ultimate limit of rotation $\theta_u = \theta_y$. For columns, instead, it has been defined a domain of resistance considering the simultaneous application of compressive and bending stress. The analysis has been conducted under displacement control, assuming as control point the geometric centre of gravity of the roofing.

Given the considerable number of frames analysed, their subdivision into classes has been done and a capacity curve representative for each of them has been plotted. The cases presented have been divided according to the types and the constraint conditions, also considering further variables, such as the geometric dimensions and the geographical location. For the early two cases A_H and A_E, indicative of the type with plane lattice beams (PLB), a parametric analysis has been conducted considering the height of columns (6m, 9m, 12m) and the roofing systems one (1.5m, 2.0m, 2.5m, 3.0m, 3.5m) as variable parameters. Therefore, a further subdivision of each of the above-mentioned schemes in the three subcases (A_{H1}, A_{H2}, A_{H3} and A_{E1}, A_{E2}, A_{E3})has been made(see Tables 1 and 2). For all the other cases (*A'*, *B*and*B'*), where other additional variabilities, other than the different constraint conditions and the different geographical location, have not been considered, three representative capacity curves, one for each structural type, have been derived. Therefore, in accordance with the preceding subdivision, nine capacity curves representative of the different frames analysed, have been obtained and compared each to other, so to grasp typical behaviour and peculiarities of the different classes of structures examined.

The analysis results have been represented in terms of base shear, normalised to the total weight of the structural system, versus the displacement of a control point coincident with the gravity centre of the roofing. The representative capacity curve, characterised by a bilinear shape (typical of a SDOF system), corresponds to the average capacity curve of the family of curves obtained for each class of frames (Figure 2).





Limitations FEMA			
drift [rad]	0.0075	0.025	0.05

	IO	LS	NC
Hcolumn	Δ top,IO	Δ top,LS	Δ top,NC
[m]	[mm]	[mm]	[mm]
6	45	150	300
9	67.5	225	450
12	90	300	600

Fig 2: Average capacity curve of a generic set of structural systems examined (a), drift and displacement restrictions derived from the FEMA 356 code for industrial steel buildings examined (b)

Another aspect to be considered for defining the capacity curves is the assessment of the ultimate displacement Δ_u related to the structure collapse mechanism. This value is taken on the basis of the FEMA 356 regulations [4], which are more complete and detailed than the Eurocode 8 provisions. This choice has been justified, besides, by the chance to compare the results with those proposed by other USA scientific researches dealing with similar topics[5]. The structure capacity in terms of displacement has been carried out on the basis of the US legislation provisions, which refer to predefined maximum values of the *inter-story drift*: 0.0075 for immediate occupancy (IO), 0.025 for life safety (LS) and, finally, 0.05 for near collapse (NC). Therefore, considering the variability of the column height (6, 9 and 12m), the ultimate displacement assumes the values of 300, 450 and 600mm, respectively, as shown in Figure 2.

4. FRAGILITY AND VULNERABILITY CURVES

In case of earthquake, each structural system is exposed to a risk correlated to both losses and the degree of damage that it can exhibit. In the seismic risk analysis, in fact, it is necessary to translate the knowledge of the built vulnerability in the damage that can occur as a result of earthquakes having different magnitudes. The seismic risk parameter of a system *R* can be expressed as a function of its vulnerability *V*, of the parameter *s*, related to the severity of the earthquake, and of the parameter *d,* intended as measure of the damage, by means of a correlation law *R = R (V, d, s).*

One of the tools for the determination of the structure seismic risk and, therefore, of the above-mentioned functional link, is represented by the fragility curves. They provide the probability of a structural system exposed to a seismic input assigned to overcome certain levels of damage. In this paper two procedures have been used for deriving fragility curves of investigated structures. The first is a *discrete / manual* procedure, punctually derived from the curves of capacity, which gives rise to those that will be defined as vulnerability curves, whereas the second procedure is an *analytical* method based on some literature indications. For the seismic reliability assessment of the structures, the current research trends are directed towards rigorous probabilistic approaches, involving both random and deterministic variables that are often difficult to be considered at all in the project. For this reason, it has been herein proposed and developed a procedure simpler than that based on analytical formulations. The procedure presented allows to validate, through appropriate comparisons, the effectiveness of the fragility curves to estimate the seismic damage of structures subjected to earthquakes of different intensity.

The *discrete / manual procedure* evaluates the structural capacity and compares it directly with the demand related to the particular seismic event on the basis of a limit state considered. In this paper, based on the FEMA 356 guidelines, three performance levels (IO , LS, NC), characterised by appropriate values of inter - story drifts, have been taken into account. Starting from the structural behaviour in the non-linear static field, some damage levels of the structure, corresponding to the limits above defined by FEMA 356 provisions, have been defined and, for each earthquake with a given hazard level, the expected building damage can be estimated by simply correlating the capacity displacement (or inter-story drift) with the demand one. The ratio between the demand parameter and the capacity one is then correlated to the damage levels of the EMS 98 scale normalised in the range [0-1]. By varying the earthquake intensity and, therefore, the seismic demand, for each of the three limit states considered, the above ratio is calculated, allowing to plot step-by-step the structure fragility in a simple way, which is herein called vulnerability curve (Figure 3).

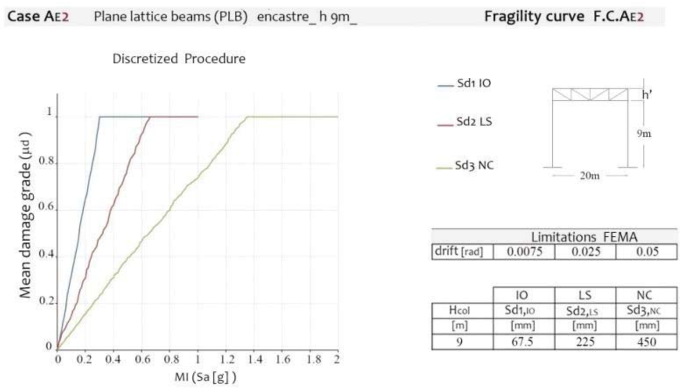


Fig 3: Vulnerability curves of the structural system AE2

Wanting to evaluate the propensity at damage of the examined industrial steel buildings considering the random nature of the earthquake, a peak ground acceleration of the demand spectrum variable between 0.01g and 1ghas been considered. It is interesting to note that, as for this procedure, the first examples of fragility or *vulnerability* curves were referred to a conventional scheme that simplified the assessment procedure of the seismic vulnerability [6].

So, the general procedure involving a number of points of the curve can be replaced by a simpler method, which is based, in absence of additional information, on two parameters only: the collapse point (acceleration y_c) and the damage starting point (acceleration y_i) of the building (Figure 4). Thus, the curve was obviously undetermined, but it could be adequately represented by a linear conventional trend.

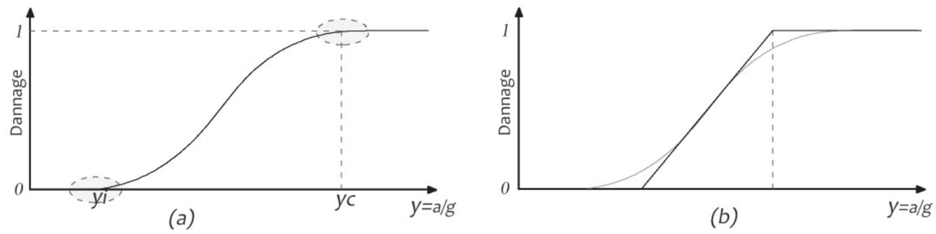


Fig 4: Qualitative trend of the damage-acceleration function

The *analytical* procedure defines, according to the variation of seismic intensity, the structure probability of reaching or exceeding a particular limit state. In mathematical terms, this is expressed by the function of conditional probability P[SL|I], where SL|I is a symbol indicative of achieving or exceeding the assigned limit state when the seismic intensity value (*I*), which can be represented under form of PGA, spectral acceleration, etc, is fixed.

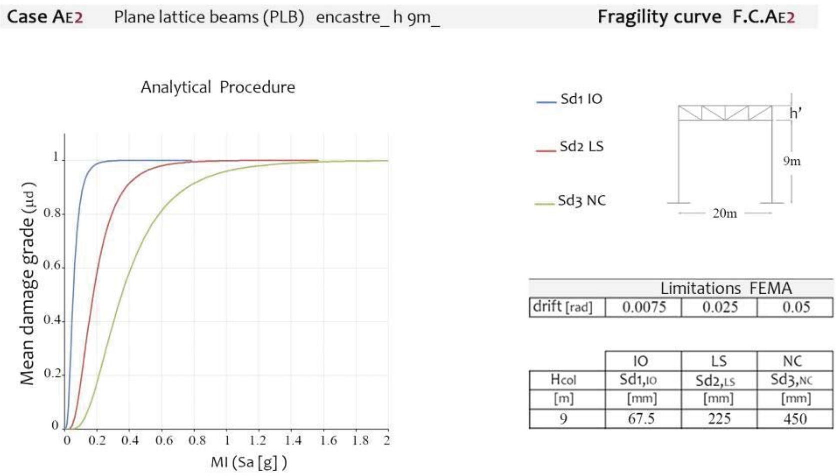
The ways to define the damage thresholds are numerous: one of these is defined as a function of the two points representative of the push-over curve, that is the yielding displacement D_y and the ultimate one D_u. The approach proposed in this paper, instead, correlates the limit states at appropriate drift values, in line with the provisions of other scientific researches on the subject [5]. As a seismic parameter (measure of the intensity *I*) the spectral displacement S_d has been adopted, because the capacity curves have been converted into the ADRS format in order to be able to compare in an easy manner the capacity values with those of the seismic demand represented by the response spectrum. As a result, the fragility curves are obtained mathematically using the following equation:

$$P[SL/I] = \Phi\left[\frac{1}{\beta}\cdot\ln\left(\frac{I}{I_{SL}}\right)\right] \qquad (1)$$

The equation (1) defines the probability of occurrence or exceeding the state limit considered by means of a log-normal cumulative distribution, where:

- Φ is the standard normal distribution function;
- I is the measurement unit of the intensity (or intensity measure);
- I_{SL} is the median of the intensity measure for which the building reaches a given limit state;
- β is the standard deviation of the intensity natural logarithm for the limit state considered, assumed equal to 0.6 according to the indications reported in.

According to this method, each fragility curve is characterized by two parameters: the first is the average value of the intensity measure responsible of reaching the limit state threshold and the second parameter is the relative standard deviation. For each structural system it is possible to trace more fragility curves, each of them associated to a predetermined limit state. An example of fragility curves constructed according to the previous analytical procedure is reported in Figure 5, where for the same structural system (case A_{E2}) three curves obtained for three different limit states (IO, LS and NC) are simultaneously reported. Later on, the seismic safety of structures examined, placed for the sake of example in Mirandola, one of the sites most affected by the Emilia-Romagna earthquake, has been assessed and, finally, the reliability of the fragility curves deriving from literature analytical formulations has been proved.



drift [rad]	0.0075	0.025	0.05

	IO	LS	NC
Hcol	Sd1,IO	Sd2,LS	Sd3,NC
[m]	[mm]	[mm]	[mm]
9	67.5	225	450

Fig 5: Fragility curves for the structural system AE2.

As regards to the first aspect, it is noticed that, at the life safety limit state, the structures, although designed for vertical loads, meet the safety requirements of the current regulations for seismic-resistant structures. In fact, they reach the collapse under an acceleration value greater than that of the seismic zone with the greatest intensity, that is characterized by a peak ground acceleration of 0.35g.

On the other hand, with reference to the second key question, the discrete fragility curves are manually defined according to a procedure much more laborious than the analytical one, the latter requiring the knowledge of a smaller number of factors to more practically assess the safety of structures. The comparisons highlight that the analytical curves show values of expected damage greater than the values obtained by discrete curves. Therefore, the literature fragility curves represent a conservative prediction method of the structural safety of steel industrial buildings.

5. CONCLUSIONS

The non-linear analyses carried out in order to determine the capacity and the fragility curves of a set of steel one-storey building typologies, that are very spread on the Italian country, have led towards the following conclusions:

1. The seismic action has a little influence on design of industrial steel structures: they are more influenced by the action of wind loads rather than those of the earthquake.
2. The structures, although are designed only for gravity loads, have demonstrated a good behaviour under earthquake. In fact, they reach the collapse for an acceleration value higher than 0.35g, that is the maximum value of the PGA for the highest Italian seismic hazard area.
3. The analytical fragility curves overestimate the damage predicted by the discrete vulnerability curves. Thus, they are a method on the safe side in forecasting the steel industrial building collapse under seismic actions.
4. The performed analyses, although they have provided interesting considerations about the seismic hazard of the steel one-storey buildings for industrial use investigated, represent only the first step towards the characterization of all types of this structural typology. As a consequence, additional analyses could be carried out by taking into account the variability of both the gravity loads and the steel grades used, as well as of the geometric parameters defining the structural schemes of the inspected typologies.

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