Multi-hazard risk assessment of a steel hangar subjected to seismic and wind threats

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ABSTRACT
A structure may be subjected to more than one critical action during its lifetime. The probabilistic multi-hazard approach can be employed in order to investigate the performance of a structure under critical events and to ensure its acceptable performance during its entire lifetime. This paper focuses on the estimation of the annual frequency associated with exceeding the limit state of collapse for a steel hangar subjected to seismic and wind threats. The seismic fragility is calculated by implementing an incremental dynamic analysis (IDA) using the method of multiple-stripe analysis. The main objective is to provide a tool for assessment and retrofit of existing steel strategic structures such as hangars subjected to both wind and earthquake hazard. As a case study, the wind and seismic fragilities of a generic steel hangar located in seismic zone are calculated and implemented in the framework of a multi-hazard procedure, leading to the evaluation of the annual risk of collapse.

Keywords: multi-hazard assessment, steel hangar, incremental dynamic analysis, seismic fragility, wind hazard.

1. INTRODUCTION

In order to investigate structural performance of structures, all possible critical events that could occur during their life-time should be considered. The assessment of structural performance in a multi-hazard framework requires a probabilistic evaluation of loads. In particular, for the limit state of collapse, the probability of collapse P(C) can be written as:

$$P(C) = \sum_A P(C|A) \cdot P(A) \tag{1.1}$$

where A stands for a critical event, such as, earthquake, wind, fire, blast, etc, $P(A)$ is the probability of occurrence of event A and $P(C|A)$ is the probability of collapse for a certain event A (Ellingwood 2006). Equation 1.1 is written, according to total probability theorem assuming that the critical events *A* are mutually exclusive (i.e., they cannot happen simultaneously) and collectively exhaustive (i.e., all of the potential A are considered). Obviously, in Equation (1.1) some of the terms can be neglected if the rate of occurrence referred to the corresponding events is practically negligible. The de minimis risk v_{dm}, which defines that risk below which society normally does not impose any regulatory guidance, is in the order of 10^{-7}/year (Paté-Cornell 1994). Therefore, if the annual risk of occurrence of any critical event A is considerably less than the de minimis level, it could be omitted from the critical events considered. Hence, the multi-hazard acceptance criteria can be written as following:

$$\mathrm{P(C)} = \sum_{\mathrm{A}} \mathrm{P(C|A)} \cdot \mathrm{P(A)} \leq \mathrm{v}_{\mathrm{dm}} \tag{1.2}$$

The above-mentioned criteria could be used both for probability based design and assessment of structures for limit state of collapse. The methodology hereafter presented is implemented for an existing steel hangar located in Ciampino Airport belonging to the Italian Air Force.

2. A BI-HAZARD APPROACH CONSIDERING EARTHQUAKE AND WIND

This work examines the case of steel hangars for airport facilities located in seismic zones; the annual frequency of collapse for these structures can be calculated by summing the contributions from wind and earthquake actions, which represent the most significant hazard. These actions are assumed mutually exclusive and collectively exhaustive. In this bi-hazard approach expression (1.1) becomes:

$$\mathrm{P_f} = \sum_{\mathrm{E}} \mathrm{P(C|E)P(E)} + \sum_{\mathrm{W}} \mathrm{P(C|W)P(W)} \tag{2.1}$$

where P_f stands for the annual rate of collapse, $P(E)$ and $P(W)$ stand for the annual rates of occurrence of earthquake and wind events, respectively. $P(C|E)$ and $P(C|W)$ represent seismic and wind fragilities. The summations used in Equation (2.1) refer to the disaggregation of both earthquake and wind hazard into different class of events.

2.1 SEISMIC CONTRIBUTION TO PROBABILITY OF COLLAPSE

The maximum displacement at the top point of the columns can be used as the structural response parameter for the hangar. Thus, the limit state of collapse is defined as the onset of the maximum top displacement reaching a specific threshold. The seismic fragility, defined as the probability of structural collapse for a given spectral acceleration level, can be calculated by following a non-linear dynamic analysis approach. In particular, an incremental dynamic analysis (IDA) can be performed using the method of multiple-stripe analysis (Jalayer 2003). This can provide statistical information about the displacement demand over a wide range of spectral acceleration values.

The structural fragility, for the limit state of collapse, is defined as the conditional probability of exceeding the limit state capacity for a given level of ground motion intensity (conditional probability of failure). If the ground motion intensity is represented in terms of the spectral acceleration, the fragility at a specific spectral acceleration s_a can be expressed as:

$$\mathrm{P(C|E)} = \mathrm{P}\left(\mathrm{S_{a,D}} > \mathrm{S_{a,C}} \middle| \mathrm{S_{a,D}} = \mathrm{s_a}\right) = \mathrm{P}\left(\mathrm{S_{a,C}} \le \mathrm{S_{a,D}}\right) \tag{2.2}$$

where $S_{a,D}$ and $S_{a,C}$ represent the acceleration demand and the acceleration capacity of the structure at its fundamental period, respectively. It can be observed from the above equation that the fragility is expressed as the probability that the random variable $S_{a,C}$ is less than or equal to the value $S_{a,D}$. Therefore, the fragility can also be stated as the cumulative distribution function of the random capacity, $S_{a,C}$. If it is assumed that the probability distribution of the spectral acceleration capacity, $S_{a,C}$, is lognormal with median $\eta_{S_{a,C}}$ and standard deviation of the natural logarithm, $\beta_{S_{a,C}}$, the fragility can be expressed in terms of the "standardized" Gaussian distribution function:

$$\mathrm{P(C|E)} = \mathrm{P}\left(\mathrm{S_{a,C}} \le \mathrm{S_{a,D}}\right) = \Phi\left(\frac{\ln\left(\mathrm{S_{a,D}} / \eta_{\mathrm{S_{a,C}}}\right)}{\beta_{\mathrm{S_{a,C}}}}\right) \tag{2.3}$$

It can be observed from the above equation that the structural fragility can be plotted as a function of spectral acceleration.

In order to assess seismic hazard, the method of probabilistic seismic hazard analysis (PSHA) can be considered and the annual probability (or rate) of exceeding some values of the intensity levels need to be calculated. The results of PSHA are commonly represented by hazard curves, which specify the site ground motion intensity (or the spectral acceleration) as a function of the annual probability of exceedance.

2.2 WIND CONTRIBUTION TO PROBABILITY OF COLLAPSE

With regard to wind, the maximum wind speed can be adopted as the measure that reflects the wind intensity. Many authors have tried to apply statistical concepts to the estimation of design wind speed. Nevertheless, even if probabilistic characterization of wind hazard has been widely examined in literature, in most cases specific characterization of wind hazard is less refined than seismic case. Most studies on the probability analysis of wind speed are mainly concerned with the determination of the probability distribution of wind speed and the prediction of extreme wind speed. The classical extreme value theory is based on three asymptotic extreme value distributions (Gumbel, Frechet and Weibull distribution). The Generalized Extreme Value (GEV) distribution combines them into a single mathematical form with the following expression:

$$F(x) = \exp\left\{-\left[1 + k\frac{x-\mu}{\sigma}\right]^{-\frac{1}{k}}\right\} \tag{2.4}$$

where x is the maximum of an epoch and F(x) is the cumulative probability distribution function of variable x; σ, μ and k are the scale factor, the location factor and the shape factor, respectively.
When k = 0 the equation above becomes the Type I extreme value distribution, which is also known as the Gumbel distribution, that is the classical model for fitting extreme values. In fact, this is the most common model to evaluate the extreme wind speed and is the most used method adopted by structural design codes and standards. The cumulative distribution function F(x) of Type I can be written in the following form:

$$F(x) = \exp\left\{-\exp\left[-\frac{x-\mu}{\sigma}\right]\right\} \tag{2.5}$$

The associate probability density function f(x) is:

$$f(x) = \frac{1}{\sigma}\exp\left[-\frac{x-\mu}{\sigma}\right]\exp\left\{-\exp\left[-\frac{x-\mu}{\sigma}\right]\right\} \tag{2.6}$$

The probability of exceedance can be estimated, according to Gumbel Method (Gumbel 1954, 1958) or to Gringorten procedure (Gringorten 1963), by the expression:

$$P(W) = P_{exc} = 1 - F(x) \tag{2.7}$$

where F(x) is the value of the cumulative distribution function for a specific value of wind speed x.
If possible, real wind records for a station close to the site of the investigate structure can be used in order to estimate the annual frequency of exceeding a specific maximum velocity level. In fact recorded data can be elaborated and fitted by an extreme value distribution to calculate the probability of exceedance referred to a specific wind speed.
In this framework, if the wind intensity is represented in terms of wind speed, the wind fragility at a specific wind speed v_d can be expressed as:

$$P(C|W) = P\left(W_{v,D} > W_{v,C}\middle|W_{v,D} = v_d\right) = P\left(W_{v,C} \leq W_{v,D}\right) \tag{2.8}$$

The wind fragility can be assumed as a deterministic function of wind speed; in other words, wind fragility is equal to 1 in the case of collapse and 0 otherwise.

2.3 TOTAL RISK EVALUATION

Once wind and seismic fragilities are evaluated, they are implemented in Equation 2.1 in order to calculate the annual risk of collapse. The contribution of the seismic risk to Equation 2.1 is calculated by integrating the seismic fragility curve for the structure and the spectral acceleration hazard at a period close to the fundamental period of the structure. Similarly, the contribution of the wind risk is calculated by summing the wind hazard probabilities that cause the collapse of structure. The annual risk of collapse can then be compared with the acceptable threshold.

3. CASE STUDY

A possible application of the methodology described in the previous section can refer to steel hangars located in seismic zones. In particular, the calculation of annual risk of collapse of a steel hangar belonging to Italian Air Force located in Ciampino Airport is here presented. The hangar is characterized by six 12 m high circular steel-concrete composite columns and two 40x40 m wide truss gratings. Non-linearity is referred only to columns that are divided into five parts, each one characterized by a specific moment-curvature relationship, depending on the axial force (Mander 1988). Figure 1 depicts a refined finite element model of the structure in which steel trusses are modeled as frames. However, in this preliminary study, in order to evaluate the seismic fragility, the top floor has been modeled as a rigid diaphragms (Figure 2).

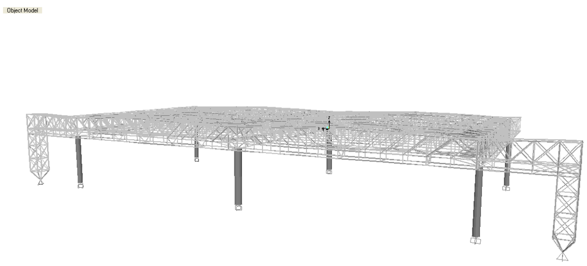

Figure 1. **Model for determination of linear dynamic properties**

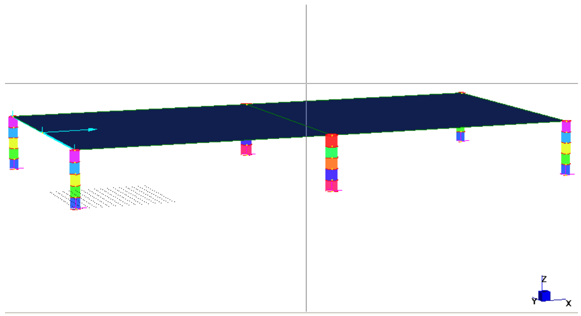

Figure 2. M**odel for non-linear response history analysis**

Seismic hazard has been characterized as the mean annual frequency of exceeding a given level of spectral acceleration at the fundamental period of structure. The hazard values are taken from the tabulated values in INGV, Italian National Institute of Geophysics and Volcanology, in which, the mean annual rate of exceeding an earthquake event of interest has been calculated using probabilistic seismic hazard analysis (PSHA) for the site of the structure. INGV has evaluated probabilistic seismic hazard for each node of a regular 5 km spacing grid that cover the whole Italian territory with over 13000 nodes (Meletti 2007). The results are provided in hazard curves in terms of PGA and spectral acceleration, Sa(T), for ten different periods from 0.1 to 2 s. Hazard curve for the site of interest is shown in Figure 3. In IDA approach the seismic motion has been represented in terms of ground acceleration time-histories. Seven recorded accelerograms have been chosen according to Italian Code specification using software REXEL (Iervolino 2008). The records were scaled from 0.1 g to 2.0 g. Figure 4 reports the spectra associated to the selected records.

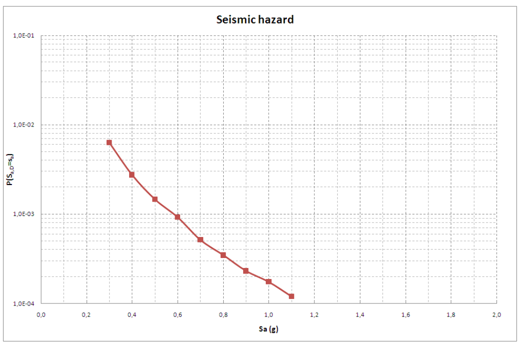

Figure 3. **Seismic hazard**

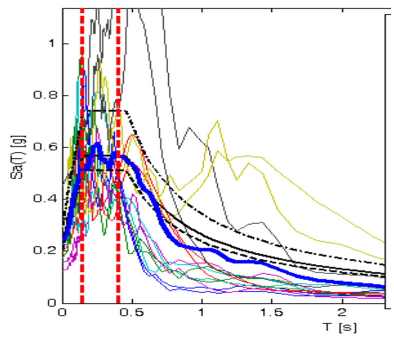

Figure 4. **Combination adopted**

Incremental Dynamic Analysis have been performed and the results in terms of multiple stripe are showed in Figure 5, where spectral acceleration is plotted against top displacement of the structure, for twenty different spectral acceleration from 0.1 g to 2.0 g. Figure 5 is plotted in log-log scale.

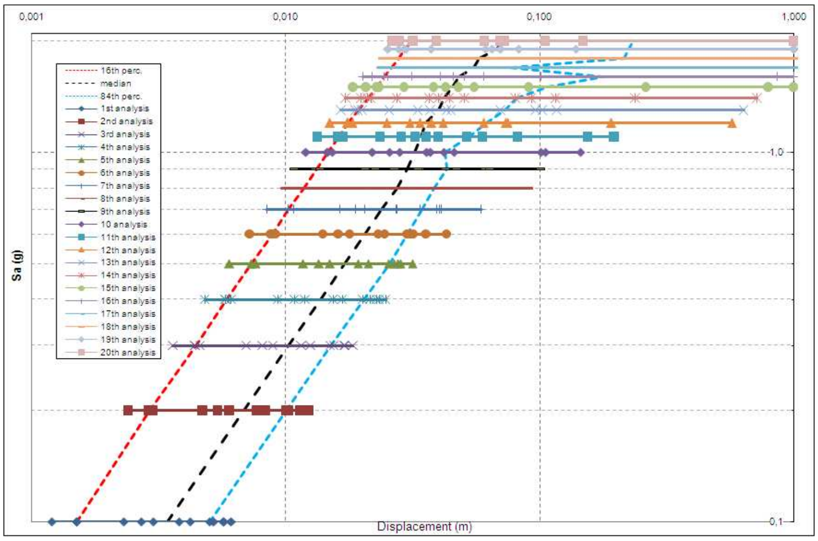

Figure 5. **IDA in terms of multiple stripes**

The three dashed lines in Figure 5 denote the 16th percentiles, the median values and the 84th percentiles of the stripes, respectively. Figure 6 shows another way to draw the results of IDA approach and, in particular, each line represents the analyses performed for an accelerogram for which the related spectral acceleration is scaled from 0.1 g to 2.0 g. The maximum drift associated to the limit state of collapse is calculated by integrating the moment-curvature relationships and it is equal to 0,085 m, as denoted by dashed line in Figure 6.

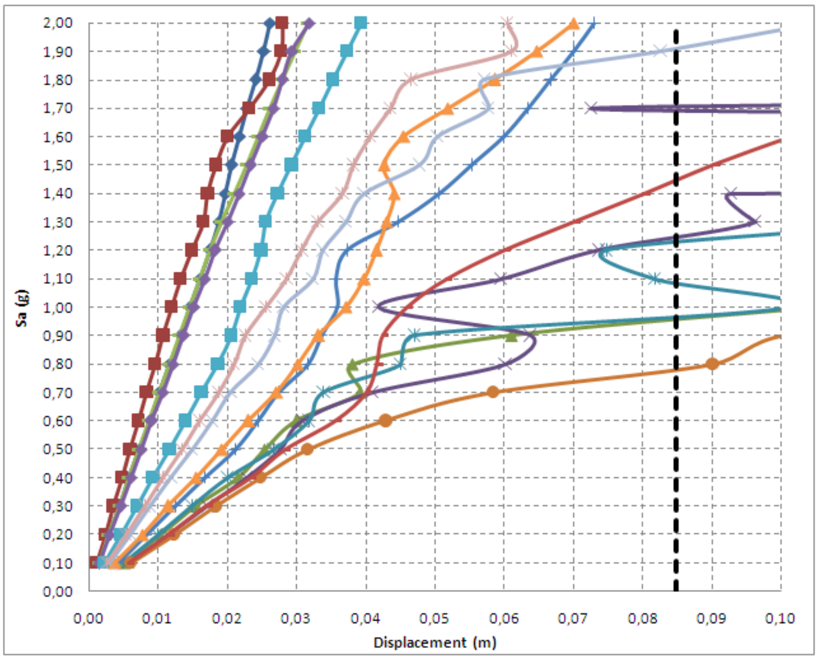

Figure 6. **IDA in terms of accelerograms**

It can be noted that some of them do not have a monotonically increasing trend and some others do not reach the maximum drift threshold.

Seismic fragility is depicted in Figure 7: the continuous line and dashed line are referred to the cases in which the standard deviation has been calculated with respect to the median value and the mean value, respectively.

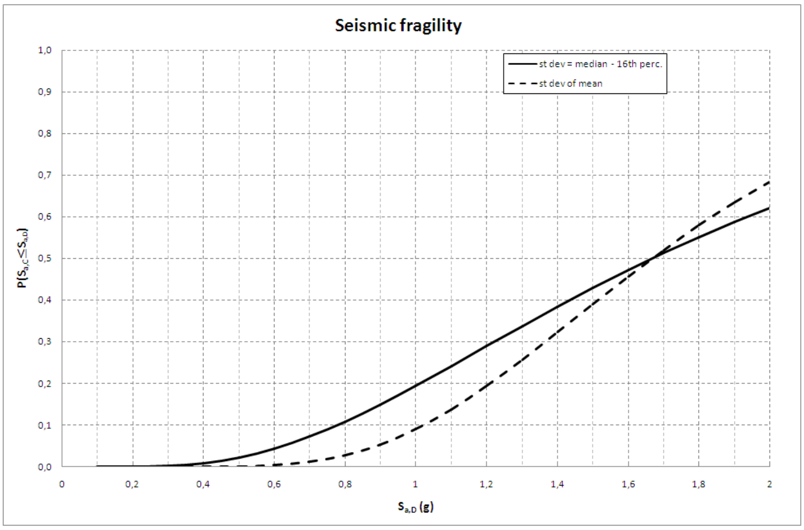

Figure 7. **Seismic fragility**

Integrating the seismic fragility curve and the seismic hazard curve, the annual frequency of collapse referred to seismic risk for the structure has been calculated and it is equal to $7{,}9 * 10^{-5}$, assuming the median value of stripe.

3.3 CHARACTERIZATION OF WIND HAZARD

The hourly wind speed data for the period 1951-2009 (58 years) recorded by Climate Department CNMCA of Italian Air Force are adopted herein to calculate the values of wind loads at the site of the structure. The probabilistic approach has been conducted with the asymptotic analysis (Fisher 1928, Gumbel 1958, Lagomarsino 1992) considering the annual maxima according to the Gumbel method and the Gringorten method. The results are shown in terms of probability of exceedance for different values of wind speed in Figure 8 and Figure 9, for the Gumbel and Gringorten methods, respectively. Figure 8 and Figure 9 are plotted in log-log scale and the y axis is in inverse order.

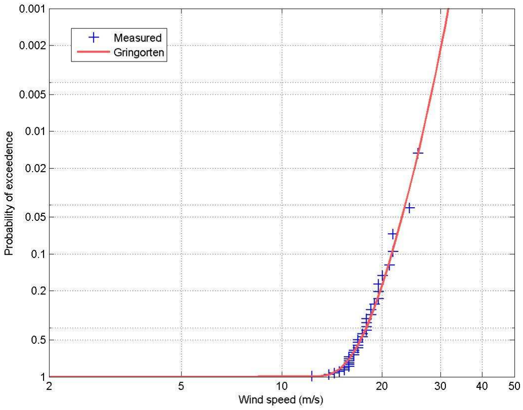

Figure 8. **Wind hazard for annual maxima by Gringorten method**

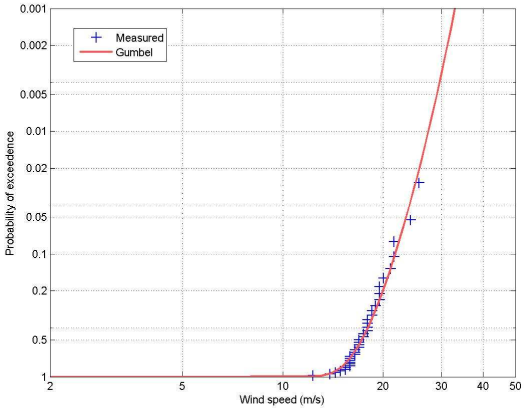

Figure 9. **Wind hazard for annual maxima by Gumbel method**

Moreover the wind estimation has been conducted by fitting the recorded wind data to the Generalized Extreme Value (GEV) distribution. The parameters of the distribution have been estimated through maximum likelihood method. The results are shown in Figure 10.

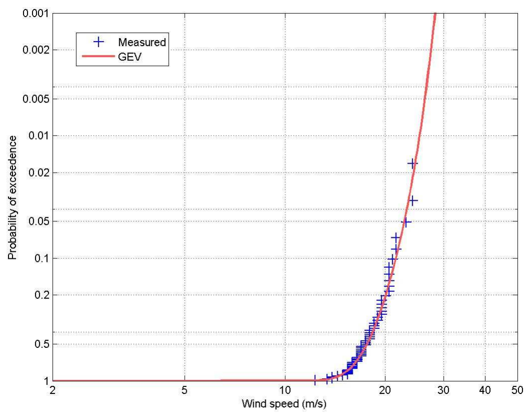

Figure 10. **Wind hazard for annual maxima by GEV distribution**

By disaggregating wind speed data by directions, the probability of collapse can be calculated by summing the contribution for each direction to have more refined estimation. In order to complete the calculation of the wind contribution to the risk of collapse, the failure mechanism induced by wind loads have to be analyzed. In fact wind loads can induce collapse by the uplift of the roof, by the yelding of the steel frames or by the failure of the steel joints. With regards to the disaggregation of the wind hazard by directions, it should be mentioned that the actual structural vulnerability of hangar structures strongly depends on the wind direction. This analysis is still on going.

4. CONCLUSIONS

The aim of the present work is to present a methodology for calculating the annual risk of collapse for steel hangars subjected to both seismic and wind threats, using a bi-hazard approach. The probability of collapse, due to earthquake, can be calculated by integrating the seismic fragility of the structure and the seismic hazard for the site. Since the wind vulnerability is assumed as a deterministic function of wind speed, the contribution of the wind risk is calculated by summing the wind hazard probabilities that cause the collapse of structure. Further investigations will focus on wind vulnerability of steel hangars and on the disaggregation of wind hazard by direction, in order to have a more refined calculation of total risk of collapse.

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