Available online at www.eccomasproceedia.org
Eccomas Proceedia COMPDYN (2025) 3032-3043

ECCOMAS
Proceedia

COMPDYN 2025
10th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis (eds.)
Rhodes Island, Greece, 15-18 June 2025

SEISMIC LOSS ESTIMATION OF STEEL PALLET RACKS

G. Mucedero[1], G. Gabbianelli[2], and R. Monteiro[1]

[1] University School for Advanced Studies, IUSS Pavia
Piazza della Vittoria,15, 27100 Pavia, Italy
e-mail: {gianrocco.mucedero, ricardo.monteiro}@iusspavia.it

[2] University of Pavia, Department of Civil Engineering and Architecture
Via Adolfo Ferrata, 3, 27100 Pavia, Italy
giammaria.gabbianelli@unipv.it

Abstract

Steel storage pallet racks are crucial for efficient industrial storage, reflected by the value of the European racking market of €5.2 billion in 2019. Earthquakes threaten these structures, as seen in Italy and other locations. Despite their importance, seismic loss assessments for steel racks have received limited attention, hindering prevention and risk mitigation efforts. This study addresses this gap by examining the seismic vulnerability of steel storage racks and employing a new practice-oriented seismic loss assessment methodology to quantify risk metrics (expected annual losses and total losses). The methodology is applied to three case-study steel rack prototypes, with different span lengths in the down-aisle direction, assumed to be located in an Italian location with moderate seismic hazard level. The results include collapse fragility curves and risk metrics, as well as the contribution of each component type (e.g., structural components, contents) to total losses. The adopted practice-oriented seismic loss assessment methodology proved useful to support industrial stakeholders in understanding and quantifying the seismic risk associated with rack systems, facilitating risk management.

Keywords: Logistic Infrastructure, Steel storage racks, Loss assessment, Seismic vulnerability; Multiple-Stripe Analysis.

Peer-review under responsibility of the organizing committee of COMPDYN 2025.
doi: 10.7712/120125.12631.25388

1 INTRODUCTION

Steel storage racks are used globally, providing versatile and efficient storage solutions in various sectors such as warehousing, logistics, retail, and manufacturing. Their adaptability and capacity to hold a wide range of goods make them important for several business activities. In logistics and supply chains, they facilitate systematic product arrangement for easy retrieval and transportation. In retail, they improve merchandise display and space organisation. Manufacturing facilities use steel racks to organise materials, reducing downtime and optimising production. The European market for racking and storage equipment was approximately €5.2 billion in 2019. These structures are essential to the supply chain, especially during pandemics or disasters, as they need to function without interruption. Earthquakes have demonstrated their high seismic vulnerability, impacting delivery and supply continuity, as evidenced by the 2012 Emilia-Romagna and 2016 Central Italy earthquakes [1].

Design-wise, steel storage racks aim to maximise space utilisation and display goods effectively. Initially, their design focused on gravity loads, while seismic considerations were integrated in the 1990s. Studies have since examined the seismic behaviour of racks, including connector stiffness and base isolation systems. European research projects have improved standards for static and seismic design, addressing issues like pallet-rack sliding and the stability of unbraced and braced racks.

Further studies on beam-to-column and base-plate connections have shown that cyclic behaviour and pinching effects reduce energy dissipation, affecting seismic performance [2-4]. Research on transverse stability has identified initial failure mechanisms, such as diagonal instability. Advanced numerical simulations consider warping effects on thin-walled cross-sections, revealing that ignoring these effects can overestimate rack capacity [5]. Assessing seismic vulnerability has led to fragility curves and high vulnerability indicators, prompting new design strategies and performance improvements, like base isolation and tuned-mass dampers. However, less attention was paid in the past to conceive comprehensive methodologies for the seismic loss assessment of steel racks. While advanced methods, such as the PEER-PBEE framework [6], exist, they require detailed component-based fragility models, which are lacking for rack components. Recently, a practice-oriented methodology for seismic loss assessment of steel was proposed by [7], to estimate the seismic risk metrics, such as expected annual losses, total losses or repair time.

This study investigates the seismic performance of three case-study steel rack prototypes, with different span lengths in the down-aisle direction, assumed to be located in an Italian location with moderate hazard level. Collapse fragility curves are derived and seismic risk metrics are quantified by employing the above-mentioned methodology.

2 LOSS ASSESSMENT METHODOLOGY

Most of the available methods for buildings and infrastructure require damage estimation phases, which involve fragility curves for structural components and contents. However, steel racks lack such component-based consequence models, presenting challenges in damage assessment and fragility curve derivation due to limited data, variability in design, and limited understanding of failure modes. At the same time, a comprehensive risk management framework, including simplified methodologies, is essential for identifying potential vulnerabilities and planning retrofitting interventions and investments. Insurance companies also require such assessments for underwriting purposes and setting premiums, driven by new government directives mandating insurance against catastrophic risks.

This study employs a recently proposed [7] loss assessment methodology for steel storage racks (Figure 1), which consists of six main steps:

1. **Numerical Model**: Assess the lateral response performance of racks based on geometry and load configuration. Uprights and beams can be modelled as elastic elements, but connections must be accurately modelled with nonlinear springs. P-delta formulations are necessary for second-order effects. Damage states are assessed by monitoring rotations and resistance and stability checks.
2. **Seismic Hazard**: Characterise seismic hazard at the site and define the hazard curve. Perform probabilistic seismic hazard analysis and select an appropriate intensity measure (IM), e.g. the average spectral acceleration (AvgSa) for its suitability to the structural typology of racks.
3. **Physical Damage**: Assess damage extent and severity based on ground motions and rack response. Identify damage types and quantify severity using a standardised damage scale. Two damage states are defined: DS1 (repairable component damage) and DS2 (failure of the component).
4. **Structural Analysis**: Conduct multiple stripe analysis (MSA) for different IM levels to quantify engineering demand parameters (EDPs) like peak floor acceleration and interstorey drift ratio.
5. **Repair/Replacement Time and Costs**: Integrate repair/replacement time and costs with exposure and vulnerability models for realistic loss estimation. This includes direct costs (materials, labour) and indirect costs (business interruption).
6. **Loss Assessment**: Use structural analysis results, damage identification, and repair/replacement costs for each IM level. Instead of querying the components' damage fragility curves (FEMA P-58 [8]), the achievement of each damage state is controlled by the structural model itself, querying the backbone curves of the rack components and assuming a pre-defined damage threshold (Step 3) to check the achievement of the damage states.

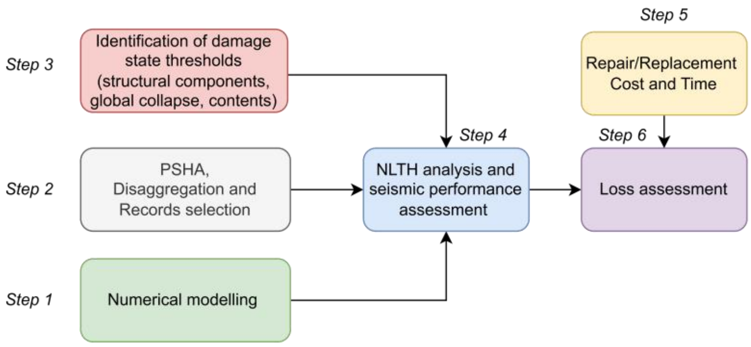


Figure 1: Main steps of the loss assessment methodology for steel storage racks [].

The loss assessment methodology focuses on repairing steel storage racks to their pre-earthquake condition, involving different assumptions. While Steps 1, 2, and 4 of Figure 1 are straightforward, further explanation is needed for Steps 3, 5, and 6. In Step 3, the main assumption is that replacing components is preferred over repairing them. This approach defines two damage states (DSs): DS1, which involves the replacement of single/multiple components of the rack, whose damage cannot compromise the rack's structural integrity, and DS2, where significant damage leads to collapse.

The lack of consequence models for steel racks renders the loss estimation for non-collapse cases more challenging. Instead of using damage fragility curves, the methodology uses backbone curves of components to determine damage states during nonlinear time history (NLTH) analysis. The damage thresholds consist in: yielding (DS1) and ultimate (DS2) rotation for beam-to-upright connections; yielding (DS1), ultimate (DS2) rotation, tensile and shear failure

(DS2) for base-plate connections; yielding (DS1) and ultimate (DS2) chord rotation, Von Mises criterion for one point of the section (DS1) or 25% of the section (DS2) for the uprights; Von Mises criterion for one point of the section (DS1) or the 25% of the section (DS2) for the beams; yielding (DS1) and plastic (DS2) tensile resistance and 85% (DS1) and 100% (DS2) of the buckling resistance for the braces; and peak floor acceleration for triggering overturning and sliding (DS2) for the pallets.

If a decision-maker considers demolition, they need to define a demolition fragility curve and query it by using structural analysis results. The damage to contents, which are often more valuable than the racks themselves, is assumed to be reached when the peak floor acceleration exceeds a pre-defined threshold. Although detailed methods could be used to explicitly model the contents [9], simplified approaches are preferred when used by practitioners.

In Step 5, repair/replacement costs and time are quantified. Both constant and piece-wise consequence functions might be employed within the methodology; in this study, constant repair/replacement costs are used for the three case studies. For the uprights, the minimum quantity for replacement and the minimum cost are one and €240, respectively. For beams are two and €41, for braces are one and €203, for beam-to-upright connections are two and €41 (considered welded to the beam), and for base-plate connections are two and €32, respectively. More details on the damage thresholds for different structural components and the methodology are provided in [7].

3 STEEL RACK PROTOTYPES

The material and geometrical properties of the selected case-study steel rack are based on a past project SEISRACKS2 [10], which experimentally assessed the seismic performance of steel racks. Three steel rack configurations were generated (Figure 2) by varying the span length of the one experimentally tested in SEISRACKS2 [10]; in particular, the different span lengths considered in the down-aisle direction are 2.0, 2.5 and 3.0 m, and the corresponding rack typologies are named as IPB2, IPB2.5 and IPB3, respectively. The prototypes share the same configuration, namely four bays in the down-aisle direction, one bay (span length of 1.1m) in the cross-aisle direction and four loading levels. Each loading level is 2m high, and each beam is loaded with 4.3kN/m. The numerical models were developed in OpenSeesPy [11]; each member was modelled with elastic beam-column elements with a P-delta formulation for the geometrical nonlinearities. Further details on the numerical modelling can be found in [7].

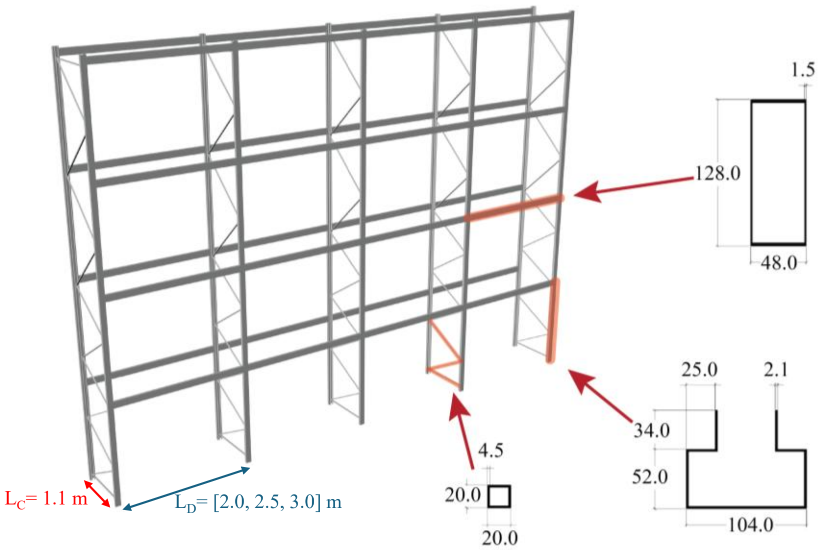
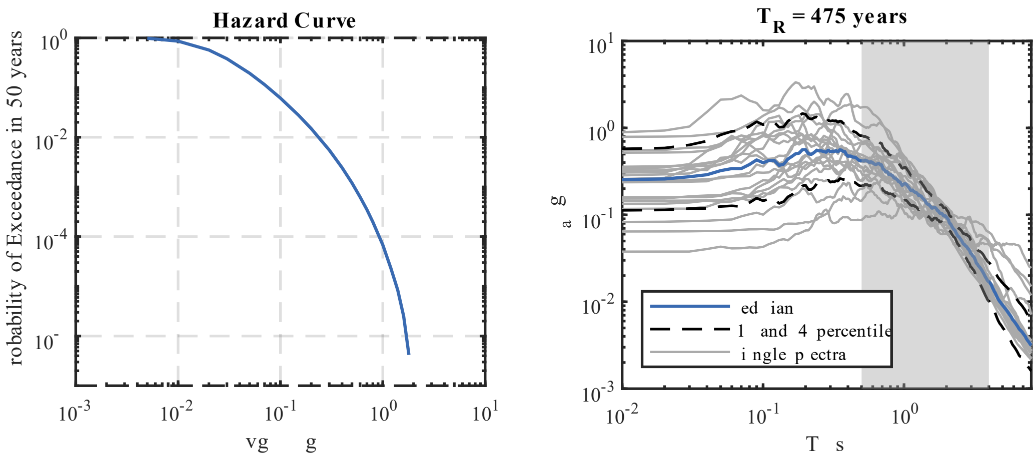


Figure 2: Geometrical properties of the three steel storage racks.

3.1 Hazard characterisation and record selection

To conduct MSA, different sets of records were selected for each IM level. Probabilistic seismic hazard analysis (PSHA) was carried out, using OpenQuake [12], to define the hazard curve for the investigated location, Finale Emilia, characterised by moderate seismic hazard. Soil type C was assumed, following the Eurocode 8 [13] and NTC18 [14] classifications. The target spectrum was the conditional spectrum (CS), based on average spectral acceleration (AvgSA), defined in the period range of $0.2T_1$ to $1.5T_1$ [15] (T_1 is the fundamental period of the structure). The resulting period range is 0.5s to 4.0s, while ten return periods (T_R) were considered, with 20 records selected for each T_R.



(a) (b)

Figure 3: (a) Hazard curve for the selected location and (b) example of selected spectra for T_R= 475 years, with grey area representing the period range adopted for the AvgSA record selection approach.

3.2 Peak floor accelerations and interstorey drift ratios

Following the MSA, the median profile and the 16th and 84th percentiles of the peak floor accelerations (PFAs) and interstorey drift ratios (IDRs) of the different racks and directions were analysed, for the different return periods (T_R). Figures 4 and Figure **5** illustrate the PFA profiles for the down- and cross-aisle direction, respectively, of the racks for six T_R. In the down-aisle direction (Figure 4) and for the lower T_R, the acceleration remains quite constant along the height, regardless of the rack typology. The dispersion for the three racks is almost similar, as seen from the 16th and 84th percentile profiles. For higher T_R, the differences in terms of medians and percentiles become more evident. Specifically, for T_R = 975 years and T_R = 2475 years, the acceleration profiles show a reduction with increasing height, with a slight increase at the top floor. Regarding the cross-aisle direction (Figure 5), the profiles show how the racks are more rigid, and the higher modes generally have less influence on the structural response. This is easily seen from all the represented profiles that show a nearly linear increase in accelerations with height. Regardless of the direction and T_R, IPB2 shows higher PFA values compared to IPB2.5 and IPD3, clearly due to the higher stiffness of IPB2, which has a smaller span length.

Figures 6 and Figure **7** illustrate IDR profiles for the down- and cross-aisle directions, respectively. In the down-aisle direction (Figure 6), for the first two T_Rs, a nearly constant IDR can be observed along the height of the racks, with dispersion levels that are not significant; the dispersion levels strongly increase for higher T_R. As regards the cross-aisle direction (Figure 7), similar considerations can be made. The IDR values and their dispersions are quite low for low return periods (T_R = 63 years and T_R = 98 years), while they strongly increase for higher return periods. Regarding the differences between rack typologies, IPD3 shows higher IDR values in both directions and almost for all the investigated T_Rs, while all the rack types show a soft storey mechanism at the third storey for higher T_Rs.

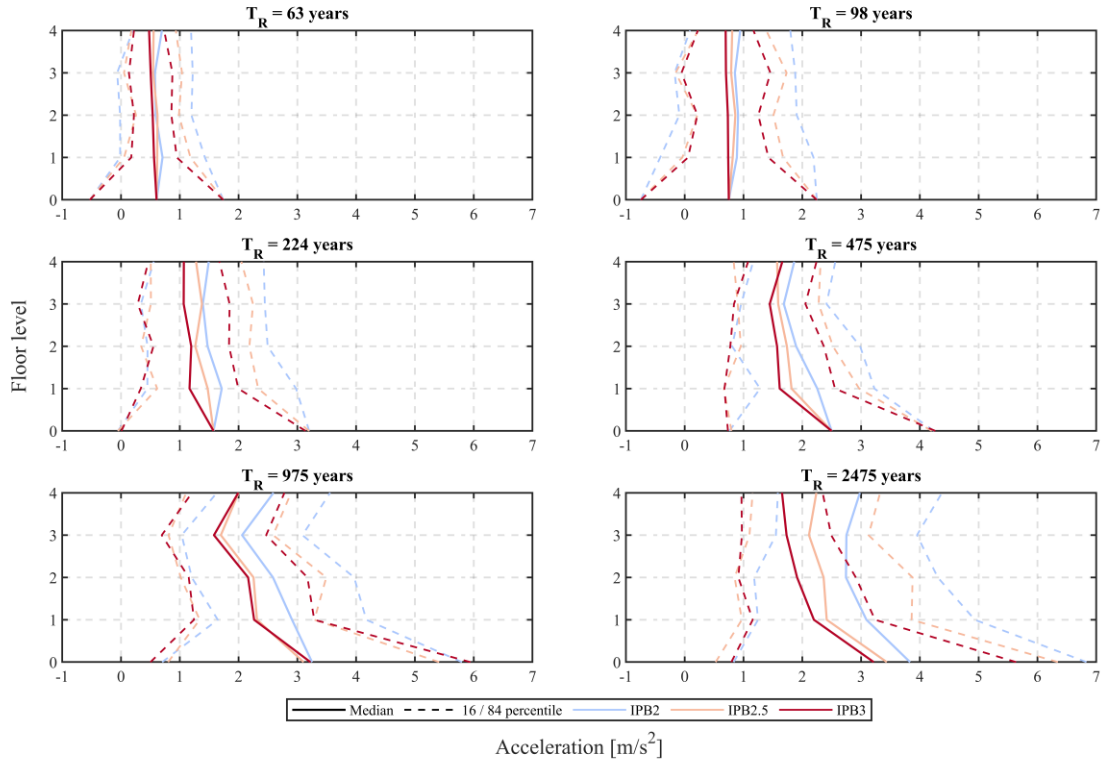


Figure 4: Acceleration profiles in the down-aisle direction of the three rack typologies, as a function of the T_R.

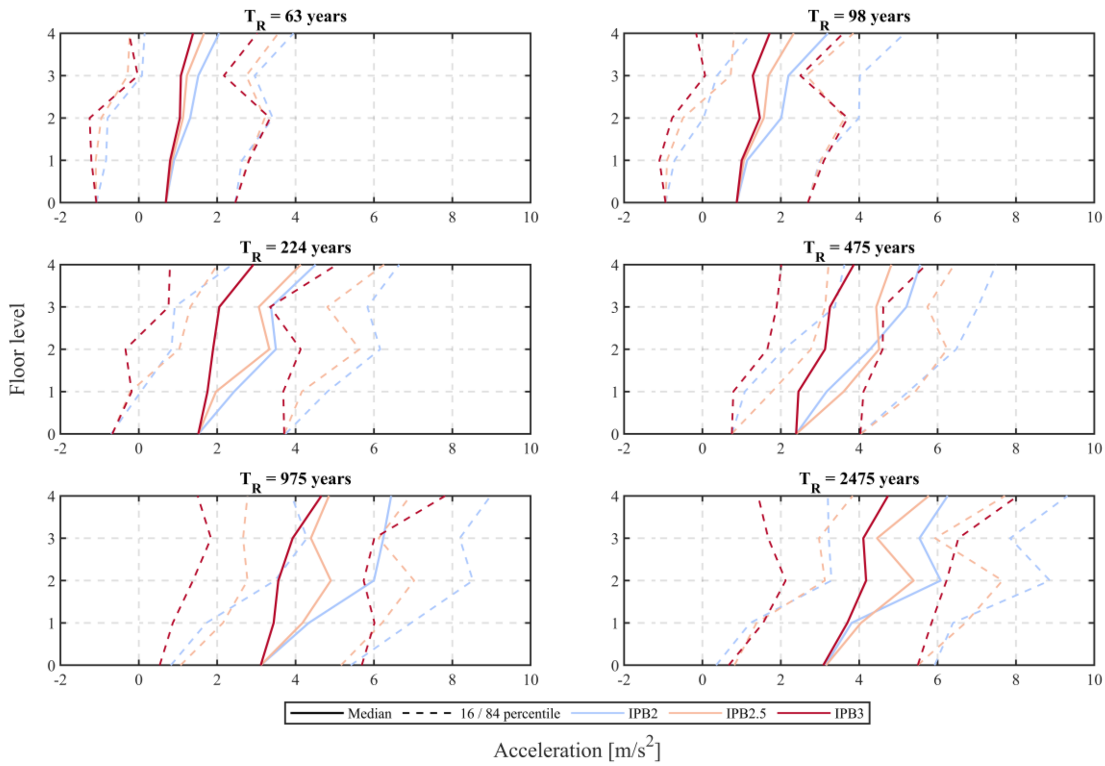


Figure 5: Acceleration profiles in the cross-aisle direction of the three rack typologies, as a function of the T_R.

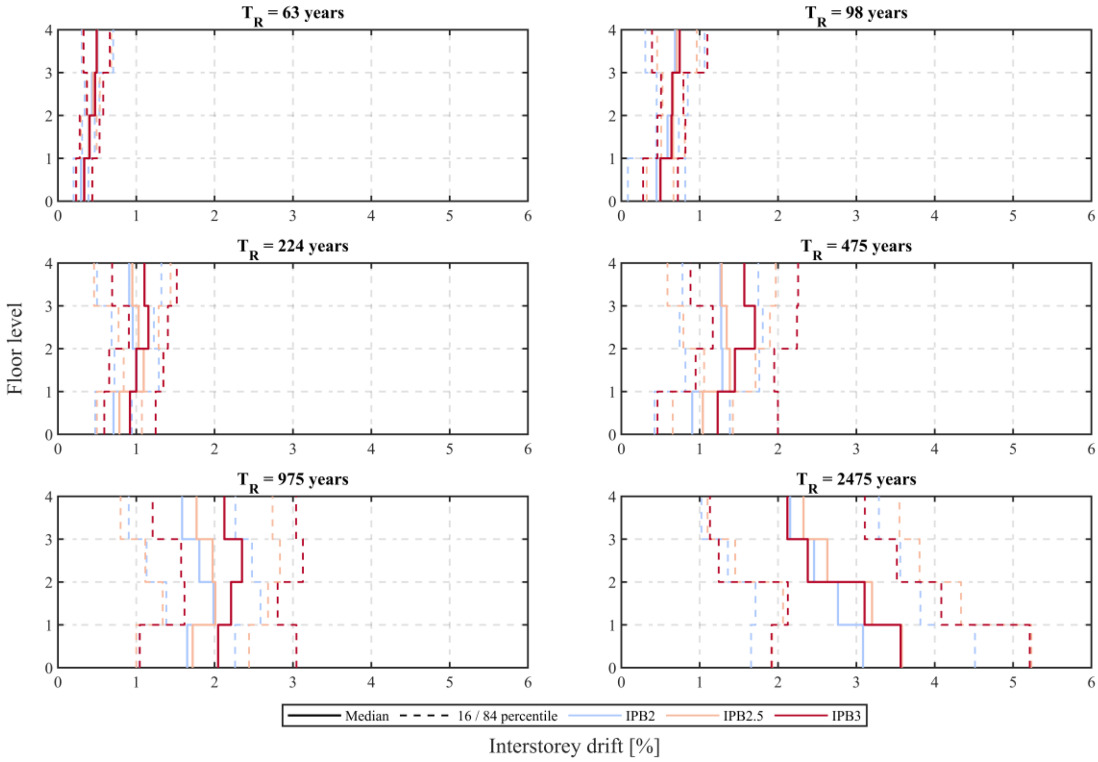


Figure 6: IDR profiles in the down-aisle direction for the three rack typologies, as a function of the T_R.

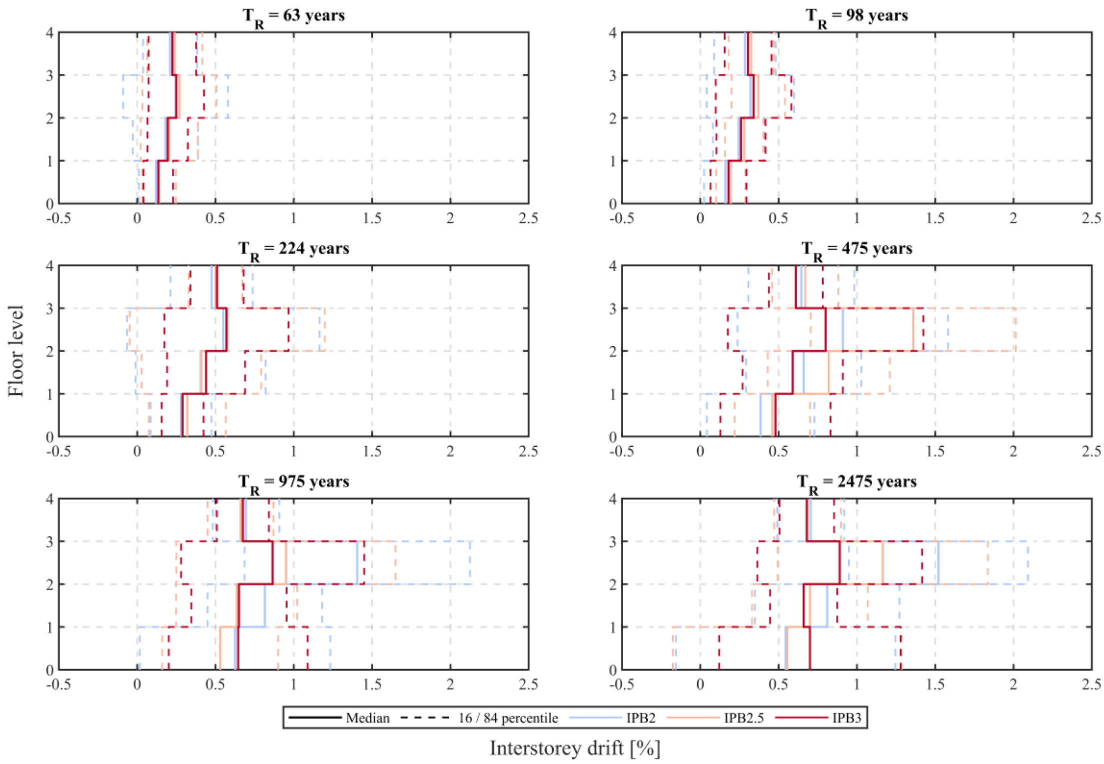


Figure 7: IDR profiles in cross-aisle direction for the three rack typologies, as a function of the T_R.

3.3 Collapse fragility

The collapse fragility functions were derived for each of the three rack typologies, and the collapse was defined once 25% of the structural components reached failure. The lognormal distribution was used to fit the collapse probability data points, defining thus a median collapse intensity, θ, and a logarithmic standard deviation, β_{RTR}, associated with the record-to-record variability. Although a comprehensive consideration of the uncertainty would require including both the epistemic uncertainty (β_{MDL}) and damage level definition uncertainty (β_{DS}) [16], the dispersion of the fragility curves provided in Figure 8 was not inflated considering such uncertainties. This choice was motivated by the lack of research studies proposing epistemic dispersion sets and uncertainty on the definition of damage levels for steel storage racks. As expected, IPB2 shows the lower probability of collapse, for the same IM level, compared to the other two rack typologies. The observed θ values were 0.17, 0.14 and 0.12 for IPB2, IPB2.5 and IPB3, respectively. With increasing span length, θ reduces of about [14.3-17.6] %, with the lower β_{RTR} (0.52) found for IPB3, while it is 0.66 and 0.62 for IPB2 and IPB2.5, respectively.

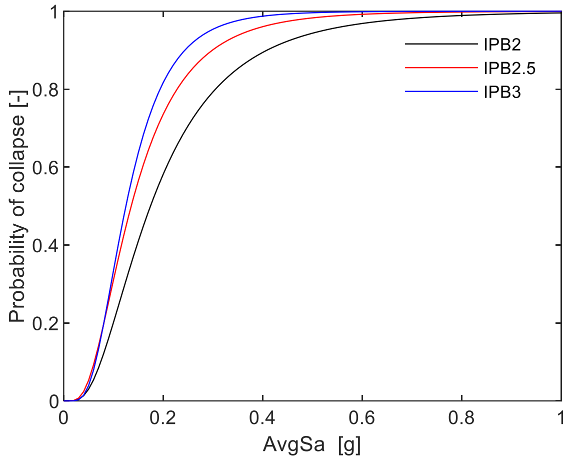


Figure 8: Collapse fragility curves, as a function of the rack typology.

4 LOSS ASSESSMENT RESULTS

The loss assessment, carried out employing the practice-oriented methodology proposed by Mucedero et al. [7], assumed the cost of contents as € 50,000.00 for each storey of the rack, whereas the structural components cost (€ 16,708.00) was estimated by a European company of the sector. The total reconstruction cost (€ 218,378.80) was obtained by inflating the total cost (€ 216,708.00) of 10% to account for the demolition costs. The peak floor acceleration capacity in the cross-aisle direction ($PFA_{tras,ovt}$) was assumed to be 0.3 g, which is slightly lower than the median intensity of collapse experimentally obtained by Filiatrault and Bachman [17].

The relative contribution to losses with decreasing mean annual frequency of exceedance ($\lambda = 1/T_R$) is presented in Figure 9. The loss curves are distinguished into: (i) collapse, representing the losses in the case of collapse ($E[L_C \mid IM]$) and (ii) non-collapse, $E[L_R \mid NC \mid IM]$, disaggregated into losses given the repairing (replacement) costs of the structural components and contents losses. For lower return periods (T_R), the higher contribution to the losses is due to the contents falling from the rack, while for increasing T_R, the higher contribution to the total losses is given by the collapse cases. Regardless of the rack typology, the contribution to the total losses of the repairing (replacement) costs of the structural components is negligible.

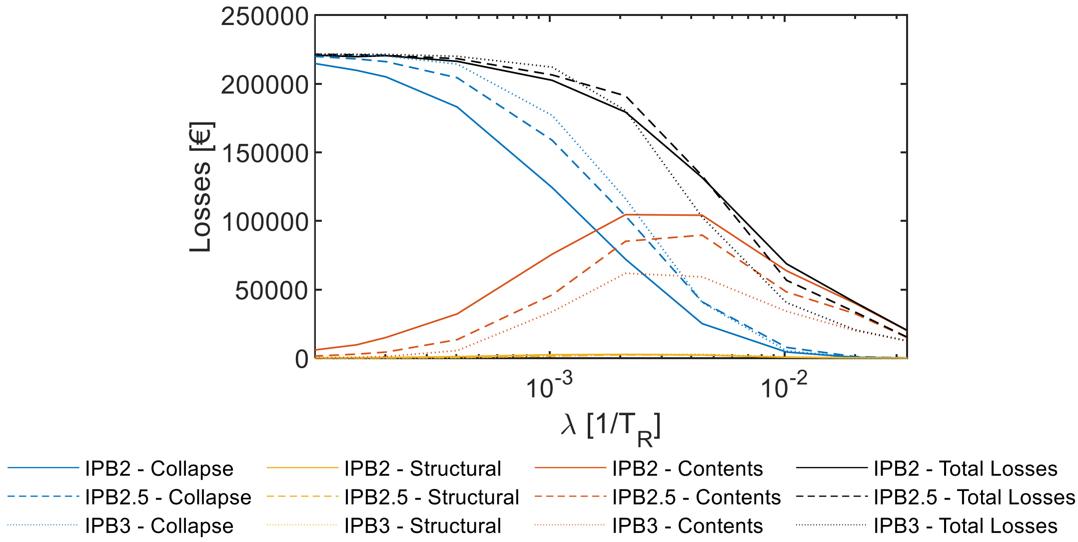


Figure 9: Relative contribution to losses with increasing return period.

A further comparison in terms of content losses and total losses for different rack typologies was carried out and is presented in Figure 10. Even at lower return periods (T_R = 98 years), the content losses (Figure 10a) for the three racks are quite different, highlighting the impact of span length on the expected losses; the content losses of IPB2 are almost 1.2 and 1.7 times higher than those of IPB2.5 and IPB3, respectively. Increasing T_R, the content losses are even more sensitive to the span length. As an example, for T_R = 475 years, the content losses for the IPB2 rack are 2.4 and 5.9 times higher than those of IPB2.5 and IPB3, respectively. This is justified by the lower probability of collapse at T_R = 475 years for the IPB2, resulting in a higher contribution of the non-collapse cases to the overall losses, as well as by higher PFA values. The maximum content losses are obtained between T_R = 224 years and T_R = 475 years. As for the total losses (Figure 10b), for T_R = 50 years, changing the rack typology leads to a variation of the total losses in the range of about 17% to 50%. For higher return periods (higher than 475 years), changes in the rack typologies have a limited effect (< 1%) on the total losses.

The obtained EAL, in percentage of the reconstruction cost, per rack typology, are depicted in Figure 11a, which shows an overall EAL variation in the range of [1.03 - 0.76] %, considered important according to the literature. The higher EAL is observed for the IPB2 rack, while the lowest is obtained for the IPB3. Hence, for this case study, changing the span length from 2 to 3m led to an EAL reduction of 26%.

For a better understanding of the relative contribution of structural components, contents and collapse losses to the EAL, a disaggregation was carried out (Figure 11b), per rack typology. Figure 11b further clarifies that the structural components' repair (replacement) costs have a limited effect on the overall EAL, with a higher relative contribution of 1.62% for the IPB3 rack. Conversely, the largest contribution to the EAL is provided by the losses of contents, which contribute to the total EAL in the range of [57 – 77.5] %, with the higher value being obtained for the IPB2 rack. This is due to the better seismic performance of this rack typology at lower return periods (lower probability of collapse at TR = 475 years) and higher PFA values. As for the relative contribution of the collapse cases to the EAL, it is in the range of [21.5 – 41.4] %, with the higher value obtained for the IPB3 rack. Comparing the different rack typologies, while the structural losses are of the same magnitude, by increasing the span length, the relative contribution of the collapse cases to the EAL increases of almost 93%, whereas that of the contents decreases by almost 26%.

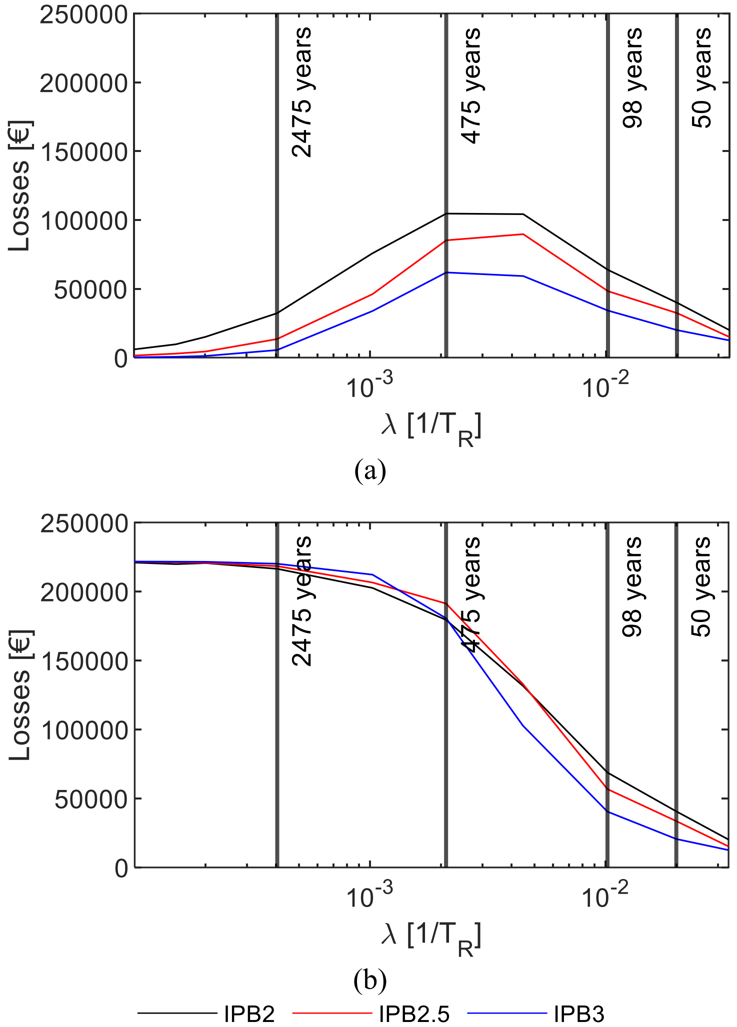


Figure 10: Comparison between different rack typologies in terms of (a) content losses and (b) total losses.

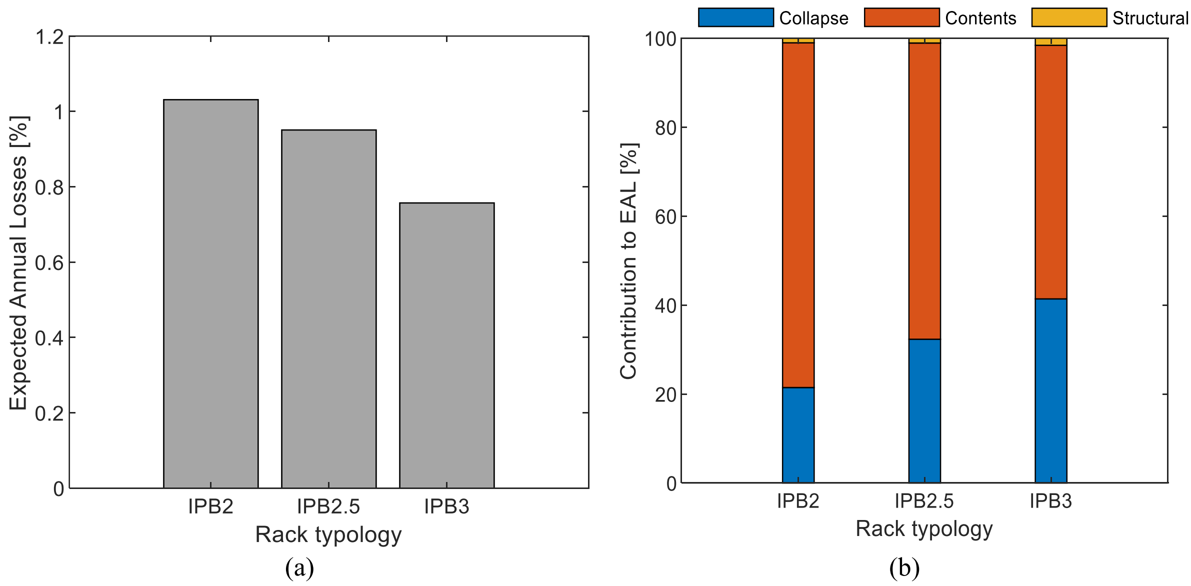


Figure 11: (a) Expected Annual Losses (EAL) and relative contribution to the EAL, as a function of the rack typology.

5 CONCLUSIONS

Research studies on steel racks have primarily focused on their design and structural performance evaluation, with limited attention being paid to the estimation of their seismic losses. This study bridged this gap by employing a recently proposed practice-oriented loss assessment methodology, providing a more holistic understanding of the seismic risk associated with steel rack systems. The methodology was applied to three different steel rack prototypes, which share the same material and geometrical properties, except for the span length. The results showed that, for lower return periods, the primary contribution to losses is due to contents falling from the racks, while, as the return period increases, the major contribution to total losses shifts to the collapse cases. The expected annual losses (EAL), as a percentage of the reconstruction cost, range from 0.76% to 1.03%, with the largest contribution coming from content losses. The impact of structural component replacement costs is minimal, whereas collapse cases contribute up to 41.4% of the EAL. For higher return periods (> 475 years), changes in the rack typology (i.e. the span length) have a limited effect on total losses but a relevant impact on the relative contribution of the collapse cases and content losses to the EAL.

Future analyses should include a portfolio of steel racks with various structural and geometrical configurations, reflecting different total construction costs, to draw more general conclusions. Furthermore, repair time and business interruption should be quantified from a functional recovery perspective, which is valuable for stakeholders and (re)insurance companies. A more comprehensive quantification of risk metrics for steel storage racks should also account for epistemic uncertainty and uncertainty in defining damage levels.

ACKNOWLEDGMENTS

The study was developed within the framework of the project “DE R CK” funded by the European Union - Next Generation EU, Mission 4 Component C2 CUP I53D23001780006.

REFERENCES

[1] D. errone, . . Calvi, R. Nascimbene, E. C. Fischer, and G. a gliulo, “ eismic performance of non-structural elements during the 201 C entral Italy earthquake,” *Bulletin of Earthquake Engineering*, vol. 17, no. 10, 2019, doi: 10.1007/s10518-018-0361-5.

[2] L. Yin, G. Tang, . Zhang, B. Wang, and B. Feng, “ onot onic and cyclic response of speed-lock connections with bolts in storage racks,” *Eng Struct*, vol. 116, 2016, doi: 10.1016/j.engstruct.2016.02.032.

[3] O. Bové, F. López- lma nsa, . Ferrer, and F. Roure, “Ductility improvement of adjustable pallet rack speed-lock connections: Experimental study,” *J Constr Steel Res*, vol. 188, 2022, doi: 10.1016/j.jcsr.2021.107015.

[4] O. Bové, M. Ferrer, F. López- lmans a, and F. Roure, “Numerical investigation on a seismic testing campaign on adjustable pallet rack speed-lock connections,” *Eng Struct*, vol. 252, 2022, doi: 10.1016/j.engstruct.2021.113653.

[5] C. Bernuzzi, . Gobetti, G. Gabbianelli, and . imoncelli, “Warping influence on the resistance of uprights in steel storage pallet racks,” *J Constr Steel Res*, vol. 101, 2014, doi: 10.1016/j.jcsr.2014.05.014.

[6] G. G. Deierlein, H. Krawinkler, and C. . Cornell, “ framework for performance-based earthquake engineering G.G.,” in *2003 Pacific Conference on Earthquake Engineering*, 2003.

[7] G. uc edero, G. Gabbianelli, . Rapone, and R. ont eiro, " practice-oriented methodology for seismic loss assessment of steel storage pallet racks," *Eng Struct*, vol. 333, p. 120187, Jun. 2025, doi: 10.1016/J.ENGSTRUCT.2025.120187.

[8] FE (Federal Emergency a nagement), "FE -58-1 Seismic Performance Assessment of Buildings Volume 1 – e thodology," *Federal Emergency Management Agency*, vol. 1, no. August, 2012.

[9] D. Tsarpalis, D. Vamvatsikos, and I. Vayas, " eismic assessment approaches for mass-dominant sliding contents: The case of storage racks," *Earthq Eng Struct Dyn*, vol. 51, no. 4, 2022, doi: 10.1002/eqe.3592.

[10] C. A. Castiglioni *et al.*, *Seismic behaviour of steel storage pallet racking systems (SEISRACKS2)*. Research Program of the Research Fund for Coal and Steel, European Union, Brussels, Belgium, 2016.

[11] . Zhu, F. c Kenna, and . H. cott, "Open ees y: ython library for the Open ees finite element framework," *SoftwareX*, 2018, doi: 10.1016/j.softx.2017.10.009.

[12] V. ilva, H. Crowley, . agani, D. one lli, and R. inho, "Development of the OpenQuake engine, the Global Earthquake ode l's open-source software for seismic risk assessment," *Natural Hazards*, vol. 72, no. 3, 2014, doi: 10.1007/s11069-013-0618-x.

[13] EN1998-1, *Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings*. European Committee for Standardisation, Brussels, Belgium, 2011.

[14] NTC18, *Norme Tecniche per le Costruzioni. DM 17/1/2018*. Gazzetta Ufficiale della Repubblica Italiana, Italian Ministry of Infrastructure and Transport, Rome, Italy, 2018.

[15] . Kohrangi, . Bazzurro, D. Vamvatsikos, and . pillatura, "Conditional spectrum-based ground motion record selection using average spectral acceleration," *Earthq Eng Struct Dyn*, 2017, doi: 10.1002/eqe.2876.

[16] G. uc edero, D. errone, and R. ont eiro, "Infill Variability and ode lling Uncertainty Implications on the eismic Loss sse ssment of an Existing RC Italian chool Building," Appl. Sci., vol. 12, no. 23, 2022, doi: 10.3390/app122312002.

[17] . Filiatrault and R. Bachman, "Fragility Curves for torage Racks," Washington, D.C., US, May 2011.