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Seismic Performance of Currently Designed Automated Rack Supported Warehouses

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Abstract **–** The paper analyses the seismic performance of Automated Rack Supported Warehouses (ARSWs), huge steel buildings developed for an optimized storage and management of palletized goods. These buildings have a load-bearing structure made up of steel racks that have the same peculiar characteristic of traditional steel racks, but with the relevant difference that the formers are demanded to resist to all the types of loads (e.g., wind, snow, earthquakes, gravity loads) while the latter to only the storage loads and the eventual associated inertial forces in case of earthquakes. Despite this relevant difference, the ARSWs have inherited almost all the structural characteristics of the traditional steel racks, resulting often in a non-satisfactory structural behaviour, as highlighted by recent collapses of ARSWs after seismic events. With the aim of assessing the seismic behaviour of ARSWs designed following the current guidelines, the paper presents 5 ARSW structures designed from 5 big European companies, analysing the different design approaches and hypothesis, and the consequences of these different choices in terms of seismic demand and performance. The results presented point out that a proper and dedicated design approach is needed for ARSWs.

Keywords**:** Automated Rack Supported Warehouse, steel rack, cold formed components, seismic performance.

1. Introduction

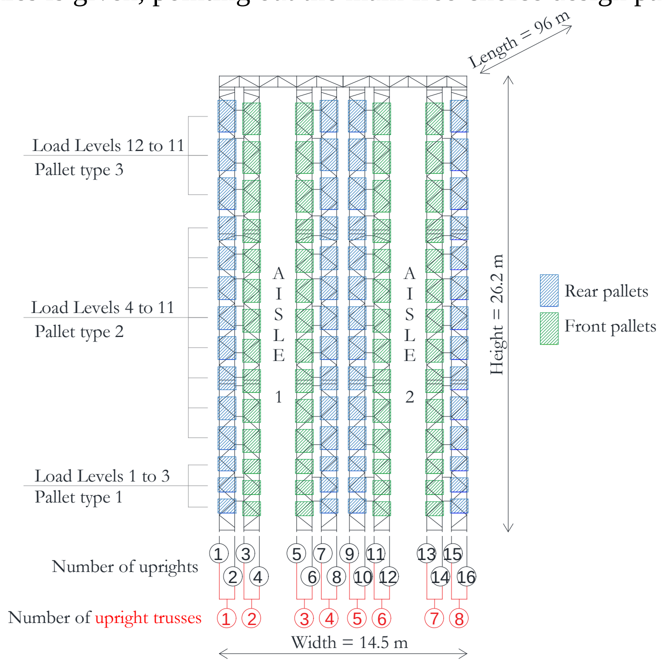
Automated Rack Supported Warehouses (ARSWs) are building offering optimized storage solutions. In these constructions, steel racks are used as both stocking spots and primary structural system of the warehouse. This is an upgrade of the normal use of traditional racks, which are placed inside warehouses for the storage purpose, only. Anyway, the use of such systems was not accompanied by the development of a dedicated regulatory framework for their design, implying the adoption of the technical standards for traditional racks (prEN15512 [1] for the static design, and prEN16681 [2] for the seismic design) or for buildings (such Eurocodes). Even if guidelines developed on the base of the design rules for traditional steel racks are not capable of offering a reliable design solution for the structure of ARSWs, the indications and rules provided by Eurocodes are equally not suited for these structures because tailored on deeply different geometries and structural configurations [6]. Indeed, referring to [1] and [2], these are made for traditional racks and are based on scientific outcomes of research projects about racks ([3], [4]). The structural functioning significantly changes from the traditional racks to the ARSWs', where racks additionally become the primary system of the building. As an example, is not scientifically justified for ARSWs' design the use of the many parameters adopted for traditional racks, that may lead in general to a strong reduction of seismic demand and to unsafe design [5].

Dealing specifically with seismic conditions, the lack of dedicated regulations for ARSWs is highlighted by Haque and Alam ([7], [8]) who proposed the a design procedure based on the direct displacement-based design for the Down-Aisle (DA) direction ("length" direction in Figure 1) and an evaluation of the over-strength and force-reduction factors. The gap created by the absence of a specific seismic design approach may result in unsafe design, and this has been proved by the recent collapses after seismic events. In this regard, Kondratenko et al [9] evaluated the structural performance of multi-depth shuttle warehouses under low-to-moderate seismic actions, pointing out the strong and weak points of current design for this structural type of ARSWs, how they affect the performance, and finally giving some proposal to improve it. Tsarpalis et al [10] proposed a simplified model to assess the seismic performance of ARSWs, individuated in [11] the macro-characteristics of steel racking systems influencing the vulnerability assessment and in [12] investigated the accuracy of some proposed models to consider the interaction of pallets with the rack structure. All of these studies mainly concern multi-depth shuttle warehouses, which are one of the two big structural types of ARSWs, while for double-depth warehouses not much can be found. The main difference among these is based on the available consecutive spots for pallets for each load level (in a large number the former and two in the latter, as in Figure 1), which implies different logistic solutions (e.g. devices for pallets handling) and different structural behaviour, since in the first case all the upright trusses are connected at least at each load level, while in the latter only the consecutive adjacent ones.

In this paper, 5 case studies designed by 5 big European companies that nowadays design and produce warehouses are analysed, focusing both on the influence of the design parameters on the seismic demand and on the evaluation of the resulting seismic performance. This study analyses more in dept the general results obtained in [5].

2. The case studies

A design problem is proposed to 5 big European companies, concerning the design of a double-depth warehouse starting from some fixed design input: (i) overall geometry (Figure 1); (ii) number of goods per load level, with relative weight and dimensions (Figure 1); (iii) static and seismic design force input; (iv) load combination factors and definition of participating mass. As regard (iii), a European high seismic zone with a Peak Ground Acceleration (PGA) equal to 0.30g for a return period equal to 475 years is selected for the seismic design. The importance class I is assumed with a design life equal to 50 years, according to [2]. About (iv): load combination factors are defined according with Eurocodes; the participating mass is defined according to Eq. (1) – (2), where the reduction factor R_F is equal to 1.0 for CA and 0.8 for DA direction, and the unit load modification factor E_{D2} is 0.8 (both the parameters are in line with [2]). In addition to the fixed design inputs, there are some free choice parameters that should highlight the current design trends as: structural types and corresponding behaviour factors; components characteristics (cross-section, steel grade and connection typologies); the number of pallets per beam pair (2 or 3). In the following, a brief description of the case studies is given, pointing out the main free-choice design parameters.



Height (H) x Width (W) x Length (L)	26.2 m × 14.5 m × 96.0 m
Number of upright trusses along Cross Aisle (CA) direction	8
Number of aisles	2
Warehouse storage capacity	10080 load units
Pallet type 1 - weight	1000 kg
Pallet type 2 - weight	800 kg
Pallet type 3 - weight	600 kg

Figure 1: Definition of some of the design input parameters for the case studies.

$$W_E = W_{E,G} + W_{E,Q} + W_{E,UL} \quad (1)$$

$$W_{E,UL} = R_F \cdot E_{D2} \cdot Q_{P,rated} \quad (2)$$

2.2. Descriptions of the main CS structural choices

From a global perspective, different structural configurations can be identified for CA (Figure 2) and DA directions of the Case Studies (CS). As regard CA frames, they all are constituted by repeated upright trusses, where diagonals are organized in different schemes: the CS1 (Figure 2a) is characterized by the K scheme; the CS2 (Figure 2b) has the V diagonals scheme; the others (CS3 – Figure 2c, CS3 – Figure 2d and CS4 - Figure 2e) have the D scheme. In each case, diagonals work both in tension and in compression, providing stiffness and resistance both for vertical and horizontal actions, and mixed design assumptions are made for the design (basically adopting [2] and [13] where specifics are not covered). The adjacent upright trusses can be reciprocally connected along their height, improving the stiffness of the structure along CA direction and affecting the distribution of the demand for horizontal actions. All the adopted configurations are allowed by the technical regulations for racks [2], but it's not always the same for Eurocode 8 [13]:

- K and Z structural types are designed in the elastic field, and if buckling is prevented, they may be also in line with Eurocode 8 prescriptions [13], as highlighted in [5];
- The D bracing type corresponds to a frame with K bracings (as defined by Eurocodes, §Figure 11.11 from [13]), which is not allowed from [13] to be used to design seismic resistant structure (§11.4.1 from [13]);
- The V scheme is allowed from Eurocode 8 prescriptions [13].

As regard DA direction, additional bracing systems are adopted in all the structures, with a tension-only X layout. This solution supplies more stiffness and resistance with respect to the one provided by the rack system (which is made of semi-rigid upright and pallet beam frames along DA). This choice is less peculiar and one of the traditional Eurocodes possible solutions for seismic-resistant structures, this is why DA direction is not be deeply analysed in this paper. What is only worthy to be highlighted is that these bracing systems may be placed in specifics spots along the length of the structures (and called "bracing towers" in this case), or diffused, and again, they can be in line with upright trusses or placed in an eccentric position (Figure 2b, 2d, 2e), implying for the external trusses an eccentricity of the centre of mass with respect to the centre of stiffness, and so, possible negligible rotational modes affecting the response of the structure to horizontal forces along DA direction.

As previously said, no global horizontal rigid plane can be found in double depths, given the necessity of leaving the aisles free for the movement of the devices for the handling of goods. This significantly affects the stiffness of the system along CA direction, as well as the distribution of forces due to horizontal actions.

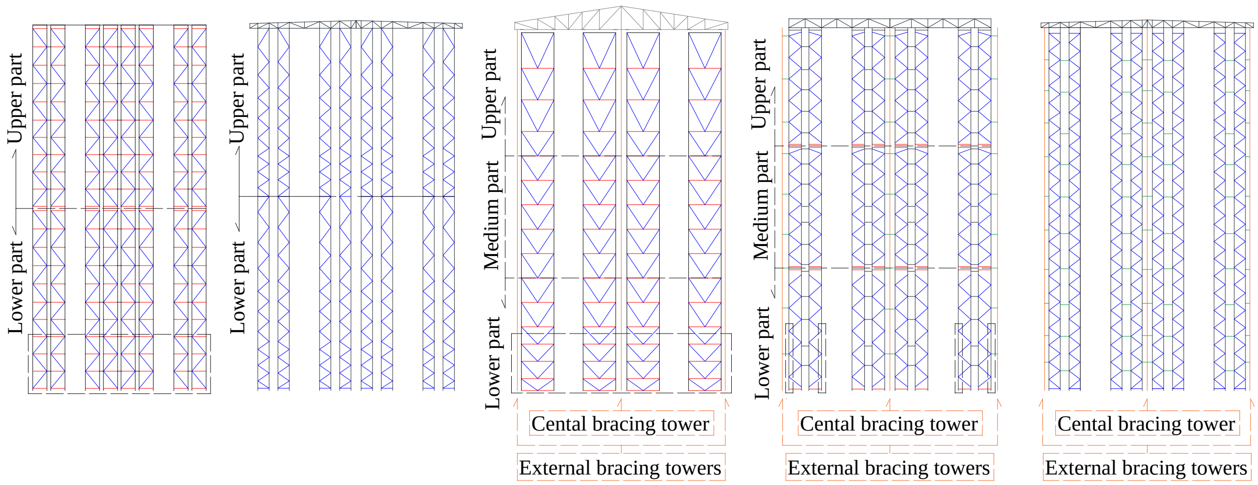
From a local perspective, similar profiles cross-sections and type of connections are adopted. As an example, the typical lipped U cross-section is used for uprights - which is derived from the traditional rack systems - with continuous perforation along the length. The U section allows a direct connection of the diagonals, with no use of additional sheets. Uprights may be also reinforced at the bottom by using an additional profile, which is connected through bolts or welds to the original one, limiting the use of too many different sections, the introduction of discontinuities, and supplying higher resistance where needed.

From the analysis of the structural choices, it seems that structural optimization is one of the fundamental goals of the design of ARSWs, in order to control the costs of the material by reducing those connected to additional workshop processes (as welds, auxiliary sheets for connections), and reduce the number of different profiles, implying a cost-effective production and with a fast and no-mistakes assembly phase.

2.3. Influence of the design parameters in the definition of the seismic demand

Table 1 highlights the free design parameters adopted for the design, which have an impact on the definition of the seismic demand and of the modal characterization of the structure. The relevant design steps are (i) the definition of the design response spectrum; (ii) the definition of the participant mass; (iii) the possible reduction of the lateral stiffness of the frames along CA direction.

As regards (i), this is obtained through the reduction of the elastic one by the behavior factor, whose value depends on the structural typology used, and by the adoption of the K_d factor [2]. In all the CSs, the value of 0.8 is assumed for the K_d, meaning that the weight of the pallets is preponderant with respect to the weight of the structure. A decrease of the seismic acceleration up to 46% corresponds to the assumption of K_d equal to 0.8 and q-factor equal to 1.5, while a decrease up to 60% can be reached adopting a q-factor equal to 2.0. Dealing with (ii), if both E_{D2} and RF factors are assumed, in line with prEN16681 [2], a reduction of the seismic mass up to 20% is reached along CA direction and up to 46% for DA direction. This assumption directly affects the total design base shear due to seismic action and since it is not possible to say a priori if these assumptions finally determine an increase or a decrease of the design seismic base share, the effects of this parameter are not considered in Table 1 considerations. Finally, the (iii) assumption can basically make the fundamental period of the structure (along CA direction) increase also up to 30-40%, being the structure more flexible, and determining a reduction of the seismic acceleration, shifting therefore the seismic design main concern from the Ultimate Limit States to the Damage Limit States. Looking at the total seismic base shear reduction within Table 1, it can be noticed that, in some cases, quite high reductions are reached (up to 60%), and could get even higher due to the reduction of the participant mass. It is necessary to evaluate if these assumptions, which are justified for traditional racks, are also suitable for ARSWs or if they actually lead to an unsafe and not conservative design.



(a) (b) (c) (d) (e)

Figure 2: CA view of the case studies.

Table 1: Design assumptions and their impact in the definition of seismic design base shear.

Case Study	Direction	Seismic acceleration			Mass		Total Seismic base shear reduction [%]
		q	K_d	Reduction of lateral stiffness	E_{D2}	RF	
1	CA	1,5	0,8	YES	1,00	1,00	**52**
	DA	1,5	0,8	NO	1,00	0,80	**46**
2	CA	1,5	0,8	YES	1,00	1,00	**52**
	DA	2,0	0,8	NO	1,00	0,80	**60**
3	CA	1,5	0,8	NO	0,80	1,00	**46**
	DA	2,0	0,8	NO	0,80	0,80	**60**
4	CA	1,5	0,8	NO	0,80	1,00	**46**
	DA	2,0	0,8	NO	0,80	0,80	**60**
5	CA	1,5	1,0	NO	0,80	1,00	**33**
	DA	1,5	1,0	NO	0,80	0,80	**33**

3. Seismic assessment

FEM numerical simulations and the evaluation of the safety level at Life Safety Limit State (LSLS) are carried out for the assessment of the seismic performance of the CSs, which results from the adopted design assumptions. Firstly, a modal analysis is performed for the modal characterization. Then, Non-Linear Time History Analyses (NLTHA) are executed using a set of 15 natural accelerograms as seismic input. These analyses are performed considering only geometrical non linearities. This choice is made because the designers, in line with [2], did not adopted any real hierarchy rules for the design of the components; thus, a very limited global ductility and a series of brittle or quasi-brittle behaviour (at least at the beginning) are expected to be observed. Finally, the safety checks of elements and connections are done, and the resulting Demand/Capacity ratios (D/C) are arranged from the highest to the lowest in order to point out the chain of failure mechanisms and the weakest parts of the structure. A visual representation of these chain of failure mechanisms is represented in the so called “hierarchy of criticalities” (Figure 4), which highlight the most critical elements. It should be pointed out that when a mechanism with relevant ductility is detected, the adopted model is no more representative of the actual behaviour of the structure, but it is far more than sufficient to observe the actual critical aspects of current design and evaluate possible improved design strategies (i.e. components to be strengthened, possible hierarchy rules, etc.).

The seismic input for the NLTHAs consists of a set of 15 natural accelerograms, which is selected from the NGA-West2 database [14] that matches the target conditional spectra [15]–[17] at a 2475 years return period (an exceedance probability of 2% in 50 years). Scale factors are then calculated to obtain other probability of exceedances [18], and for the purpose of the research, the 10% in 50 years has been adopted (Figure 3).

	2% / 50 Years	10% /50 Years
AvgSA [g]	0.4990	0.2410
Accelerogram	Scale Factors	
	SF (2%/50)	SF (10%/50)
1	2.3577	1.1387
2	4.1442	2.0015
3	5.7066	2.7561
4	2.3164	1.1188
5	9.7842	4.7255
6	3.6942	1.7842
7	7.9841	3.8561
8	5.0573	2.4425
9	4.3039	2.0787
10	5.5614	2.6860
11	0.8991	0.4342
12	4.0280	1.9454
13	2.5756	1.2439
14	5.4605	2.6372
15	9.6424	4.6570

Figure 3: Selected records corresponding to a probability of exceedance of 10% in 50 years, and corresponding scale factors.

All the numerical models were developed in OpenSEES [19]. Proper 2D or 3D models where adopted, according to the possibility to consider the behaviour in the two directions (CA and DA) independent one from the other. The elements are modelled as beams with a linear elastic material (elastic beam column element, [19]). The participant mass is defined accordingly to the CS (Table 1). The safety checks of the main components of the structures are carried out according to Eurocode 3 prescriptions [20] and EOTA technical documents for base connections, when post-installed anchors are used

[21]. The post-process of the huge quantity of data resulting from the analyses and the associated safety checks are handled through a tailored-developed MATLAB® code [22]. All the safety checks are re-arranged and represented in the previously introduced hierarchy of criticalities.

Among all the structures, a quite similar behaviour can be observed along the two directions. Figure 4 represents the hierarchy of criticality of the CA direction of CS1, which is selected as indicative of the behaviour observed for this direction in all the CSs. Some significative steps are gathered in Figure 4, where each step contains a specific range of D/Cs (referring to Figure 4, the first step gather the highest ones – from 1.59 to 1.55). For the CA direction, the highest D/Cs ratios are placed at the base of the structure, and then criticalities spread through the height of the structure. If structural optimization is made along the height of the structure, using for example less thick profiles, a better exploitation of the capacity of the components is reached, with a major number of elements along the height involved in the mechanism. In all the structures, diagonal connections and uprights base connections are those characterized by the highest D/Cs: and diagonal connections fail for bearing (due to the very low thickness of these elements), while base connections fail due to tensile and shear force on anchors (concrete cone mechanism + shear failure of anchors). It should be pointed out that upright base connections are among the first criticalities when post-installed anchors are used, while in case of using traditional solutions (as threaded bars), lower D/Cs are detected. As regard diagonal-to-upright connections, the failure of the diagonal element is always anticipated by the connection because of two reasons: the first one is that diagonals are cold-formed elements characterized by very low thicknesses, implying that it is not easy to have bearing resistance of connection major than the resistance of the element; secondly, no hierarchy rules are applied for the design of these components, with the only request of having the shear resistance of bolts at least 1.20 times the bearing resistance. This strategy surely allows to avoid a brittle failure of connection, but in any case, no over-resistance of connection with respect to the element can be guaranteed. After the failure of connections, buckling failure of uprights usually occur, starting from the base. The use of reinforcement at the bottom seams to help the delay of such criticality, ad is especially helpful to control local and global buckling phenomena, increasing the buckling resistance for axial compression of uprights. On the other hand, the section modulus of uprights is quite low due to the geometry of the U lipped sections, which results in poor capacity in bending. This implies that, despite bending force is quite low, its weight in the combined axial-force + bending check is significant and cannot be neglected.

Similar behaviour can be observed in the DA direction: the highest D/Cs start from the bottom and are placed in the bracing systems, involving diagonal to upright connections (bearing of the diagonal or of the plate connecting the diagonal to the upright) and upright base connections (concrete cone mechanism + shear failure of anchors). The consequent criticality occurs in the uprights belonging to the bracing towers, due to axial compression, while all the other components not belonging to the bracing systems are characterized by lower D/C ratios.

4. Conclusions

The current design strategies adopted for the design of ARSWs and resulting structural behaviour are critically analysed and assessed by means of 5 CSs structures, which have been designed by 5 big European companies that nowadays design, manufacture and build such structures. These structures are designed starting from common design inputs, which are given to make the solutions comparable, and setting other parameters free (i.e. structural types, profiles' material and cross-sections) to highlight the current trends in terms of global and local structural solutions and technical features. In this paper, the focus is made on seismic design conditions.

From the analysis of the free choice parameters and their impact on the definition of the seismic demand, it can be noticed that the use of different values and the various combination of them can lead to significantly different reduction amount of design seismic base shear (up to 54% for CA direction and to 69% for DA with respect to the elastic design value, Table 1). This confirm the outcomes of [5], where this matter was addressed by not referring to a specific ARSW structure. It should also be pointed out that, although the use of these parameters is validated for traditional racks, it is not for ARSWs, possibly leading to unsafe design.

The vulnerability assessment of the structure highlights that in both CA and DA directions the first criticalities are in connections with leading mechanisms which are brittle or quasi-brittle. Indeed, diagonal-to-upright connections fail for bearing, while upright base connections, when post-installed anchors are adopted, reach the ultimate concrete-cone plus shear resistances. The current design, which is not based on the adoption of hierarchy rules for behaviour factors up to 2 (in agreement with [2]) do not provide any kind of post-elastic sources, and this is deeply in contrast with

Eurocodes provisions, even is a low ductility is expected. This issue, added to the confusion in the use of design parameters not scientifically justified for ARSWs, highlight the urgency of the definition of a dedicated design approach for these structures, with an aware utilization of the design inputs, structural types and technical solutions, to which should correspond a safe and controlled structural behaviour.

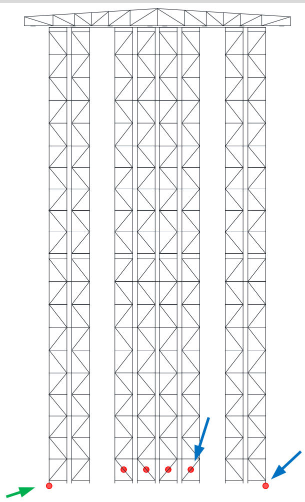
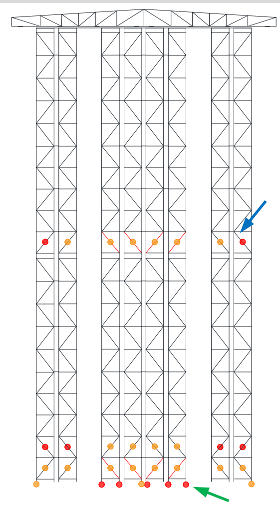
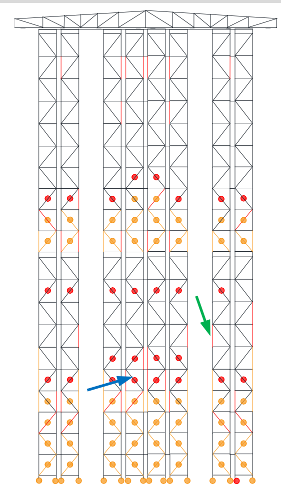
Key: ● failure in connection happening in the step; ● failure in connection already happened;		
▬ failure in element happening in the step; ▬ failure in element already happened;		
→ first mechanism happening in the step; → last mechanism happening in the step		
1st Step	**Intermediate Step**	**Final Step**
		
1.59 - 1.55	**1.42 - 1.33**	**1.08 – 1.00**
First mechanisms: failure of diagonal connection due to plastic ovalization + failure of base connection in post-installed anchors due to tensile+shear force.	First mechanism: failure of diagonal connection due to plastic ovalization.	First mechanism: failure of upright due to axial compressiona and bending (stability).
Last mechanism: failure of base connectors (post-installed anchors) due to tensile+shear force.	Last mechanism: failure of base connectors (post-installed anchors) due to tensile+shear force.	Last mechanism: failure of upright due to axial compression and bending (stability).
Note: only connections are involved.	Note: diagonal elements start to buckle (failure of connection already happened).	Note: failure of connections of diagonals spreads, as well as of diagonal elments and uprights. All the base connections reach their ultimate capacity.

Figure 4: Hierarchy of criticality of the CA direction of CS1.

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References

[1] «prEN 15512 "Steel static storage systems - Adjustable pallet racking systems - Principles for structural design"». 2018.

[2] «prEN 16681:2015 "Steel static storage systems - Adjustable pallet racking systems - Principles for seismic design"». 2015.

[3] European Commission. Directorate-General for Research, *Storage racks in seismic areas*. LU: Publications Office, 2009. Consultato: 20 novembre 2020. [In linea]. Available at: https://data.europa.eu/doi/10.2777/60886

[4] C. A. Castiglioni, A. Kanyilmaz, C. Bernuzzi, A. Drei, H. Degee, C. Braham, B. Hoffmeister, C. Heinemeyer, I. Vayas, A. Konstantinos, S. Sesana, B. Orsatti, J. Hermanek, J. W. Frederiks, T. Cudini, L. Rovere, D. Jehin, O. Kraus, D. Bakalbasis, *Seismic behaviour of steel storage pallet racking systems (SEISRACKS2): final report.* Luxembourg: Publications Office, 2014.
[5] Agnese Natali, Francesco Morelli, Walter Salvatore, «Influence of the design parameters on the current seismic design approach for Automated Rack Supported Warehouses», *ICSECT'22 - 7th International conference on Structural Engineering and Concrete Technology*.
[6] S. Caprili, F. Morelli, W. Salvatore, e A. Natali, «Design and Analysis of Automated Rack Supported Warehouses», *TOCIEJ*, vol. 12, n. 1, pagg. 150–166, giu. 2018, doi: 10.2174/1874149501812010150.
[7] A. B. M. Rafiqul Haque e M. S. Alam, «Direct Displacement-Based Design of Industrial Rack Clad Buildings», *Earthquake Spectra*, vol. 29, n. 4, pagg. 1311–1334, nov. 2013, doi: 10.1193/080611EQS195M.
[8] A. B. M. R. Haque e M. S. Alam, *Preliminary Investigation on the Overstrength and Force Reduction Factors for Industrial Rack Clad Buildings*. 2015.
[9] A. Kondratenko, A. Kanyilmaz, C. A. Castiglioni, F. Morelli, e M. Kohrangi, «Structural performance of automated multi-depth shuttle warehouses (AMSWs) under low-to-moderate seismic actions», *Bull Earthquake Eng*, ott. 2021, doi: 10.1007/s10518-021-01193-y.
[10] D. Tsarpalis, D. Vamvatsikos, I. Vayas, e F. Delladonna, «Simplified Modeling for the Seismic Performance Assessment of Automated Rack-Supported Warehouses», *Journal of Structural Engineering*, vol. 147, set. 2021, doi: 10.1061/(ASCE)ST.1943-541X.0003153.
[11] D. Tsarpalis, D. Vamvatsikos, F. Delladonna, M. Fabini, J. Hermanek, P. D. Morgotan, S. Sesana, E. Vantusso, I. Vayas, «Macro-characteristics and taxonomy of steel racking systems for seismic vulnerability assessment», *Bulletin of Earthquake Engineering*, feb. 2022, doi: 10.1007/s10518-022-01326-x.
[12] D. Tsarpalis, D. Vamvatsikos, e I. Vayas, «Seismic assessment approaches for mass-dominant sliding contents: The case of storage racks», *Earthquake Engineering & Structural Dynamics*, vol. n/a, n. n/a, doi: 10.1002/eqe.3592.
[13] «prEN 1998-1-2:2019.3 Eurocode 8: - Design of structures for earthquake resistance - Part 1-2: Rules for new buildings». 2019.
[14] Y. Bozorgnia, G. M. Atkinson, J. W. Baker, «NGA-West2 research project», *Earthquake Spectra*, gen. 2014.
[15] J. Baker, «Conditional Mean Spectrum: Tool for Ground-Motion Selection», *Journal of Structural Engineering*, vol. 137, pagg. 322–3311943, mar. 2011, doi: 10.1061/(ASCE)ST.1943-541X.0000215.
[16] T. Lin, C. B. Haselton, e J. W. Baker, «Conditional spectrum-based ground motion selection. Part I: Hazard consistency for risk-based assessments: CONDITIONAL SPECTRUM-BASED GROUND MOTION SELECTION-I», *Earthquake Engng Struct. Dyn.*, vol. 42, n. 12, pagg. 1847–1865, ott. 2013, doi: 10.1002/eqe.2301.
[17] T. Lin, C. B. Haselton, e J. W. Baker, «Conditional spectrum-based ground motion selection. Part II: Intensity-based assessments and evaluation of alternative target spectra: CONDITIONAL-SPECTRUM-BASED GROUND MOTION SELECTION-II», *Earthquake Engng Struct. Dyn.*, vol. 42, n. 12, pagg. 1867–1884, ott. 2013, doi: 10.1002/eqe.2303.
[18] Kohrangi M., Tsarpalis D., Vamvatsikos D., «Selection of seismic input for nonlinear analyses». Deliverable D.4.2. From Steelwar Research Project.
[19] Mazzoni, *The open system for earthquake engineering simulation (OpenSEES) user command-language manual*. 2017.
[20] «prEN 1993-1-3: 2019 Eurocode 3 - Design of steel structures - Part 1-3: General rules - Supplementary rules for cold-formed members and sheeting».
[21] *EOTA Technical Report TR029 'Design of Bonded Anchors'*. 2010.
[22] *MATLAB® version (R2019b). Natick, Massachusetts: The MathWorks Inc.*