On the Seismic Design and Behavior of Automated Rack Supported Warehouses

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ON THE SEISMIC DESIGN AND BEHAVIOR OF AUTOMATED RACK SUPPORTED WAREHOUSES

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Abstract

Automated Rack Supported Warehouses (ARSWs), consisting in huge steel buildings offering optimized storage solutions, have been facing a huge diffusion in the last decade, mainly due to the necessity of having bigger and more efficient places to stock and handle goods through automated systems. However, there is not a specific regulatory framework for them, and they are currently being designed adopting the approach used for traditional steel racks. Even if traditional steel racks and ARSWs have several common aspects, there are relevant differences that do not allow to adopt the same design approach, especially for seismic actions. With the aim of highlighting the factors and parameters currently influencing the seismic design and the behaviour of these constructions, and the consequent need of a different design approach, the present paper presents a critical analysis of the seismic design approach currently adopted by technicians and designers. Then, a set of 5 structures designed by 5 of the major European companies specialized in this field is used to perform both the critical analysis of the different design approaches adopted and the assessment of the seismic performance. The results of the seismic assessment highlight the main typical criticalities and the necessity of a novel proper design approach.

Keyword: Automated Rack Supported Warehouses, seismic design, steel racks, cold formed profiles.

1. Introduction

Automated Rack Supported Warehouses (ARSWs) are huge steel buildings offering optimized storage solutions when the number of pallets is relevant and the daily handling operations are numerous (Azadeh, De Koster, e Roy 2019). Indeed, the whole volume of the construction is dedicated to the pallet storage, except for small portions used for the transit of storage devices (i.e. shuttle or stacker-cranes), which are automatically controlled assuring the reduction of the number of damaged/lost items and of the functioning costs (Lerher, Ficko, e Palčič 2021). It is generally possible to classify the ARSWs into single-depth, double-depth, and multi-depth, based on the global and functional organization and on the number of continuative available spots for each load level, see Figure 1a. For all the structural types it's possible to individuate a Cross-Aisle (CA) and Down-Aisle (DA) directions; the former is perpendicular to the aisles and is in-plane with the upright trusses composing the racks, and the latter is parallel to the aisles and in-plane with pallet beams (Figure 2).

ARSWs constitute the direct upgrade of traditional pallet racks. In such buildings, racks have the double function of stoking areas and being the structure of the warehouse. Despite this upgrade of structural function, ARSWs racks acquired most of the traditional ones' structural features to follow the fast-evolving market request, but they were

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not supported by a specific regulatory framework. The difference between racks and structures is evident also from the definition of these terms given by Eurocode 0 («prEN 1990:2018 “Eurocode - Basis of structural and geotechnical design”», s.d.) and by EN 15512 («BS EN 15512:2020 Steel Static Storage Systems. Adjustable Pallet Racking Systems. Principles for Structural Design» 2020). The former at § 3.1.1.2 defines "structure" as "part of the construction works that provides stability, resistance, and rigidity against various actions". In contrast, the latter at § 0.1 defines "racking systems" as "load bearing structures for the storage of goods in warehouses". The confusion of these two systems and the adoption of the same guidelines defined for traditional steel racks to design ARSWs («BS EN 16681:2016 Steel Static Storage Systems. Adjustable Pallet Racking Systems. Principles for Seismic Design» 2016; «BS EN 15512:2020 Steel Static Storage Systems. Adjustable Pallet Racking Systems. Principles for Structural Design» 2020) led to the same structural choices and technical solutions that the last seismic events proved to be not always adequate (Haque e Alam 2015; Caprili et al. 2018).

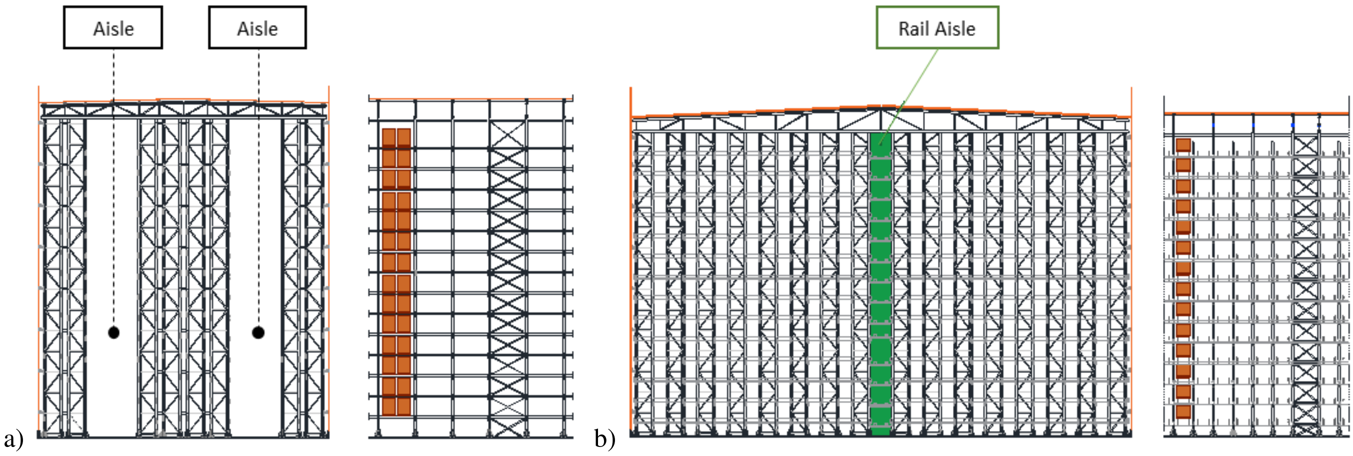


Figure 1: Typical Cross Aisle and Down Aisle views of (a) double-depth and (b) multi-depth warehouses.

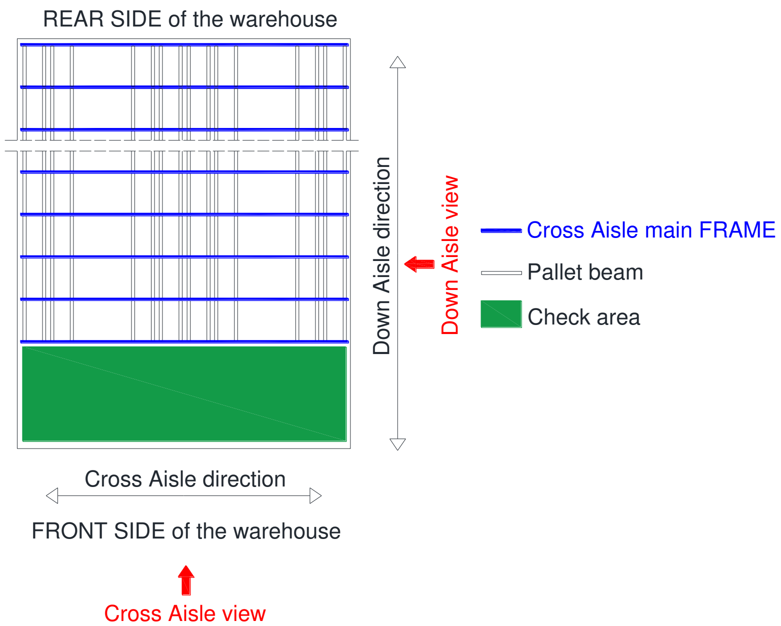


Figure 2: Plan view of an ARSW.

ARSWs are characterized, on one hand, by peculiar structural configurations that strongly influence the global behaviour and, on the other hand, by unique non-standard and basically *ad-hoc* made structural components and connections. Following this observation, the numerous researches carried out on steel racks, being them traditional or innovative as ARSWs', can be divided between those dealing with the analysis of the global behavior and those with the single elements and connections (as uprights, diagonal-to-upright connections, pallet beam-to-upright connections and upright base connections).

In the last decades, the global structural behaviour of traditional racks along both CA and DA directions has been investigated focusing especially in seismic response. In this framework, Baldassino et al (1999; 2000) analysed, tested and carried out a numerical study on the response of pallet racks commonly used in Europe. Within SEISRACKS and SEISRACKS2 research projects (European Commission. Directorate-General for Research 2009; Castiglioni et al. 2014; Kanyilmaz, Castiglioni, et al. 2016; Kanyilmaz, Brambilla, et al. 2016), experimental tests were carried out in both CA and DA direction, performing quasi-static monotonic, pseudo-dynamic and dynamic tests, aiming at pointing out the failure modes and defining q-factors to be used in seismic design. Degee et al (2011) conducted a parametric study comparing the various methods commonly used in practice for analysing the seismic structural behaviour of racks (i.e. modal response spectrum analysis and lateral force method analysis) as well as the different ways to account for geometrical nonlinear effects in these conventional methods of analysis in case of structures designed for low ductility. In the more recent literature, Tsarpalis et al (2021) investigated the influence of the pallets and their sliding in the seismic assessment of racking structures. Tsarpalis et al (2021; 2022) show first a simplified modelling method for the assessment of the seismic performance of ARSWs and then the main relevant characteristics of steel racking systems for seismic vulnerability analysis. Kondratenko et al (2021) evaluate the structural performance of ARSW multi depth structures under low-to-moderate seismic actions. These are among the very few contributions about the structural analysis of steel racks implemented in ARSWs.

Many recent researches are focused on the behaviour of components, as the ones of Becque e Rasmussen (2009b; 2009a), Schafer (2000), Gilbert et al (2012), Kanyilmaz et al (2016; 2016), stressing the high influence of their peculiarities on the global behaviour. As an instance, the typical uprights adopted in racks are usually realized with thin-walled sections and are often characterized by open mono-symmetric and lipped cross section, with continuous perforation along their height. These characteristics make the behaviour of this element different not only from hot-rolled sections but also from common structural cold-formed profiles used in light-weight constructions, especially in compression and bending (Kesti e Davies 1999; Godley e Beale 2008; Smith e Moen 2014; Dinis, Young, e Camotim 2014; Liu et al. 2021). Despite plenty of studies have been conducted, it seems that the awareness of the change of structural function of racks is missing. This aspect inevitably changes the structural design and opens to different construction and engineering issues, as highlighted by Haque and Alam (2015).

The present research mainly aims at providing an insight about the current technical regulations, design strategies and technical solutions adopted by the major designing and producing companies as well as about the seismic performance of nowadays ARSWs. To this end, the technical regulations and guidelines adopted in the current practice are first critically analysed and described, highlighting the inconsistencies mainly deriving from the use of the regulations originally developed for traditional racks. Then, the effects of adopting these regulations and guidelines in the practical design of ARSWs are highlighted by comparing the design solutions of the same structure carried out by five big European racks designing and producing companies. Finally, the seismic performance of these structures is assessed through numerical simulations, pointing out the main parameters that influence the structural behaviour, as well as the main structural criticalities, suggesting then the necessity of a befitting design approach for ARSWs.

2. Analysis of current regulations and standards for the seismic design

In current applications, the lack of specific regulations for the design of ARSW is covered by adopting the design strategies that are defined for traditional racks, often without proper theoretical and experimental evidence. EN 15512 (2020) and EN 16681 (2016) are the most used references respectively for the static and the seismic design. These documents are dedicated to traditional steel pallet racks but, as stated by both, they can also be taken as guidelines for the design of clad rack buildings when requirements are not covered in Eurocodes (§1). They take into account several aspects typical of racks and ARSWs that can be individuated in: i) the contribution to the global damping due to the possible sliding of the pallets; ii) the automated handling of goods and or the nature of their contents, which can strongly influence the seismic mass in the CA and DA directions; iii) the influence of the shear deformability of the upright frames, that can sensibly increase the fundamental period of vibration; iv) the design rules associated to behaviour factors.

The indications for the seismic design of racks included in EN 16681 (2016) that consider these aspects can be in deep contrast with the design principles included in Eurocodes and strongly influence both the seismic demand and the design approach for the ARWs. The following paragraphs analyse in detail these aspects.

2.2 Design parameters for seismic analysis: importance factor and definition of the design response spectrum

As provided by prEN 16618 (2015), the seismic design of racks can be performed using the modal response spectrum analysis and adopting a design spectrum $S_{d,mod}$ based on Eurocodes but reduced by a factor K_D equal to:

$$\boldsymbol{S_{d,mod}(T) = K_D \cdot S_d(T)} \quad \textbf{eq. I}$$

where $S_d(T)$ is the design spectrum as defined in Eurocodes for the associated important factor, considering a viscous damping factor ξ equal to 3%, and K_D is a factor that takes into consideration the participation of the pallets in the dissipation of the seismic energy and possible consecutive effects, when friction is overcome. In particular, K_D can be calculated as:

$$K_D = 1 - {}^{P_{E,prod}}\!/_{P_E} \cdot (1 - E_{D1} \cdot E_{D3}) \quad \textbf{eq. II}$$

where $P_{E,prod}$ and P_E are respectively the weight of the stored goods and the total weight of the structure (dead, permanent and goods weight), whereas E_{D1} and E_{D3} are the modification factors that specifically consider the dissipating phenomena where pallets are involved. Basically, the $P_{E,prod}$ over P_E ratio can be very close to 1, since the weight of the structure and of the permanent loads are a very low percentage of the total weight of the fully loaded rack. Indeed, when $P_{E,prod}$ is greater than or equal to 90% of P_E, K_D can be assumed equal to $E_{D1} \cdot E_{D3}$. E_{D1} is a parameter that depends on the force that can be transmitted by the unit loads to the beams (when pallets are fixed to the beams is equal to 1.0), and E_{D3} is introduced to consider the phenomena typical of the dynamic behaviour of these structures under seismic action, and that are not directly included in the formulation of the spectrum. E_{D3} is equal to 0.8, as resulting from the outcomes of Seisracks II (Castiglioni et al. 2014).

2.3 Design parameters for seismic analysis: definition of the participant mass

The seismic participant mass takes into consideration the quasi-permanent value of the permanent loads, $W_{E,G}$, and the variable loads, $W_{E,Q}$, together with the design seismic weight of the goods, $W_{E,UL}$, and itis expressed by:

$$\boldsymbol{W_E = W_{E,G} + W_{E,Q} + W_{E,UL}} \quad \textbf{eq. III}$$

$W_{E,UL}$ is strongly influenced by the characteristics of the pallets and stored goods, and on the direction of the seismic action. The resulting value can be determined as follows:

$$\boldsymbol{W_{E,UL} = R_F \cdot E_{D2} \cdot Q_{P,rated}} \quad \textbf{eq. IV}$$

where R_F is the rack filling grade reduction factor, that is related to the occupancy of the stored goods in the rack that can be assumed during the seismic event: it is assumed 1.0 in the CA direction, where it is more probable to have frames fully loaded, and values between 1.0 and 0.8 in the DA direction, considering the lower probability of full load conditions. E_{D2} is the unit load modification factor that represents the effects of the interaction between goods and racking structure: if the stored goods are compact, constrained or liquid, this factor is equal to 1.0, while, in the opposite condition, meaning that the content is loose and unconstrained, E_{D2} is equal to 0.7. Finally, $Q_{P,rated}$ is the specified value of the weight of unit loads, and this is given by the owner of the warehouse, basing on the foreseen goods flows.

2.4 Other relevant parameters: shear flexibility of upright frames along CA direction

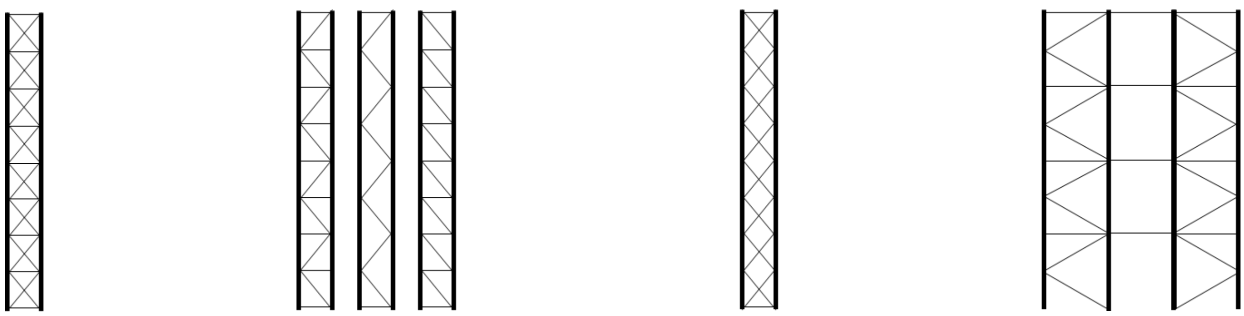
The typical connection between uprights and diagonals along CA direction is realized by directly connecting the diagonal to the column through a bolt. This solution implies a reduction of the stiffness of the structure along the CA direction and affects the dynamic response of the system. Given the many parameters that influence the value of the lateral stiffness of these structures (e.g. the eccentricity of connection with respect to the centreline of the upright, the looseness and the deformability of connection), it's not easy to analytically determine it. Different codes for steel racks (RMI (1997), FEM (2000)) provide Timoshenko's formula to evaluate the stiffness of built-up columns, that some researches (Sajja, Beale, e Godley 2008; Gilbert et al. 2012; Talebian et al. 2019) highlighted to be not appropriate for racks, causing a relevant over-estimation of the stiffness of these systems. The use of this formula could assure a higher safety for the design in terms of resistance, but lateral deformations could be under-estimated, and deformability limits may not be respected, affecting the serviceability states and the global stability of the structure. Within EN 15512 (2020), the use of Timoshenko's formula is suggested to be replaced by the execution of experimental tests on the upright truss. The procedure for these tests is given by this standard.

2.5 Possible approaches for dissipative design and behaviour factors

Besides elastic design, earthquake resistant racks can be designed also adopting dissipative approaches (§ 8.1 EN 16681 (2016)). In particular, low dissipative and dissipative structural behaviour concepts can be assumed: in the former, the effects of seismic action are calculated by means of elastic global analysis without considering non-linear material behaviour, while, in the latter, plastic deformations are expected in particular elements of the structure (the dissipative ones), and their capacity in dissipation is considered in the design phase through the behaviour factor q. In case of adopting the low dissipative concept, q-factor up to 2 can be used, while in the other case major than 2. It is useful to remember that this definition of “low dissipative” and “dissipative” is in contrast to the definition of the Eurocode 8, where behaviour factors up to 1.5 are associated to low dissipative structures.

2.6 Structural types, behaviour factors and design rules

In general, it is allowed to use moment resisting frames and braced frames to design low dissipative racks ($1.5 \leq q \leq 2$). The former is mostly used for DA direction. Referring to the CA one, the upright trusses that constitute steel racks are designed to resist both vertical and horizontal actions, and many layouts for diagonals are allowed: the X-one with tension-only diagonals (Figure 3a); the K, D, Z bracings (Figure 3b); the X bracings without horizontal members, in which the resistance to horizontal actions is provided also by diagonals in compression (Figure 3c). In case of adopting the tension-only X bracings, a behaviour factor of 2 is suggested, and no capacity design rules are mandatory to be applied in the design of the components (neither in the dissipative elements, nor in their connections, nor in the other non-dissipative components), and this is in deep contrast with Eurocode 8 prescriptions (2019). In case of adopting one among K, D and Z configurations, a behaviour factor equal to 1.5 is suggested, and no capacity design rules are mandatory to be applied in the design of the components, with only few exceptions (e.g. amplify the design effect due to seismic action by 1.5 for selected components). In any case, no safety factors shall be adopted for the design of the other non-dissipative elements and of their connections. The K and Z structural types are basically designed in the elastic field, and if buckling is prevented, they may be also in line with Eurocode 8 design prescriptions for such systems (2019). On the contrary, the D one can be associated to a system with K bracings (as defined by Eurocodes, Figure 3d), which is not allowed by Eurocodes (2019) to be adopted for seismic-resistant structures (§11.4.1). Finally, dealing with the X bracings without horizontal members, the EN 16681 (2016) provide to use a behaviour factor of to 1.5 with no adoption of capacity design, with the only exception of diagonal elements to be designed by amplifying the design seismic force by 1.5 if their buckling mode in axial compression is local. Anyway, this structural type can be associated to the K bracings one as defined by Eurocodes (Figure 3d), which is not allowed to be adopted according to them, as previously highlighted.
For the design of dissipative racks ($q > 2$), Eurocodes prescriptions have to be adopted, basing on the chosen structural type («prEN 1998-1-2:2019.3 Eurocode 8: - Design of structures for earthquake resistance - Part 1-2: Rules for new buildings» 2019).



a) X tension-only bracings b) K, D, Z bracings c) X tension-compression bracings d) K bracings (steel buildings)

Figure 3: Structural types for racks according to EN16681 (2016) (from a to c), and K bracings type for steel structures according to Eurocode 8 (2019) (d).

2.7 Influence on the final seismic design approach

The possibility of adopting a different-from-Eurocodes design approach can lead to an evident reduction of the seismic demand for ARSWs, which is in most parts not very well justified and demonstrated. Depending on the combinations of design hypotheses, and therefore of the key design parameters previously introduced, different reductions of the spectral accelerations can be obtained, whose value varies from case to case. However, to give an idea of the order of magnitude of the reduction that it is possible to achieve, it is assumed a reference configuration as a starting point to assess the maximum reduction of the seismic action. The reference configuration is characterized by:

- a design life of 50 years and an importance factor of 0.8.
- an European high seismic zone with a Peak Ground Acceleration (PGA) equal to 0.30g for a return period equal to 475 years;
- a period of 2.10 s associated to the CA frame, and of 1.50 for the DA.

Starting from this reference configuration, Table 1 gives the design hypotheses (DH) that induce the most sensitive reduction for the seismic action in terms of base shear for both the CA and DA directions (DH "Reduction"). Figure 4 shows the variation in terms of spectral acceleration. The reduction of stiffness along CA direction is not included in this comparison, since it depends on many parameters (profiles and connections geometric characteristics) and cannot be standardized, but supported by experimental evidence.

The maximum reduction of the base shear associated to the seismic action is quite sizeable, up to 54% for CA direction and to 69% for DA. Even if these values are obtained for a specific configuration, they give the idea of the influence of these parameters on the seismic design of the ARSWs. It is worth highlighting that the choice of different parameters in the range used for the above example, does not lead to different relevant design constraints.

The real issue is that the possibility of using most these parameters is experimentally and scientifically justified for traditional racks, but not for ARSWs. Furthermore, a deep inconsistency among Eurocodes and EN 16681 (2015) (when EN 16681 are used to design ARSWs) is in the possibility of using a behavior factor up to 2 without adopting properly capacity design rules. This comparison points out the confusion created by the absence of a specific standard, which may result in an unsafe design.

Table 1: Possible combinations of key design parameters and resulting acceleration and base shear for the defined elastic response spectrum.

M = participant Mass; T = period; S_d = design seismic acceleration; q = behavior factor

	Design Hypothesis	**Participant mass**					**Design response spectrum**			**Total ΔS_d [%] (*)**	**Total variation of design base shear [%] (*)**
		R_F	E_{D2}	**ΔM** [%] (*)	**ΔT** [%] (*)	**ΔS_d** [%] (*1)	q	k_d	**ΔS_d** [%] (*2)		
CA direction	Reference	1.0	1.0	**-**	**-**	**-**	1.0	1.0	**-**	**-**	**-**
	Reduction	1.0	0.8	**-20**	**-11**	**+9**	1.5	0.8	**-47**	**-42**	**-54**
DA direction	Reference	1.0	1.0	**-**	**-**	**-**	1.0	1.0	**-**	**-**	**-**
	Reduction	0.8	0.8	**-36**	**-20**	**+20**	2.0	0.8	**-60**	**-52**	**-69**

(*) Calculated for each direction with respect to the respective reference design hypothesis.

[1] Due to the variation of T only.

[2] Due to the variation of q and k_d only.

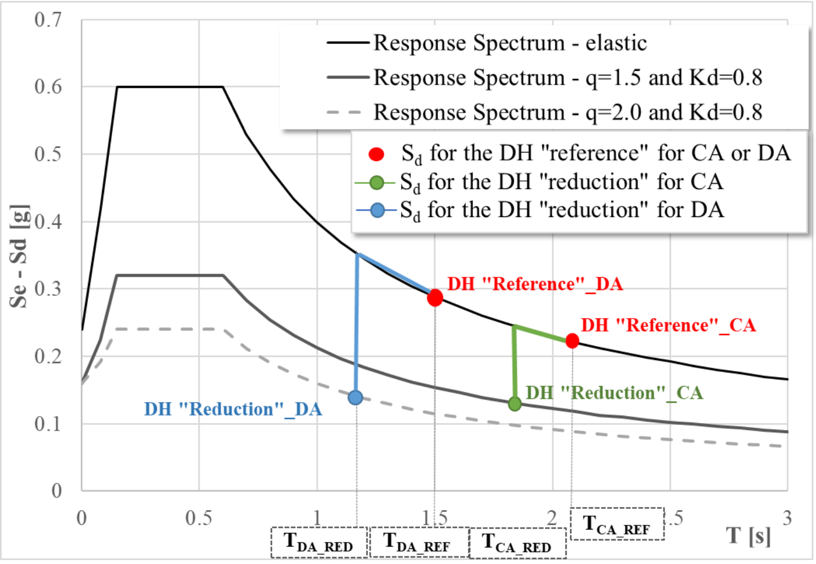
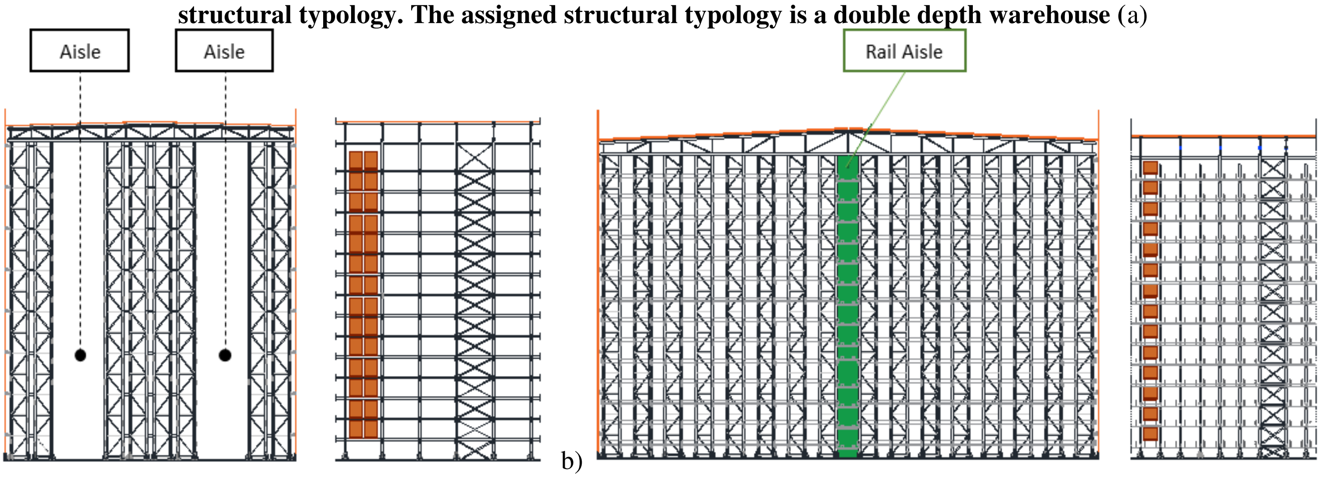


Figure 4: Maximum **e**ffect of the key parameters on the period and seismic acceleration for the CA and DA DHs.

3. Current design approach

To understand to which extent the design choices actually influence the final design, a common design problem was proposed to 5 of the biggest Europeans companies specialized in designing and manufacturing of steel racks and ARSWs and the resulting solutions were compared in terms of both structural choices and seismic performance. The design had the following common input design parameters:

structural typology. The assigned structural typology is a double depth warehouse (a)



(i) Figure 1a);

(ii) input geometry parameters, see Table 2 and Figure 5;

(iii) number of pallets per load level and relative characteristic (weight and dimensions);

(iv) design force input, both in static and seismic conditions;

(v) load combination factors and definition of participating mass.

Table 2: Main geometrical dimensions of the case study structure.

Height (H) x Width (W) x Length (L)	26.2 m × 14.5 m × 96.0 m
Number of upright trusses along CA direction	8
Number of aisles	2
Warehouse storage capacity	10080 load units

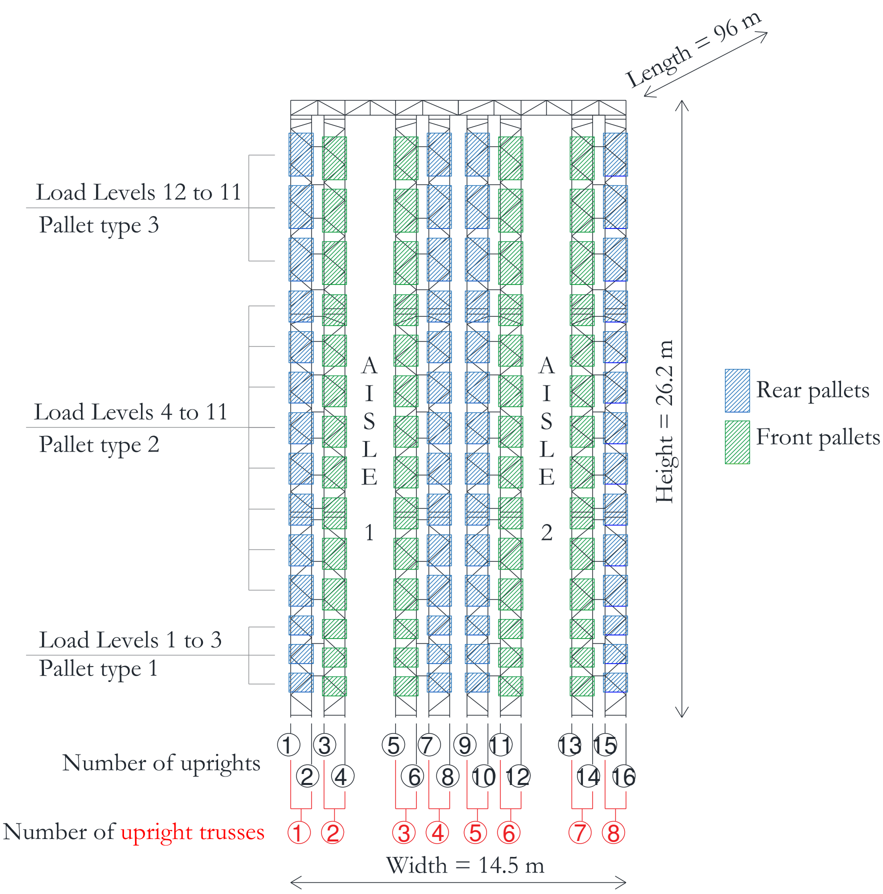


Figure 5: CA frame main characteristics.

The seismic design is carried out for a European high seismic zone with a Peak Ground Acceleration (PGA) equal to 0.30g for a return period equal to 475 years. Figure 6 represents the horizontal acceleration elastic response spectrum assumed, the two corresponding design response spectra corresponding to behaviour factors equal to 1.5 and 2. The importance class I is assumed, with a design life equal to 50 years, according to EN 16681 (2016).

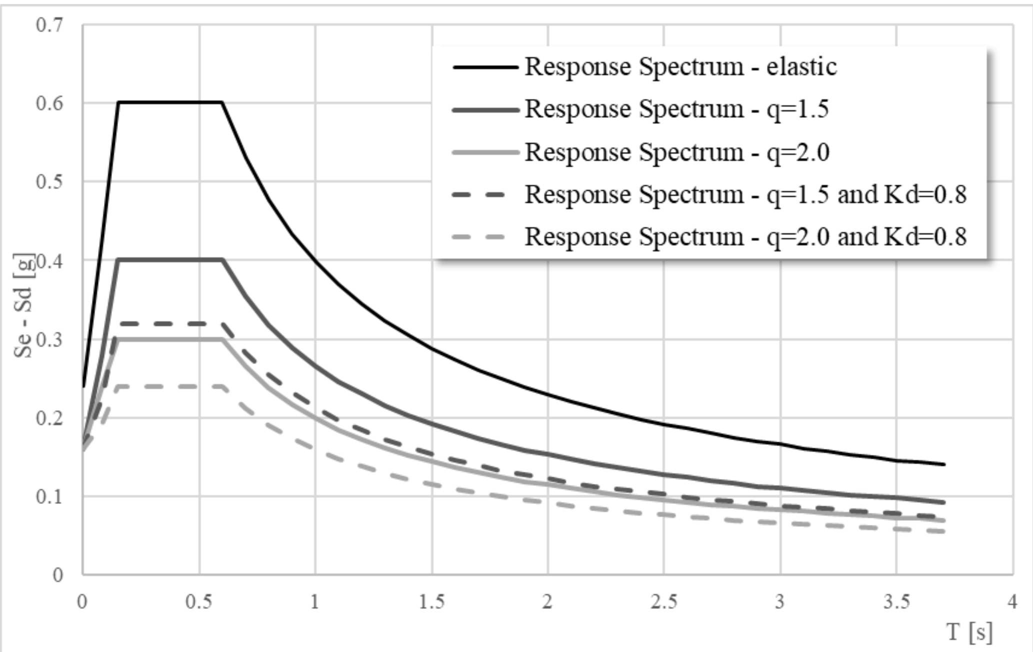


Figure 6: Elastic and design acceleration response spectra.

Load combinations and relative factors have also been fixed for static and seismic design, as well as material factors to calculate design capacity forces, and defined in agreement with Eurocodes. Regarding the design in seismic conditions, the combination of vertical loads to be used for the definition of seismic mass has been defined as in §2.3, where the RF is taken equal to 1.0 and 0.8 respectively for CA and DA direction. The Ψ_2 factor is equal to 0.8 (it replaces the E_{D2} parameter). Besides the fixed design inputs, several parameters have been set free to be chosen by the designers to highlight the current trends:

(i) structural types and corresponding behaviour factors.

(ii) components characteristics, as cross-section shapes, steel grade and connection typologies

(iii) the number of pallets per beam pair (2 or 3).

Finite Element Modelling (FEM) is adopted to perform numerical analyses of the structures. In the following, the Case Studies (CS), numbered from 1 to 5, are described in the following paragraphs. The main characteristics of the structural elements and the design assumptions are given respectively inTable 3 and Table 4. Due to the sensitivity of some information, only the general characteristics of the structural elements are provided.

3.1 Case Study 1

The structure is composed of 46 CA frames, among which the standard and non-standard ones can be individuated. The latter are those in correspondence of bracing towers, which are the horizontal forces resisting system for DA direction and are characterized by different uprights in steel quality and sections (Figure 7b). Structural optimization is made along the height of the structure (lower, upper part, Figure 7a). The upright trusses are organized in the K layout (tension-compression diagonals and horizontal elements), and the adjacent ones are connected through hinged elements called "spacers" in jargon. Along DA direction, each frame is constituted by uprights, pallet beams and bracing towers. The X-shaped tension-only horizontal forces resisting systems (bracing towers) are placed at the beginning and at the middle length of the frame (Figure 7b).

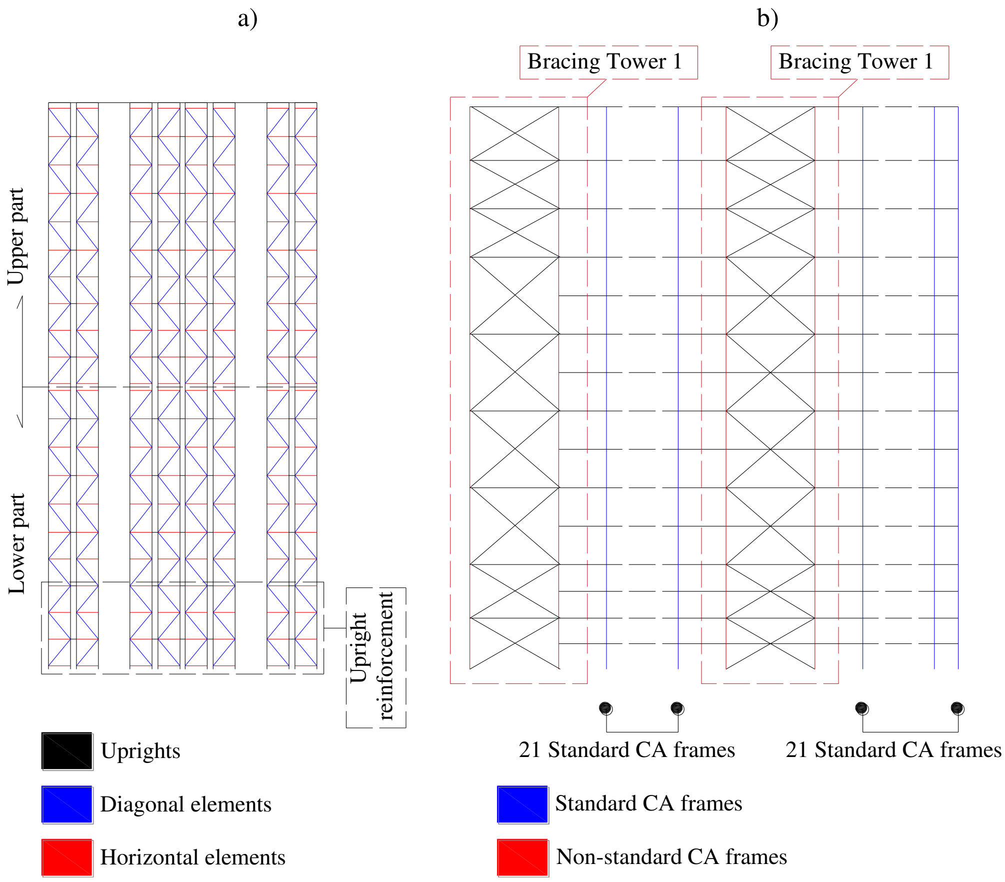


Figure 7: CS1 CA and DA views, where the main components are highlighted: a) structural elements belonging to CA frames, b) layout and organization along DA direction.

3.2 Case Study 2

The structure is composed of 50 CA frames, among which the standard and non-standard ones can be individuated. Structural optimization is made along the height of the structure (lower, upper part, Figure 8a). Each frame is composed of 8 upright trusses, whose diagonals are arranged in a D layout (tension-compression diagonals, no horizontal beams), and are not reciprocally connected. Along DA direction, each frame is constituted by uprights, pallet beams and 4 bracing towers (Figure 8b). Table 3 gathers the cross-sections and the relevant characteristics of the main structural elements and their connections. Table 4 gathers the definition of the free design parameters for CS2.

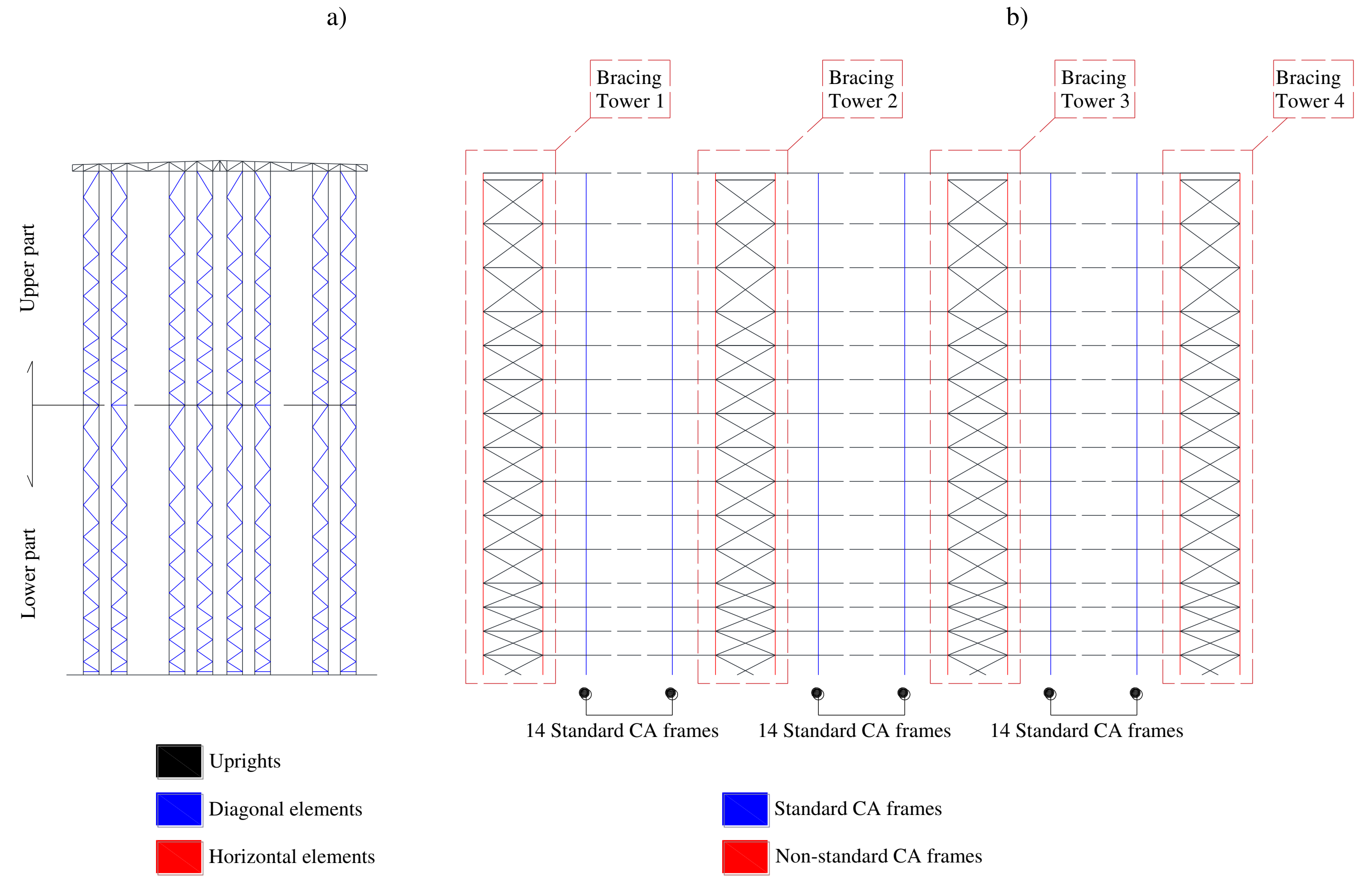


Figure 8: CS2 DA and CA views, where the main components are highlighted: a) structural elements belonging to CA frames, b) layout and organization along DA direction.

3.3 *Case Study 3*

The structure is made of a total of 33 CA frames, with 4 bracing towers along DA direction (Figure 9). There are 8 uprights trusses in the CA frames, whose diagonals are organized in a V layout (tension-compression diagonals, Figure 9a). Along DA direction, bracing towers constitute the horizontal resisting system, where diagonals are arranged in the X layout (tension-only, Figure 9b). The bracing towers are placed in an eccentric position with respect to uprights (Figure 9a). The connection between the bracing towers and the respective trusses is assured at each load level by both horizontal bracing system and transversal beams. Structural optimization is made along the height of the structure (lower, medium, and upper part, Figure 9a). Along DA direction, the CA upright trusses are connected through the pallet beams. All the uprights' base connections are fixed to the concrete slab with threaded bars.

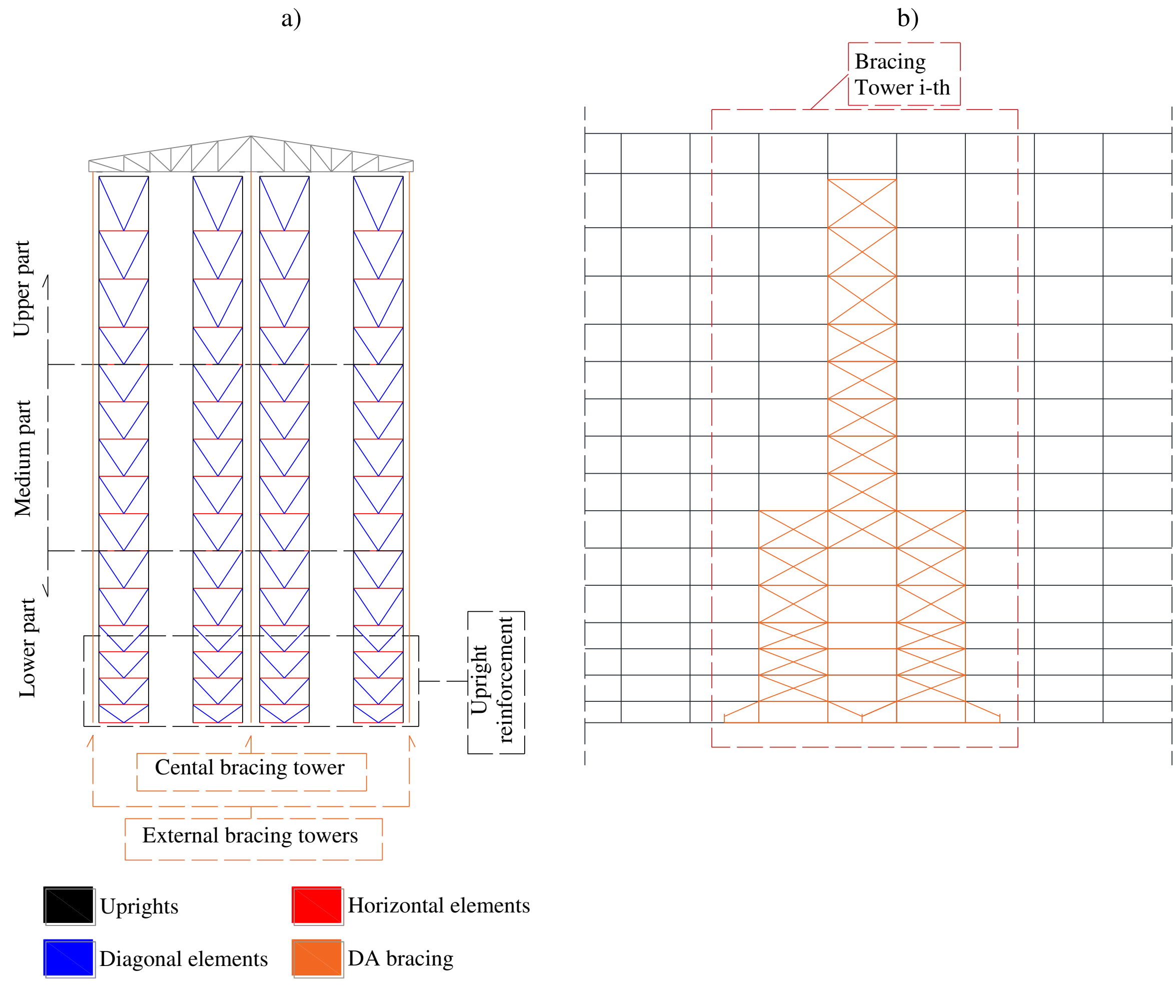


Figure 9: CS3 DA and CA views, where the main components are highlighted: a) structural elements belonging to CA frames, b) representative module for DA direction.

3.4 Case Study 4

The structure is made of a total of 33 CA frames, that are connected along DA direction through the pallet beams. Each CA frame is composed of eight upright trusses, where diagonals are organized in a D layout (tension-compression diagonals, Figure 10a). The adjacent upright trusses are connected through fixed spacers. Structural optimization is made along the height of the structure (lower, medium and upper part, Figure 10a). Along DA direction, the rectangular-sectioned cold-formed bracings are diffused all along the length of the structure and are arranged in the X-shaped tension-only layout (Figure 10b). The horizontal forces resisting systems are placed in an eccentric position with respect to the upright trusses (Figure 10a).

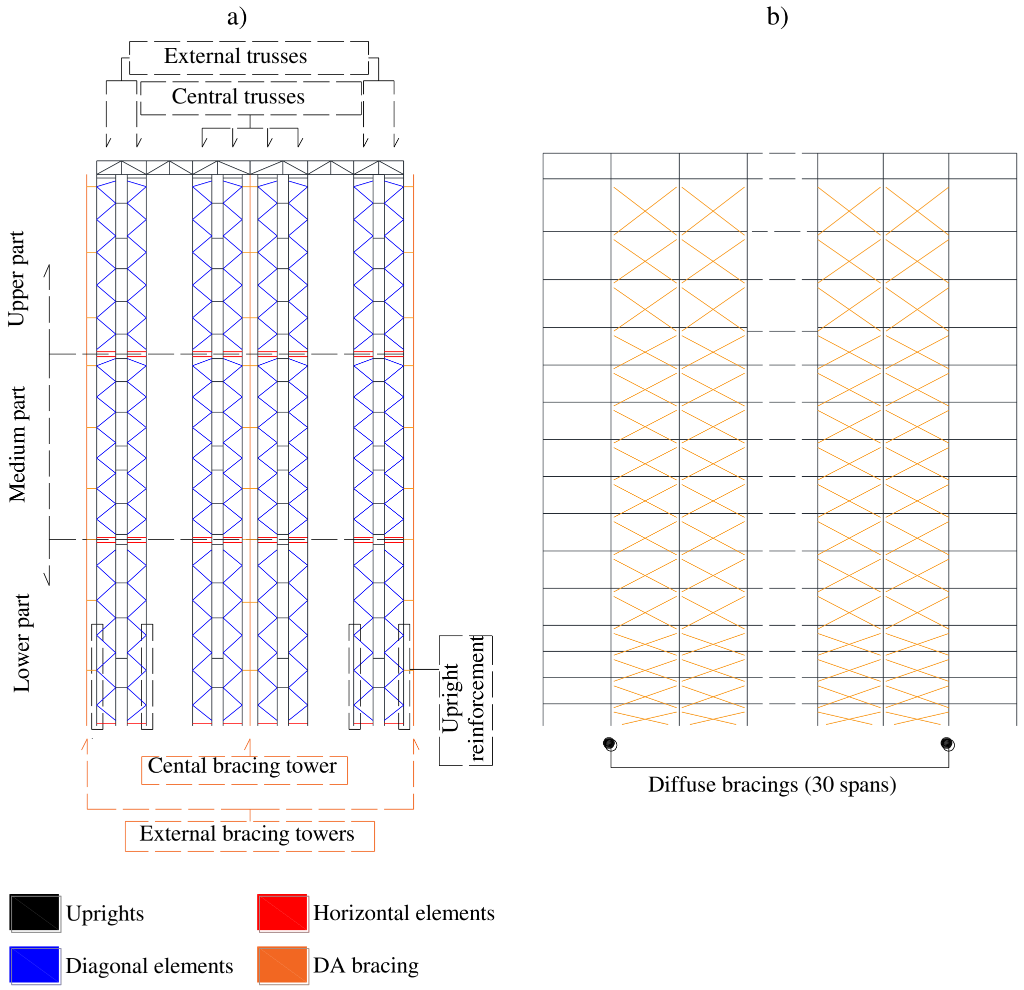


Figure 10: CS4 DA and CA views, where the main components are highlighted: a) structural elements belonging to CA frames, b) layout and organization along DA direction.

3.5 Case Study 5

The structure is made of 33 CA frames which are mutually connected through the pallet beams. Each CA frame has 8 upright trusses, where diagonals are organized in a D layout (tension-compression diagonals, Figure 11a). The adjacent upright trusses are connected through fixed spacers. Different thicknesses are used for the external and central shelves for structural optimization. Along DA direction there are 6 bracing towers (Figure 11 gives a representation of a representative module). Within each bracing tower, diagonals are arranged in the X layout (tension-only diagonals). The horizontal resisting systems are placed in an eccentric position with respect to the upright shelves (Figure 11a).

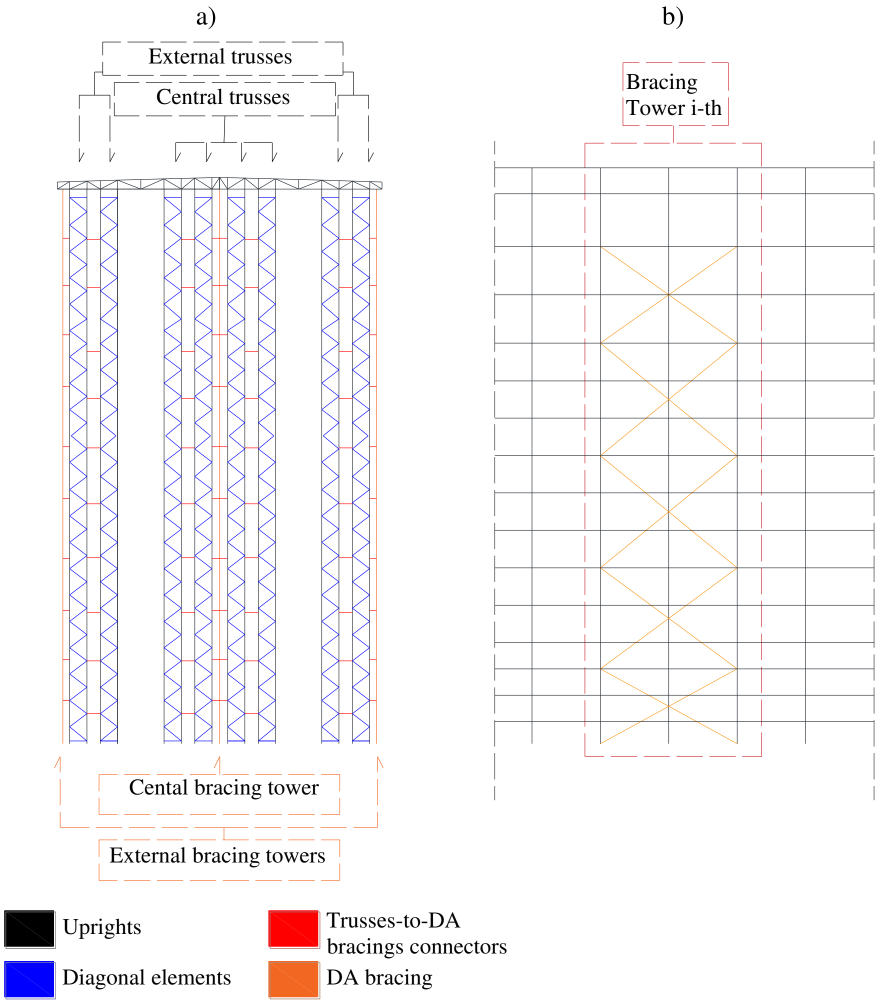


Figure 11: CS5 DA and CA views, where the main components are highlighted: a) structural elements belonging to CA frames, b) representative module for DA direction.

Table 3: Cross sections and characteristics of the main structural elements.

			CS1	CS2	CS3	CS4	CS5
CA	Upright	Cross-section	U lipped, reinforced at base Cold-formed	U lipped Cold-formed	U lipped, reinforced at base Cold-formed	U lipped, doubled at base Cold-formed	U lipped, reinforced at base Cold-formed
		Material	S350GD	S355MC	S420MC	S500MC	S500MC
		Base connection	Post-installed anchors		Threaded bars	Post-installed anchors	
		Continuous perforation	✓	✓	✗	✓	✓
	Diagonals	Cross-section	C section. Cold-formed	U section Cold-formed	C section Cold-formed	C section Cold-formed	Rectangular Cold-formed
		Material	S350GD	S350GD	S350GD	S355MC	S220GD and S235JR
		Connection	bolted to upright - hinged				
	Horizontal elements	Cross-section	C section Cold-formed	Not used	2xC section Cold-formed	Not used	Not used
		Material	S350GD		S350GD		
		Connection	bolted to upright - hinged		bolted to upright - hinged		
	Spacers	Cross-section	U section Cold-formed	Not used	Not used (not necessary for the adopted configuration)	Rectangular Cold-formed	Rectangular Cold-formed
		Material	S350GD			S355MC	S235JR
		Connection	Bolted to upright - hinged			Bolted to upright - fixed	Bolted to upright - fixed
DA – bracing system	Type	BT = Bracing Tower BTE = Bracing Tower with Eccentricity D = Diffuse	BT	BT	BTE	D	BTE
	Uprights	Cross-section	HE Hot-rolled	U lipped Cold-formed	2xC section Cold-formed	U lipped Cold-formed	Not used
		Material	S275JR	S355MC	S350GD	S500MC	
		Base connection	Precast anchor system with threaded bars	Post-installed anchors	Connected to a rigid beam - hinged	Post-installed anchors	
	Diagonals	Cross-section	L Hot-rolled	L Cold-formed	2xC section Cold-formed	Rectangular Cold-formed	Rectangular Cold-formed
		Material	S275JR	S355JR	S350GD	S275JR	S275JR
		Connection	bolted connection - hinged	bolted connection - hinged	bolted connection - hinged	bolted connection - hinged	bolted connection - hinged
DA	Pallet beams	Cross-section	Rectangular Cold-formed	E section Cold-formed	2xC section Cold-formed	Rectangular Cold-formed	Rectangular Cold-formed
		Material	Cold-formed	Cold-formed	Cold-formed	Cold-formed	Cold-formed
		Connection	Hooked – semi-rigid	Bolted to upright - hinged	Bolted to upright - hinged	Hooked – semi-rigid	Hooked – semi-rigid

Table 4: Definition of free design parameters for all the CSs (D=Diagonals).

		CS1	CS2	CS3	CS4	CS5
Horizontal resisting systems	**CA**	K layout (tension-compression D)	D layout (tension-compression D)	V layout (tension-compression D)	D layout (tension-compression D)	D layout (tension-compression D)
	DA	X layout (tension-only D)	X layout (tension-only D)	X layout (tension-only D)	X layout (tension-only D)	X layout (tension-only D)
Number of pallets for each couple of beams		2	2	3	3	3
Mass definition	**CA**	$G_1 + G_2 + Q_1$	$G_1 + G_2 + Q_1$	$G_1 + G_2 + 0.8 \cdot Q_1$	$G_1 + G_2 + 0.8 \cdot Q_1$	$G_1 + G_2 + 0.8 \cdot Q_1$
	DA	$G_1 + G_2 + 0.8 \cdot Q_1$	$G_1 + G_2 + 0.8 \cdot Q_1$	$G_1 + G_2 + 0.8 \cdot 0.8 \cdot Q_1$	$G_1 + G_2 + 0.8 \cdot 0.8 \cdot Q_1$	$G_1 + G_2 + 0.8 \cdot 0.8 \cdot Q_1$
Definition of design response spectrum	q **CA**	1.5	1.5	1.5	1.5	1.5
	q **DA**	1.5	2.0	2.0	2.0	1.5
	K_d	0.8	0.8	0.8	0.8	Not applied
Reduction of stiffness along CA direction		axial springs	axial springs	Not applied	Not applied	Not applied
Modelling strategy		3D full model	3D full model	2D for CA 3D reduced for DA	3D full model	3D full model

3.6 Comparison among the design solutions

The analysis of the configurations, of the structural choices, and of the design strategies adopted for the 5 case studies highlights that there are some common paths and distinguishing features that identify the current seismic design approach for double depth ARSWs.

From the global point of view, different configurations can be identified along CA and DA directions: the CA frames are constituted by repeated modular upright trusses, whose diagonals are arranged in different layouts (K, V, D schemes), corresponding to different structural behavior. These configurations allow the trusses to resist both to vertical and horizontal loads. As highlighted in §2.6, the K layout can also be assumed according to Eurocodes for seismic-resistant structures, if the building is designed in the elastic field, or with low dissipation concept (q-factor up to 1.5). On the other hand, the use of the D layout – which is comparable to the K layout as defined by Eurocode 8 (2019) (Figure 3d) - is not allowed, due to the bad demand on columns when one of the two diagonals buckles. The V layout can be adopted according to Eurocodes, in addition to observe the hierarchy rules for the design of components for major than 1.5 q-factors. On the other hand, EN 16681 (2016) allows to not adopt hierarchy rules for q-factors up to two. This is indeed in deep contrast with Eurocodes.

The consecutive adjacent upright trusses may be reciprocally connected through transversal elements (spacers) that can be hinged or fixed to the uprights (in case of using more rigid connection, the adjacent trusses are coupled, significantly affecting the stiffness of the structure along CA direction, as well as the distribution of forces in the uprights in case of horizontal forces acting). Looking at Table 6, the use of hinged and flexible spacers (CS1,

T_{main_CA}=2.61) implies similar effects of not adopting them (CS2, main period T_{main_CA} is 2.70) in terms of stiffness and load distributions (especially on uprights) along CA direction. In the same way, the use of fixed and more rigid spacers (CS4 and CS5) makes the structures more rigid (the main periods are respectively 0.96 and 1.10).

The CA frames are repeated and connected by the pallet beams along DA direction. The DA horizontal forces resisting systems can be diffused along the length of the structure (CS4, Figure 10b) or located in strategic positions (in this case called "bracing towers", CS1 - Figure 7b, CS2 - Figure 8b, CS3 - Figure 9b, and CS5 - Figure 11b), and the only structural scheme adopted for them within the analyzed case studies is the X one (tension-only diagonals). The bracing system can be aligned with the upright trusses (CS1 - Figure 7a, and CS2 - Figure 8a) or placed in an eccentric position (CS3 - Figure 9a, CS4, Figure 10a, and CS5 - Figure 11a). In this second configuration, the connection of the bracing system to the respective shelves is made through rigid transversal elements, that are usually placed in correspondence with load levels and connect all the uprights of the trusses. This kind of solution implies, for the external shelves, an eccentricity of the center of mass with respect to the center of stiffness, and so, there may be not negligible rotational modes in addition to the translational ones that can influence the response of the structure to horizontal forces along DA direction.

The horizontal bracing system is placed in line with the load levels, directly connected to pallet beams or to uprights. Being necessary to leave the aisles free, no global rigid plane can be found in these structures: each group of upright trusses which are separated by the aisles can be considered almost independent from the other ones. The only connection that involves all the upright trusses is the one at the top of the structure (constituted of the roof truss), that in any case in very far from the base of the uprights.

From the local point of view, the solutions proposed by the 5 different companies share similar choices in terms of main profiles cross-sections and type of connections (Table 3). All the uprights are characterized by a lipped U cross-section and are perforated along their length (only one CS out of the 5 considered has uprights with holes only where needed, in correspondence with connections with diagonals and pallet beams). The lipped U section, which is opened in the inner side, allows faster and easy connection of diagonals, that are directly bolted to the uprights in correspondence with the lips without using additional sheets (Figure 12). The diagonals used for the CA frames are characterized by C, U or rectangular sections. Besides, uprights are often locally reinforced at the bottom, where forces are higher, through an additional profile that is welded or bolted to the original column. This solution allows to limit the number of different cross-sections for uprights, without introducing discontinuities but providing higher resistance where needed.

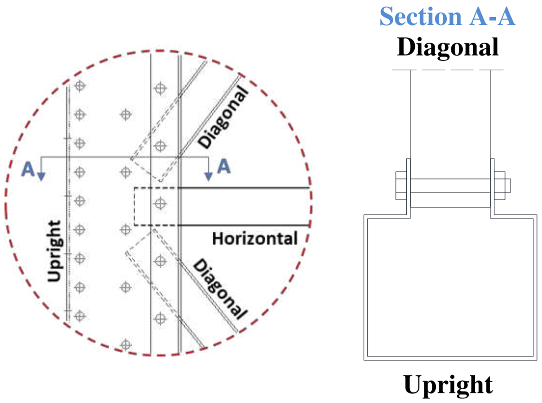


Figure 12: An example of diagonal and horizontal profile – to – upright connection (schematic version).

According to the analyzed case studies, the main path that guides all the structural choices is the structural optimization, that aims to balance the structural needs with: limiting the costs connected to the necessary amount of steel and to the additional processes at workshops (i.e. welds or additional sheets for connections are very limited); limiting the number of different cross-sections needed for the same element (i.e. diagonals, uprights, pallet beams), which implies an easier and cost effective production and a less probability of mistakes during the construction phases that may occur due to the very low involved thicknesses. Indeed, the cross-section is always kept the same for each type of element, and thicknesses change a maximum of three times along the height of the structure.

The design parameters to be freely adopted by the designers are all gathered Table 4, and the effects of the resulting design strategies on the determination of the design base shear are highlighted in Table 5. The considerations previously made in §2.7 can be here reintroduced considering the influence of the parameters in the definition of the design response spectrum, in the definition of the participant mass, and in the possible reduction of the lateral stiffness of the frames along CA direction. With respect to the analysis made in §2.7:

- The reduction of lateral stiffness is also taken into consideration, only when considered by the designer.
- The definition of the participating mass and its reduction due to the use of Ψ2 and RF factors reduces the total seismic base shear, but also increase the design seismic acceleration. For sake of simplicity, the consequences of the definition of participating mass are not considered in the calculation of the total seismic base shear reduction.

The reduction of lateral stiffness surely enables the design in terms of resistance at Ultimate Limit State (ULS), allowing the use of smaller and less thick elements, which could imply an increase of the deformation of the structure toward to horizontal actions (as wind). Indeed, deformability should be limited according to Serviceability Limit State (SLS) requests connected to the movement of the devices for handling the pallets. This implies that the design of the structure could be driven by SLS.

Looking at the total seismic base shear reduction within Table 5, 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 should be assessed if the use of these parameters, which is justified for traditional racks, is also admissible for ARSWs' racks, or if this could lead to an unsafe and not conservative design.

Table 5: Influence of the design assumptions in the reduction of the seismic design base shear.

Case Study	Direction	Seismic acceleration			Mass		Total Seismic base shear reduction [%]
		q	Kd	Reduction of lateral stiffness	Ψ_2	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**

4. Seismic assessment of double-depth ARSWs and critical analysis of the seismic design

The analysis of only the design seismic demand is surely not sufficient to understand the effects of the design choices on the overall seismic behavior of the ARSWs and on their safety levels. The design seismic demand shall be necessarily analyzed together with the resultant seismic performance and their coherence thoughtfully checked.

With this purpose, the seismic assessment of the case studies is made through FEM numerical simulations and the evaluation of the safety level at Life Safety Limit State (LSLS) by executing safety checks of the main components (elements and connections). The final aim is to point out the criticalities and the structural behavior resulting from the adopted design strategies. All the input data are the same adopted by the designers. Firstly, for each structure, a modal analysis is performed to point out the main modes and the corresponding natural frequencies. Then, Non-Linear Time History Analyses (NLTHA) are carried out using a set of 15 natural accelerograms as seismic input.

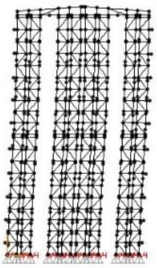
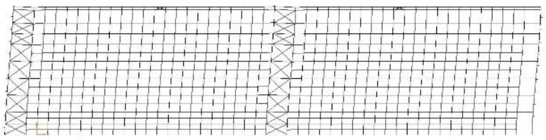
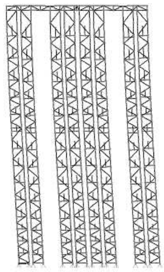
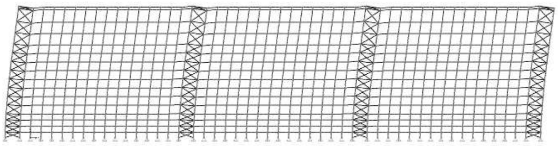
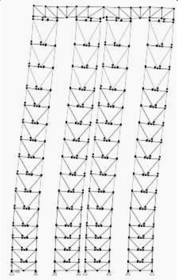
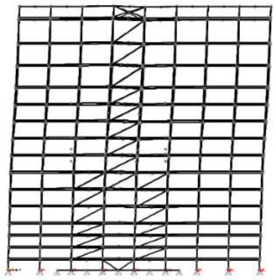
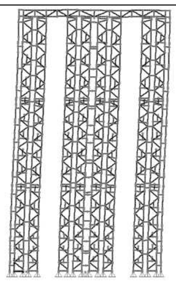
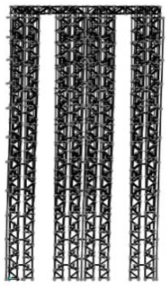
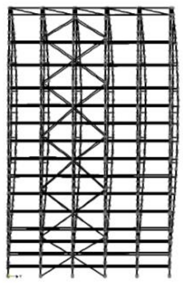
Even if 3 out of 5 designers assumed a dissipative behavior (q=2), no real capacity design is adopted for the dimensioning of the structural elements (§3.6). Therefore, a brittle or quasi-brittle behavior, with limited values of global ductility, are expected from the structural analysis of these structures. For this reason, and to understand the eventual benefits deriving from the local strengthening of the firsts elements to fail, the non-linear time history analysis used to evaluate the structural performance are executed considering only the geometrical non-linearities. Then, the weaker parts of the structure and the chain of failure mechanisms are individuated by putting in order (form the highest to the lowest) the demand-capacity ratios obtained from the execution of the safety checks of elements and connections. These re-arranged demand-capacity ratios represent the so called “hierarchy of criticalities”. This hierarchy is representative of the order of the most critical elements until a mechanism with relevant ductility is highlighted, after that the model adopted is no longer able to represent the effective behavior of the structure.

4.1 Modal Analysis

To consider the relevant influence of second order effects, 3Dmodal analysis of the representative portion of each CS starts from a deformed condition, which is obtained by a non-linear analysis with P-Delta effects included and considering only gravitational loads. Material is linear elastic, the elements are modelled as mono-dimensional, and participant mass is lumped and placed at each load level.

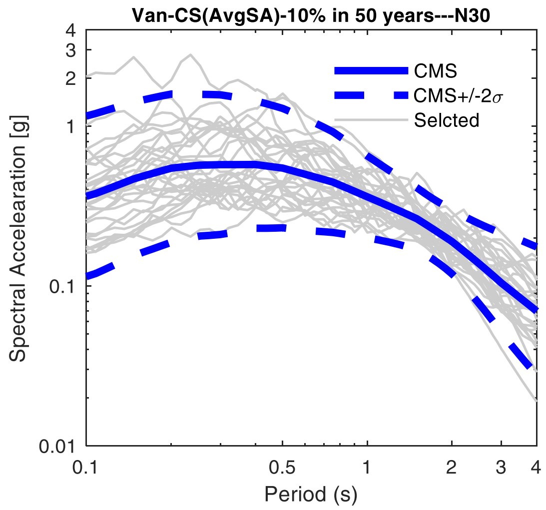
Table 6 gathers the relevant eigen modes of all the CSs. No relevant variability of periods can be found among the main modes of DA direction, especially referring to the case studies with bracing towers along DA direction. On the contrary, a wider range of values is detected for the CA modes, depending on presence of and type of connection between adjacent upright trusses, as highlighted in §3.6. In fact, CS1 and CS2 are characterized by the highest periods: in the former the adjacent upright trusses are connected by hinged and very flexible elements and in the latter there is no connection at all. In all the other case studies, the adjacent upright trusses are connected by rigid transversal beams at each load levels, which also allow the connection of the trusses to the corresponding bracing tower. These rigid elements couple the trusses, making the structure more rigid.

Table 6: Relevant eigen modes resulting from modal analysis of the case studies.

<table>
<tr><th rowspan="3">CS</th><th colspan="3">Relevant eigen modes</th><th colspan="2" rowspan="2">Representation of the main relevant modes</th></tr>
<tr><th rowspan="2">Type of mode</th><th colspan="2">Corresponding period and participating mass</th></tr>
<tr><th>T [s]</th><th>Mass [%]</th><th>CA direction</th><th>DA direction</th></tr>
<tr><td rowspan="5">1</td><td>Translational, CA direction</td><td>2.61</td><td>67</td><td rowspan="5">
T 2.61s mass 67%</td><td rowspan="5">
T 1.26s mass 64%</td></tr>
<tr><td>Translational, DA direction</td><td>1.26</td><td>64</td></tr>
<tr><td>Translational, CA direction</td><td>0.88</td><td>12</td></tr>
<tr><td>Translational, CA direction</td><td>0.50</td><td>3</td></tr>
<tr><td>Translational, DA direction</td><td>0.49</td><td>17</td></tr>
<tr><td rowspan="2">2</td><td>Translational, CA direction</td><td>2.70</td><td>67</td><td rowspan="2">
T 2.70s mass 67%</td><td rowspan="2">
T 1.71s mass 57%</td></tr>
<tr><td>Translational, DA direction</td><td>1.71</td><td>57</td></tr>
<tr><td rowspan="4">3</td><td>Translational, DA direction</td><td>1.48</td><td>57</td><td rowspan="4">
T 0.92s mass 52%</td><td rowspan="4">
T 1.48s mass 57%</td></tr>
<tr><td>Translational, CA direction</td><td>0.92</td><td>52</td></tr>
<tr><td>Translational, CA direction</td><td>0.88</td><td>6</td></tr>
<tr><td>Translational, DA direction</td><td>0.66</td><td>18</td></tr>
<tr><td rowspan="2">4</td><td>Translational, DA direction</td><td>1.08</td><td>67</td><td rowspan="2">
T 0.96s mass 52%</td><td rowspan="2">
T 1.08s mass 57%</td></tr>
<tr><td>Translational, CA direction</td><td>0.96</td><td>64</td></tr>
<tr><td rowspan="6">5</td><td>Translational, DA direction</td><td>1.29</td><td>37</td><td rowspan="6">
T 1.09s mass 40%</td><td rowspan="6">
T 1.29s mass 37%</td></tr>
<tr><td>Translational, CA direction</td><td>1.10</td><td>11</td></tr>
<tr><td>Translational, CA direction</td><td>1.09</td><td>40</td></tr>
<tr><td>Translational, CA direction</td><td>1.03</td><td>9</td></tr>
<tr><td>Translational, DA direction</td><td>0.98</td><td>29</td></tr>
<tr><td>Translational, DA direction</td><td>0.61</td><td>5</td></tr>
</table>

4.2 Non-Linear Time History analyses

NLTHAs are carried out using a set of 15 natural accelerograms as seismic input, which is selected from the NGA-West2 database (Bozorgnia et al. 2014) that matches the target conditional spectra (Baker 2011; Lin, Haselton, e Baker 2013a; 2013b) at a 2475 years return period, or equivalently an exceedance probability of 2% in 50 years, and that has been extended to a wider range of probabilities of exceedance (Kohrangi M., Tsarpalis D., Vamvatsikos D. Deliverable D.4.2. From Steelwar Research Project). For the purposes of this study, the set corresponding to an exceedance probability of 10% is adopted (Figure 13).



	2% / 50 Y	10% /50 Y
AvgSA [g]	0.4990	0.2410
Acc	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 13: 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 (Mazzoni 2017). Where possible, and more specifically for the case studies in which the behavior of the CA direction can be assumed as independent from the one in the DA direction, separate 2D finite models were used. Where this hypothesis was considered not realistic, 3D models of the structures were assumed. In all the models, the elements are modelled as beams with a linear elastic material (elastic beam column element, (Mazzoni 2017)). The participant mass was defined accordingly to the hypotheses of the case studies' designers (Table 4).

4.3 Vulnerability assessment and hierarchy of criticalities

The vulnerability assessment is carried out through the execution of the safety checks of the main components according to Eurocode 3 (in particular, prEN 1993-1-1:2019, prEN 1993-1-3:2019 and prEN 1993-1-8:2019) and EOTA documents for base connections, when post-installed anchors are used (EOTA TR45 and EOTA TR49

(2010)). The post-process of the huge quantity of data from the analyses and the associated safety checks are handled through a tailored-developed MATLAB® code (*MATLAB® version (R2019b). Natick, Massachusetts: The MathWorks Inc.*, s.d.).
The D/C ratios are re-organized from the highest to the lowest and gathered in consecutive steps of which a graphical representation is given in the hierarchy of criticalities. The most representative steps of the hierarchies are showed, for each direction, in Figure 14 and Figure 15. In these representations, two steps are gathered: the criticalities belonging to first one, which is characterized by the highest D/Cs, are represented in red, while those of the second one (lower D/Cs) in yellow. The range of D/Cs for each step is defined by the ratio among the highest D/C detected and lower/upper bound ones represented in the step: for example, if the highest D/C is 1.50, and the first step gathers the mechanisms from 1.50 to 1.30 while the second one from 1.30 to 0.80, the range of the first step is 1.00 – 0.87, and the range for the second step is 0.87 – 0.53. To give the major homogeneity to the results, the first step gathers the criticalities in the first 10% of variation of the D/Cs for the CA direction, and in the first 25% for the DA direction.
A quite similar behavior among all the case studies can be individuated for CA direction. The highest D/Cs ratios are detected at the base of the structure, where high forces are acting. From the bottom, the criticalities spread through the height of the structure. A major diffusion is allowed when different thicknesses along the height of the structure are used for diagonal or upright elements. In all the case studies, the components that are characterized by the highest D/C ratios – belonging to the first represented step - are diagonal connections and uprights base connections: the leading mechanism for diagonal connections is bearing (mainly due to the very low thickness of these elements), while the leading one for base connections is failure due to tensile (concrete-cone mechanism) and shear force on anchors. Failure of base connections is detected in all the case studies where post-installed anchors are used, while threaded bars allow better performance (see CS3, where this last solution is adopted, Figure 14c).
As regards diagonal elements, it can be noticed that the ultimate resistance of the element (both in tension and in compression) is always higher that the resistance of the connection (at least 40% higher). This is the consequence of not applying hierarchy design rules for behaviour factors up to 2, in agreement with EN 16681 (2016). The only request is to avoid a fragile failure of connection by having the shear resistance of bolts at least 1.20 times the bearing resistance. This design strategy implies that, although a fragile failure of connection is prevented, no over-resistance is provided to connection with respect to the diagonal element. In any case, for the adopted profiles, which are characterized by a low thickness, it is very hard for the bearing resistance of the connection to be higher than the tensile resistance of the element. Looking at the formula for the bearing resistance («prEN 1993-1-3: 2019 Eurocode 3 - Design of steel structures - Part 1-3: General rules - Supplementary rules for cold-formed members and sheeting»), it depends on the ultimate strength of the steel, on the thickness of the profile and on the number and position of the holes. Increasing of one or both the steel grade and the thickness leads to the increasing of both the bearing resistance and the resistance of the element, keeping the ratio between the resistances associated to the two mechanisms unchanged. On the other hand, using more bolts would allow the increasement of the bearing resistance only. Anyway, this option is not easily applicable to ARSWs' market, where a one-bolt connection is preferred, since it makes the system more flexible and its geometry more adaptable. In fact, the assembly is faster and easier, the production of the diagonal profiles is more standardized, as well as the production of uprights that would need a different pattern of the holes in correspondence of connections.

The next criticality that occurs is buckling failure of uprights for combined axial compression and bending at the bottom of the structure or where the uprights reinforcement stops.

Along DA direction, the highest D/C ratios are concentrated in the bottom part of the bracing systems and, particularly, in the central bracing tower for which the participant mass is higher (CS3 Figure 15c, CS4 Figure 15d and CS5 Figure 15e). The firsts components involved are diagonal to upright connections and upright base connections. For the former the leading mechanism is mainly the bearing of diagonal, or of the plate connecting the diagonal to the upright. For the latter, the leading failure is due to the failure of the anchors under tensile (concrete-cone mechanism) and shear forces. The next criticality occurs in the uprights belonging to the bracing towers, due to axial compression. All the other elements of the structure not belonging to the bracing systems are characterized by lower D/C ratios: when the DA bracing system is aligned with the upright trusses (CS1 Figure 15a and CS2 Figure 15b), the elements that are involved after the bracing towers are pallet beams, that work in compression to transfer the forces among the bracing towers. On the contrary, in case of eccentric bracing system (CS3 Figure 15c, CS4 Figure 15d and CS5 Figure 15e), the elements that are involved after the bracing systems are the uprights that are closer and directly connected to the bracing system itself, and failure is due to axial compression and bending.

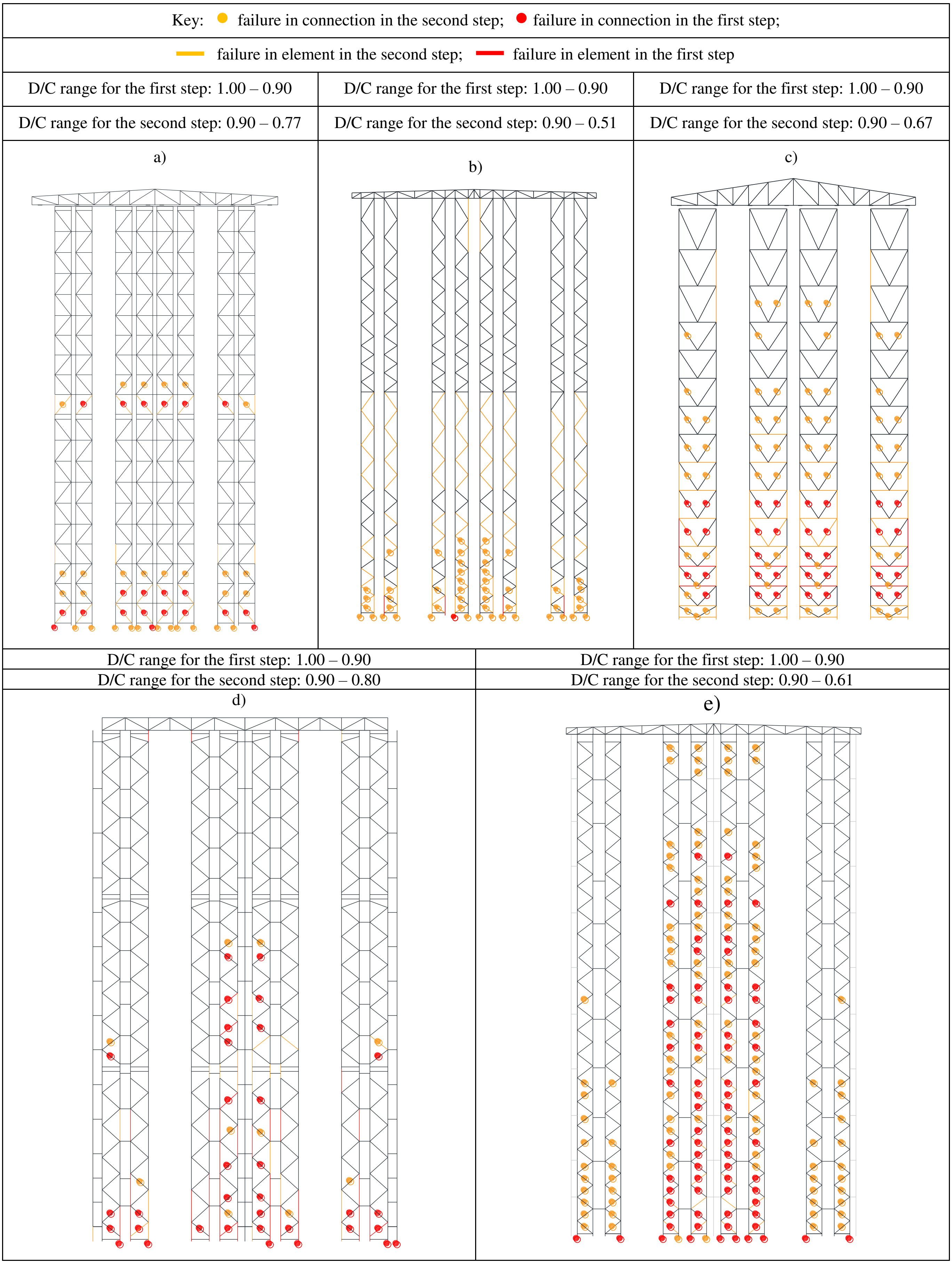


Figure 14. Hierarchy of criticalities in the CA direction for the a) CS1, b) CS2, c) CS3, d) CS4 and e) CS5.

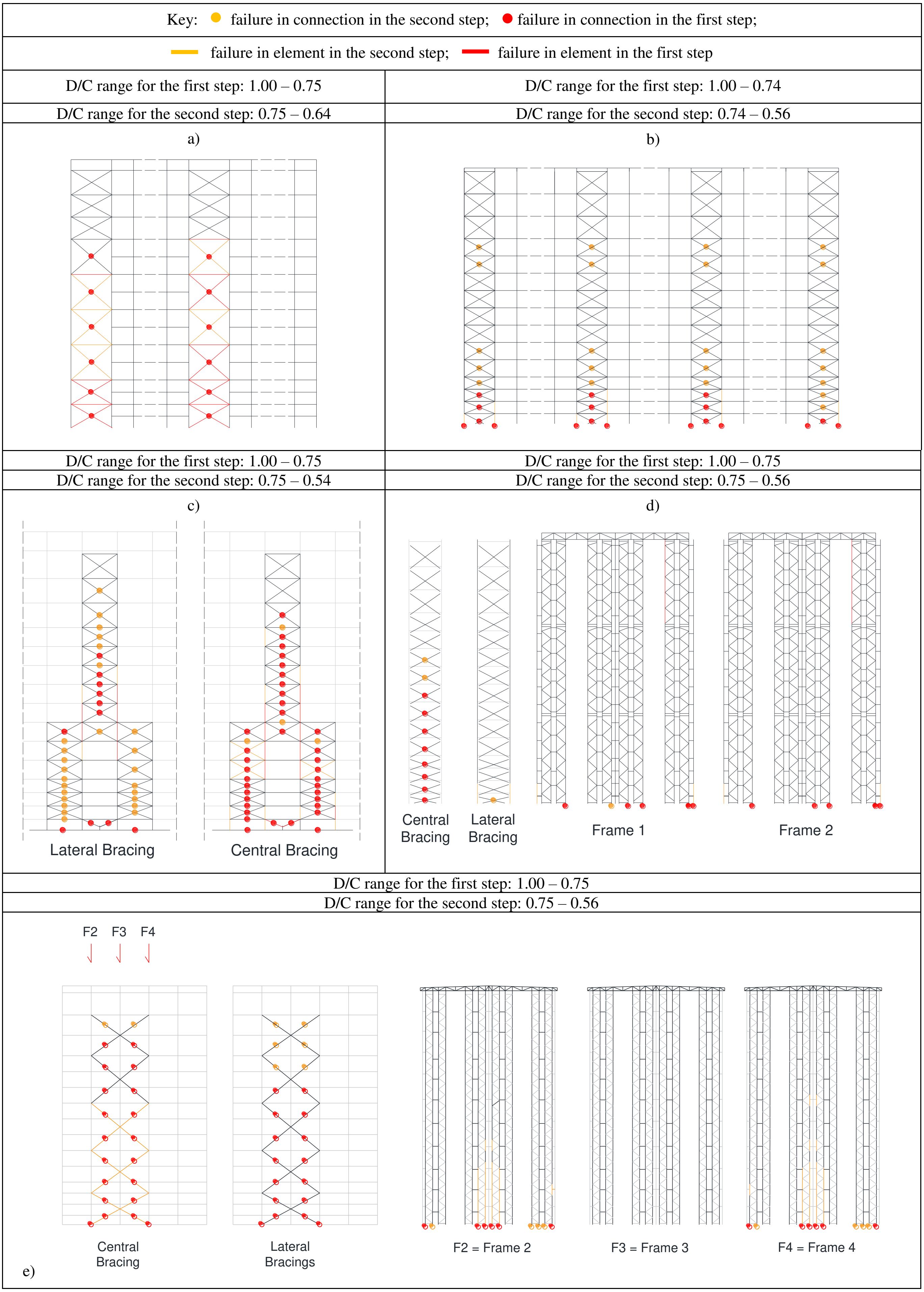


Figure 15. Hierarchy of criticalities in the DA direction for the a) CS1, b) CS2, c) CS3, d) CS4 and e) CS5.

5. Conclusions

This paper critically analyses the current structural choices, solutions and resulting performance of Automated Rack Supported Warehouses. These buildings are largely used all over the word as automated and advance storage systems for palletized goods, and are currently designed by adopting approaches and regulations not specifically developed for them, causing confusion in application, possible not-safe design and, in some case, poor performance, especially for seismic applications

The key design parameters have demonstrated of having a relevant impact on the definition of the design response spectrum and on the modal parameters of the structure (natural frequencies), and consequently on the value of the seismic design force. As highlighted in Table 1, the adoption of different values of these parameters and their combinations can bring to relevant differences in the definition of the design seismic base shear, with a maximum reduction of 54% for CA direction and to 69% for DA with respect to the elastic design value. Even if this evaluation is made for a specific configuration, it gives the idea of the influence of these parameters on the seismic design of the ARSWs. It should be also highlight that the choice of one value for a parameter with respect to another one is not associated to different design constraints. Besides, the use of most of these parameters is widely scientifically justified for traditional racks, but it is not for ARSWs, so it's not clear if they lead to an unsafe and not conservative design.

From the analysis of the configurations and of the structural choices made by 5 European manufacturers companies, it seems that the main path that guides all these structural choices is structural optimization, which aims to balance the structural needs with limiting the costs connected to the necessary amount of steel and to the additional processes at workshops (i.e. welds or additional sheets for connections are very limited), and, for the same element (i.e. diagonals, uprights, pallet beams), limiting the number of different cross-sections needed. In any case, the structural solutions can be deeply different among each other.

Dealing with the structural performance and vulnerability assessment along the CA direction a similar behavior can be individuated among all the case studies in terms of hierarchy of criticalities, and the components that are characterized by the highest D/C ratios are diagonal connections (due to bearing) and uprights base connections (with post-installed anchors, due to tensile and shear force). Along DA direction, the same criticalities can be found: the highest D/C ratios are concentrated in the bracing systems, starting from the bottom, and firstly involving diagonal to upright connections, where the leading failure is bearing, and upright base connections, especially when post-installed anchors are used, due to tensile and shear force. Basically, the failure of connections is the most critical in both directions. This is one of the possible consequences of not applying any hierarchy rule in the design of the structure, although a behavior factor major than 1.5 has been used. If plastic ovalization of diagonal-to-upright connection occurs first, these connections would become loose without the possibility of dissipating energy anymore and with an asymmetric behavior (due to the different distance of the hole form the free edges in tension and in compression). Indeed, this could be accepted only if limited dissipative capacity is expected, and preventing any other fragile mechanism in diagonal-to-upright connection and all the other components from failing. On the other hand, if crisis of an upright base connection is the first to happen, this could trigger a series of chain collapses, leading to the collapse of the whole structure.

These outcomes highlight that the current design approach could be applied only if the structure is designed to remain in the elastic field, but also in this case, the use of the key design parameters for the definition of the response spectrum should be justified. On the other hand, if the structure is designed to be dissipative, post-elastic sources appear to be very limited, especially if the indications of EN 16681 (2016) are applied. In this purpose, the fragile or quasi-brittle failures that are detected from the structural assessment shall be avoided and prevented by developing a proper design approach for these structures, that should foresee the use of capacity design to drive the chain of failure in the proper direction and control the global collapse mechanism.

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STATEMENTS AND DECLARATIONS

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