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Influence of the Design Parameters on the Current Seismic Design Approach for Automated Rack Supported Warehouses

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Abstract **-** Automated Rack Supported Warehouses (ARSWs) are huge steel buildings offering storage solutions. These constructions have been facing a huge diffusion in the last decade, mainly due to the necessity of having bigger places to stock goods and handling them through automated systems. They constitute the upgrade of traditional steel racks, with the considerable difference of racks being the primary structural system of the building, besides being the storage place for goods. The fast evolving of the market and request of an efficient management of high volumes of goods brought a rapid development and use of such structures without their design being supported by a specific regulatory framework. This gap also involves seismic design, and this is evident from the recent collapses and damaging of such structures after seismic events. The current design of ARSWs is made adopting the same regulations for steel racks, but, even if traditional steel racks and ARSWs have several common aspects, there are relevant differences that do not allow to adopt the very same design approach. With the aim of highlighting the factors and parameters currently influencing the design of these constructions, the present paper analyses the technical guidelines and regulations currently adopted by technicians and designers, highlighting the key parameters and their influence on the definition of the seismic demand. This critical analysis is made taking into consideration typical structural configurations for ARSWs.

Keywords**:** Automated Rack Supported Warehouse, steel rack, cold formed, design parameters, seismic demand.

1. Introduction

Large scale economy and trade are widely diffused all over the world, involving both smaller and bigger realities. Big quantities of goods can be produced and moved, leading to the necessity of organized stocking areas are needed when the volume of goods increases. As a traditional solution to these necessities, steel racks have been available from the last decades of the XX century: they are mostly made up of repeated modular trusses, very easy to be assembled, changed in geometry and removed, where goods can be placed after being organized and gathered in pallets. Steel racks are usually placed inside warehouses, and they are completely independent from the structure of the building. They are usually composed of cold-formed components, with specifically manufactured cross-sections and connections, thus very peculiar and sometimes non-standard, whose structural behaviour has been analysed and studied by many authors and in many research projects. In Europe, the outcomes of two main projects ([1], [2]) have been implemented in the current standards for the design of steel racks (prEN15512 [3], and prEN16618 [4], used respectively for static and seismic design). However, racks have structural limits - as height - that may become restrictive and prohibitive when the number of pallets significantly increases. Besides, the bigger are the racks, the bigger shall be the structure of the warehouse, implying unaffordable additional costs, which is against the cost-benefit policy that characterizes this field.

Automated Rack Supported Warehouses (ARSWs) offer a solution to these issues: in these buildings, the function of racks is not limited to storage, but they also constitute the structure of the warehouse, with the further benefit of a total automated handling of pallets. Thus, ARSWs merge the optimization of both spaces and management of goods flows. Being the natural and rapid development of traditional steel racks, ARSWs have inherited several structural characteristics that are typical of them, although the global structural functioning is different, and their increasing use was not supported by a dedicated and specific regulatory framework. In fact, in Europe, ARSWs are currently designed by using the regulations for traditional racks ([3], [4]), that are allowed to be adopted when specifics are not covered by Eurocodes. This way of proceeding is actually allowed by [3] at §1.1 and by [4] at §1, but, in any case, the standards for traditional racks treat racks as “load bearing structures for the storage of goods in warehouses” (§0.1 in [3]), which is different from their actual structural functioning in ARSWs. At the same time, Eurocodes may be not fully applicable, due to the peculiar characteristics of

ARSWs (as overall geometry and dimensions, structural types, shape of components, atypical and non-standard components) that make them different from ordinary structures ([5]). The confusion between the two systems and the normative gap can lead to unsafe design, with possible catastrophic consequences.

ARSWs structural behaviour can be analysed at local and global point of view. The global one has not been much investigated by the scientific community: in literature there are contributions from a few authors ([6], [7]) which focused mainly on finding proper strategies for seismic design, stating the lack of specific guidelines. From the local perspective, there aren't specific references to ARSWs' components, but, actually, we can fully refer to the studies made for traditional racks', since ARSWs inherited all of them. As an instance, plenty of research can be found about: (i) racks' columns ("upright" in jargon), which are mainly characterized by a U lipped section and are usually continuously perforated along their length, implying uncertainties and the necessity of experimental evidence to determine their resistance in compression and bending [8], [9]; (ii) upright trusses, which constitute the load resisting and stiffness providing structure in one of the two direction of the racks (also valid for ARSWs), which are characterized by a reduced shear stiffness due to the eccentricity of diagonal-to-upright connection and its deformability (Figure 1) ([10]). This significantly affects the global behaviour of the full structure in this direction referring to the main fundamental period and the deformability; (iii) pallet beam-to-upright connections, which are not standard and manufactured *ad hoc* for racks, and which stiffness is a relevant parameter to be determined, especially for traditional racks ([11]). The same is for upright base connections ([12]). The study of these components was deeply investigated within SEISRACKS and SEISRACKS 2 research projects ([1], [2]), which gave a great contribution to traditional racks design, both in static and seismic conditions.

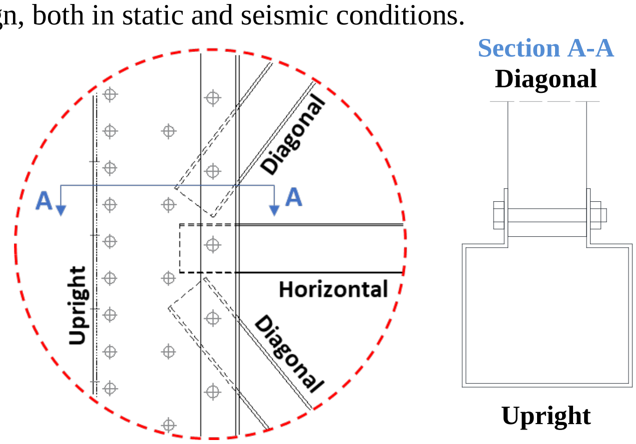


Figure 1: An example of diagonal and horizontal profile – to – upright connection.

All these studies highlighted the extreme peculiarity of racks' components, and the resulting difficulty of characterizing the local and global structural behavior, which must be always validated by experimental evidence. Although plenty of research has been conducted, especially for steel racks and their components, it can be observed that what is missing is the awareness of the change of structural functioning of racks, and the consequent questioning of possibility of application of available research outcomes and related regulations to ARSWs design. The impact of this uncertainties can be observed also in seismic design and the poor performances of some ARSWs that collapsed after some seismic events.

This paper is focused in pointing out the relevant parameters for the seismic design of ARSWs, which mostly derive from traditional racks design strategies. A critical analysis is conducted to highlight their influence in the structural behavior of the building and in the definition of the seismic demand, which could lead to an unsafe design.

3. Regulations and Relevant Parameters for the Seismic Design of ARSWs

As previously stated, ARSWs are currently designed by adopting the standards for traditional racks, which are prEN15512 [3] and prEN16681 [4]. Dealing with seismic design, according to [4], several parameters which are connected to typical aspects of racks shall be considered for the definition of the seismic demand and the dynamic characterization of the structure:

1. The sliding of pallets when friction is overcome contribute to the damping of the structure, affecting the definition of the response spectrum [2].
2. The distribution of goods inside the warehouse and their content can influence both the response spectrum and the definition of the participant mass.
3. The dynamic response can be affected by the reduction of shear stiffness of the upright trusses in the Cross Aisle (CA) plane (which is one of the two relevant directions of ARSWs – Figure 2).
4. Different behaviour factors can be adopted, depending on the structural type, to which are associated different design rule, affecting both seismic demand and the structure performance.

All these aspects are included in [4], but they could be in deep contrast with the design principles for buildings included in Eurocodes, as highlighted in the following.

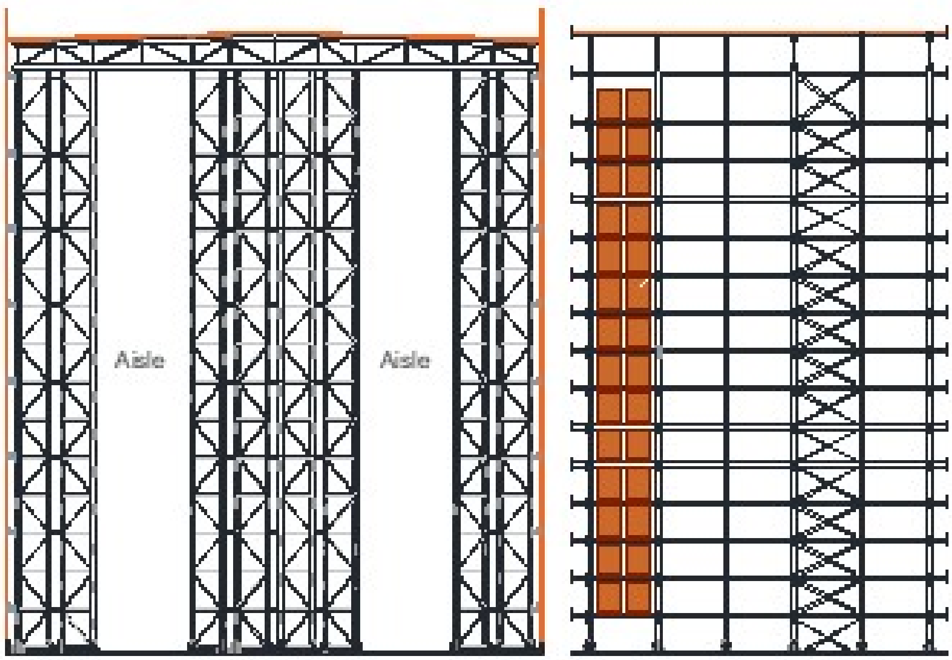


Figure 2: Main directions of ARSWs: on the left, the Cross Aisle (CA) direction; on the right, the Down Aisle (DA) direction.

2.1. Definition of design response spectrum

Modal response spectrum analysis can be used for the design of racks, and Eq. (1) can be used for the definition of the design response spectrum $S_{d,mod}$:

$$S_{d,mod}(T) = K_D \cdot S_d(T) \tag{1}$$

where $S_d(T)$ is the design spectrum defined by Eurocodes for the associated importance factor, considering a viscous damping factor ξ equal to 3%, and K_D (Eq. (2)) is a factor that consider the dissipation of seismic energy due to the sliding of pallets when friction is overcome.

$$K_D = 1 - {}^{P_{E,prod}}\!/_{P_E} \cdot (1 - E_{D1} \cdot E_{D3}) \tag{2}$$

In Eq. (2), $P_{E,prod}$ and P_E are respectively, the weight of the stored goods and the total weight of the structure (dead, permanent and goods stored weight). Given that, in case of racks, the weight of the structure and of the permanent loads are a very low percentage of the total weight of the fully loaded racks, 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} and E_{D3} considers the dissipation phenomena: E_{D1} 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); E_{D3} is defined to include the phenomena typical of the dynamic behaviour of racks under seismic action in the formulation of the spectrum. E_{D3} is equal to 0.8, as resulting from SEISRACKS 2 [2].

2.2. Definition of participant mass

The seismic participant mass is defined as Eq. (3), with $W_{E,G}$ as the permanent loads, $W_{E,Q}$ as the variable ones, and $W_{E,UL}$ is the design seismic weight of the goods (Eq. (4)).

$$W_E = W_{E,G} + W_{E,Q} + W_{E,UL} \tag{3}$$

$$W_{E,UL} = R_F \cdot E_{D2} \cdot Q_{P,rated} \tag{4}$$

In Eq. (4), R_F is the filling grade reduction factor, which is defined on statistical evaluations based on the occupancy of the stored goods in the rack that during a seismic event. Considering the higher probability of full occupation of CA frames, R_F is equal to 1.0 in this direction, while lower values (between 1.0 e 0.8) can be assumed for Down Aisle (DA) direction (pallet beam direction – Figure 2), since usually the amount of busy pallet spots decreases from the entrance to the rear of the warehouse, according to the logistic rules [13]. E_{D2} is for the interaction among unit loads and structure depending on their content: if the stored goods are compact, constrained or liquid, it is equal to 1.0, while, if they are loose and unconstrained, E_{D2} is equal to 0.7. Finally, $Q_{P,rated}$ is unit loads weight value, which is defined basing on the foreseen goods flows.

2.3. Shear flexibility of upright frames along CA direction

In upright frames, diagonals are usually directly connected to uprights through a single-bolted connection. The eccentricity of this connection with respect to the centreline of the upright, the looseness and deformability of connection and of the connected elements cause a reduction of the global stiffness of the system, affecting the dynamic response of the structure. Many authors proved that Timoshenko's stiffness formula for built-up columns, which is suggested by different codes (RMI [14], FEM [15]) for the evaluation of shear stiffness of steel racks upright trusses, is not trustable for such systems, bringing up to a relevant over-estimation of the value [16], [17]. Although this may be conservative for design in terms of resistance, lateral deformations may be under-estimated, and deformability limits may not be respected, affecting the operational phases and overall system stability. PrEN 15512 [3] suggests that the use of Timoshenko's formulas should be limited and replaced by experimental activity, and provides the procedure for the execution of the tests.

2.4. Possible approaches for dissipative design: structural types, behaviour factors and design rules

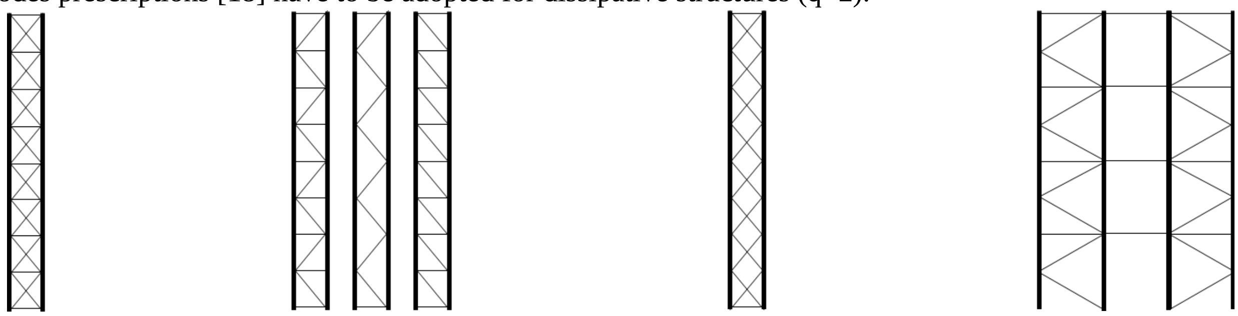
Earthquake resistant racks can be designed also adopting dissipative approaches (§ 8.1 prEN16681 [4]) by applying low dissipative and dissipative concepts. In the former, elastic global analysis without considering non-linear material behaviour can be used for seismic response assessment, while, in the latter, plastic deformations are expected in the dissipating elements, and the behaviour factor q is used to consider the expected dissipation capacity. For the low dissipative concept, q-factor up to 2 can be used, while, in the other case, a q factor major than 2 can be used.

To design low dissipative racks ($1.5 \leq q \leq 2$), many structural types can be adopted. In traditional racks, the mostly used structural type for DA direction is the moment resisting frame: uprights are fixed or semi-rigidly connected to the concrete slab, and pallet beams are connected to uprights through semi-rigid connections. In ARSWs, higher stiffness and resistance is needed in this direction, more than the one provided by the simple rack system, thus implying the necessity of additional bracing systems specifically dedicated to the seismic response.

Dealing with the CA upright frames, constituting traditional racks or ARSWs ones, diagonals can be arranged in (i) the X-shape with tension-only diagonals (Figure 3a); (ii) the K, D, Z shapes (Figure 3b); (iii) the X-one without horizontal members, with tension-compression diagonals (Figure 3c). To the (i) solution corresponds a q-factor of 2 with no hierarchy rules for the design of the components (neither in the dissipative elements, nor in their connections, nor in the other elements and their connections), in contrast to Eurocodes prescriptions [18]. Dealing with (ii), a behaviour factor of 1.5 must be used, and the seismic design force of diagonals and their connections has to be increased of 1.5. Besides, if bolted connections are adopted for diagonals, shear resistance of bolts should be at least 1.2 times the bearing resistance of the connected profiles to reduce the risk of brittle failure mechanisms. The non-dissipating elements are not included in these safety design rules.

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 prescriptions [18]. On the contrary, the D bracing type corresponds to a frame with K bracings (as defined by Eurocodes, Figure 3d), which is not allowed from these regulations to be used to design seismic resistant structure (§11.4.1 [18]). For the (iii), a behaviour factor of to 1.5 is suggested with no adoption of hierarchy design rules, 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. This structural type could be compared to a frame with K bracings (as defined by Eurocodes, Figure 3d), which is not allowed to be adopted, as previously highlighted.

Eurocodes prescriptions [18] have to be adopted for dissipative structures (q>2).



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

Figure 3: Main structural types for racks according to prEN16681[4] (from a to c), and K bracings type for steel structures according to Eurocode 8 [18] (d).

3. Influence of the Design Parameters on the Seismic Demand and Structural Characterization of ARSWs

The effects of the possible design approaches (or Design Hypotheses - DHs) resulting from different combinations of the key design parameters previously introduced are gathered in

Table 1. All the DHs discussed in the following derive from the adoption of parameters that are nor foreseen in Eurocodes, and all of them lead to a relevant reduction of seismic demand, although not very well justified and demonstrated for ARSWs. For sake of simplicity, the CA and DA directions are investigated separately. For each DH, the variations of participating mass, period, design horizontal seismic acceleration, and base shear are pointed out with respect to a reference DH, that corresponds to elastic design (q=1.0 and K_D=1) and no reductions for the participating mass (R_F=1.0 and E_{D2}=1.0). The followings further assumptions are made for the comparison:

- a design life of 50 years and an importance factor of 0.8 (corresponding to an importance class I and a 20% probability of exceedance in 50 years, as suggested for racks with completely automated handling of goods within table 1 of prEN16618 [4] (§6.3)) are assumed to define the elastic response spectrum. An European high seismic zone with a Peak Ground Acceleration (PGA) equal to 0.30g for a return period equal to 475 years is considered;
- for the DHs (but the reference one), K_D is equal to 0.8, based on the hypothesis that the weight of the goods is at least 90% of the total weight of the structure (Eq. (2), §2.1);
- the stiffness of the structure is a fixed parameter. A period of 2.10 s is associated to the CA direction, and of 1.50 for the DA one. These values belong to the usual range for such structures. The reduction of stiffness along CA direction (§2.3) is not included in this comparison, since it depends on many parameters (profiles and connections geometric characteristics) and cannot be standardized, but shall be supported by experimental evidence;
- for the DHs (but the reference one), participant mass is defined as Eq. (3) and assuming R_F equal to 1.0 for CA direction and 0.8 for DA direction. The effects of considering different values of E_{D2} are investigated by assuming alternately 1.0 or 0.8;
- a behavior factor of 1.5 is assumed for CA direction, which is associated to the trellis scheme (tension-compression diagonals) that is the one typically adopted. For the DA frame, the consequences of adopting both the behavior factors 1.5 and 2 are investigated.

The effects of the different combinations of the key parameters are firstly highlighted separately for the participant mass and the design response spectrum, and then gathered in the synthesis parameters of design acceleration and base shear, which variation is given with respect to the reference DH. A visual support of the considerations made in

Table 1 is given in Figure 4a and Figure 4b respectively for the CA and DA directions. The parameters for the reference DH are always marked in red. In particular:

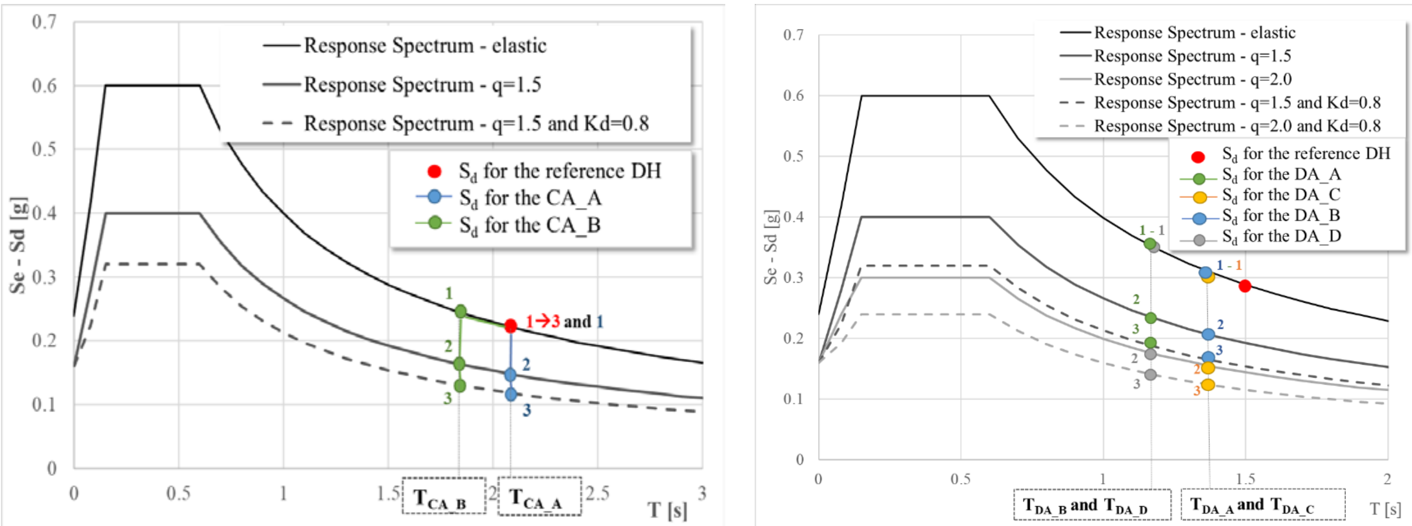
- *STEP 1: Effects of participant mass definition on the seismic acceleration.* R_F and E_{D2} are the key parameters, which affect the mass (which is reduced), and consequently the period (which is reduced) and the design acceleration (which is increased) (
- Table 1 and points 1 of Figure 4a and 4b);
- *STEP 2: Effects of design response spectrum definition on the seismic acceleration.* The behavior factor q and the K_D affect the design acceleration (which is reduced) (
- Table 1 and points 2 and 3 of Figure 4a and 4b);
- *STEP 3: Final definition of the seismic acceleration.* Although there are opposite effects on seismic acceleration, a reduction is always detected (the reduction due to q and the K_D is always more relevant than the increase due to the reduction of period), as well as in the base shear.

The differences among the variations of the base shear among the DHs are quite sizeable, going from -47% to 54% for CA direction and from -54% to 69% for DA. Regarding the CA directions, these values could have also been worse if the reduction of stiffness due to shear flexibility would have been considered. 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 prEN16681 [4] (when prEN16681 is used to design ARSWs) is in the possibility of using a behaviour factor up to 2 without adopting properly hierarchy 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 = fundamental period; S_d = design seismic acceleration; q = behavior factor

	Design Hypothesis (DH)	**1- Participant mass**					**2- Design response spectrum**			**3- 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 DH	1.0	1.0	**-**	**-**	**-**	1.0	1.0	**-**	**-**	**-**
	A	1.0	1.0	**0**	**0**	**0**	1.5	0.8	**-47**	**-47**	**-47**
	B	1.0	0.8	**-20**	**-11**	**+9**	1.5	0.8	**-47**	**-42**	**-54**
DA direction	Reference DH	1.0	1.0	**-**	**-**	**-**	1.0	1.0	**-**	**-**	**-**
	A	0.8	1.0	**-20**	**-11**	**+9**	1.5	0.8	**-47**	**-42**	**-54**
	B	0.8	0.8	**-36**	**-20**	**+20**	1.5	0.8	**-47**	**-36**	**-59**
	C	0.8	1.0	**-20**	**-11**	**+9**	2.0	0.8	**-60**	**-56**	**-65**
	D	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.



(a) (b)

Figure 4: Effects of the key parameters on the period and seismic acceleration for the CA (a) and DA (b) DHs, where: 1 is for the variation of S_d due to definition of participant mass; 2 is the reduction of S_d due to the behaviour factor; 3 is the reduction of S_d due to the K_d factor.

5. Conclusions

In this paper, a critical analysis of the current approach for the design of ARSWs is performed. Given the lack of a dedicated regulatory framework, these structures are designed by adopting the standards for the traditional steel racks ([3], [4]) without considering the relevant change of structural functioning of these systems in ARSWs. Firstly, the key parameters affecting the definition of the seismic demand are pointed out and discussed. Then, their influence is analysed with respect to a reference Design Hypothesis (DH) (elastic design and no reductions for the participating mass) by combining the different parameters in possible DHs for both CA and DA direction.

The differences of the base shear variations of the DHs are quite relevant, going from -47% to 54% for CA direction and from -54% to 69% for DA, and, regarding the CA directions, worse outcomes could have been founded if the reduction of shear stiffness was considered. Anyway, what is worthy to be pointed out is not only the size of reduction of the design base shear, but that it is the consequence of using parameters that have a scientific and experimental validation for traditional racks only, and not for ARSWs. This could actually lead to unsafe design. As a complement to this analysis, the effects of the use of this design approach should also be observed through the structural seismic assessment of real ARSWs. In addition, this could give an idea of the impact of not using any hierarchy rules even if adopting q factors higher than 1.5, which is deep in contrast with Eurocodes [18].

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