Structural performance of automated multi-depth shuttle warehouses (AMSWs) under low-to-moderate seismic actions

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Abstract

Automated Multi-Depth Shuttle Warehouses (AMSWs) are compact storage systems that provide a large surface occupation and therefore maximum storage density. AMSWs represent the future of storage technology, providing substantial savings in terms of cost, space, and energy with respect to traditional warehouses. Currently, designers refer to the standard building codes for the seismic design of AMSWs. Since structural characteristics of AMSWs are considerably different from the steel structures of typical buildings, this current approach used by designers is questionable in terms of safety and efficiency. In this article, the behavior of 5 AMSW structures has been studied performing 150 time-history analyses by direct integration including P-Delta effects. Demand/capacity ratios calculated for each element showed the dominance of the brittle failure mechanism in AMSWs subjected to low-to-moderate seismic actions. These mechanisms mainly took place in upright columns and their base connections prior to the activation of ductile energy dissipation mechanisms of the structure. Based on the results, further improvements have been recommended for the future design provisions, which may lead to a safer seismic design of AMSWs.

Keywords Automated multi-depth shuttle warehouses (AMSWs) · Clad racks · Racking structures · Cold-formed structural members · Seismic vulnerability · Low-to-moderate seismicity

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1 Introduction

A warehouse is a building where a massive amount of goods can be stored before its large distribution to the consumers. Due to the increasing mass production of goods and increasing consumption levels, demand for storage space increases. As the land becomes every day more valuable in both economic and environmental terms, highly optimized, reliable, and safe warehouses are needed (Baker and Canessa 2009). Reduced running costs, automation, and larger sizes are the future trends of the warehouse sector (Liu et al. 2018; Lerher et al. 2021). The need of bigger and optimized working spaces, together with the possibility to increase the market, led to the continuous development of the storage technology (Bernuzzi and Simoncelli 2017b; Salvatore et al. 2016). Automated Multi-Depth Shuttle Warehouses (AMSWs) are one of the most modern types of warehouses that provide a large surface occupation. They are a particular type of Automated Rack Supported Warehouses (ARSWs). At present, ARSW also known as Clad Rack warehouses are usually built by manufacturers specialized in structural systems for logistics with the same or similar cold-formed profiles used for Warehouse Storage Pallet Racks. However, in the case of ARSW, the rack forms the load-bearing structure of the whole building by itself. Figure 1 (<https://www.mecalux.com>) shows this type of building during the construction.

There is no official specific reference document for the design of automated high-rise warehouses, which leads designers to adopt the rules and parameters conceived for steel buildings to these particular structures, without any control of the specific construction characteristics of ARSWs. To design this type of highly sophisticated non-building structures, designers usually refer to the EN 15512 (2009) which provides principles for the structural design of pallet racking systems and EN 16681 (2016) which indicates principles for the seismic design for pallet racking systems, applying them in a combination often more by their own experience and engineering practice rather than by well-established principles and rules supported by experimental evidence and theoretical research (Bernuzzi and Simoncelli 2017a; Bernuzzi et al. 2015). However, ARSWs are much larger, taller and complex systems with respect to usual pallet racks. As a consequence, the big question mark for producers, design offices, and experts is about the suitability of these norms to be used for high-tech massive, automated warehouse buildings.

The structural collapse of ARSWs may have large economic impacts not only for the owner but also, more in general, for the community, in terms of loss of vast amounts of goods/merchandise. Furthermore, despite the limited number of workers required in automated warehouses, the life of the employees working inside and around the warehouse

Fig. 1 ARSW during the construction (<https://www.mecalux.com>)

Fig. 2 An ARSWs collapse in Sant'Agostino. **a** Collapsed and undamaged parts of ARSW Galli et al. (2012). **b** Longitudinal frame and its base connections failure in ARSW's collapse in Sant'Agostino EPI-Centre Field Observation Report No. (2012)

might be at risk. Hence, the solution to the problems connected with the safe and reliable design of clad rack buildings has a huge economic and safety impact. Some ARSWs were damaged during the Emilia Earthquake. In particular, the ceramic warehouse in Sant'Agostino collapsed. Figure 2a (Galli et al. 2012) shows that part of the structure collapsed, while another part of it was barely damaged. The main cause of the failure was inadequate lateral resistance in a longitudinal direction, where approximately 70% of the structure collapsed. For this warehouse, the connection system between the frames with the foundation was also very weak as the vertical elements were observed to fail at the vertical element-to-foundation interface as indicated in Fig. 2b (EPI-Centre Field Observation Report No. EPI-FO-200512 2012).

In this article, the focus is given to Automated Multi-Depth Shuttle Warehouses (AMSWs), as a specific ARSW type. AMSWs are compact systems providing large surface occupation and maximum storage density. In AMSWs, the “aisles” enable the navigation of machines (shuttle and satellites) to reach the storage positions. Semi-automatic pallet

Fig. 3 Automated multi-depth shuttle warehouse (<https://www.ferrettogroup.com>)

shuttle system minimizes operational time. Operators do not need to use forklifts to handle the goods, it is done by shuttle, which carries out the movements autonomously. By removing the need to drive forklifts into the lanes, storage capacity is increased in terms of depth, the risk of accidents and damage to the racks is negligible, operator movements are optimized, and warehouse operation is modernized and made more flexible (<https://www.mecalux.com>). The main elements of storage distribution in AMSWs are (Fig. 3 (<https://www.ferrettogroup.com>)):

- Unit load—basic storage and transport unit. All the goods in AMSWs are stored in unit loads.
- Pallet—support for the unit load.
- Satellite—machine, that is controlled remotely. It distributes pallets to their position in the warehouse.
- Shuttle—machine that distributes satellites along the longitudinal direction of the warehouse.

In a typical AMSW satellite moves from the shuttle tunnel up to the rear of the single cell, transporting one unit load at a time. Shuttle moves from an elevator tower to the opposite one, transporting the satellite with the unit load on top of it.

Today, a limited number of studies about ARSWs and their structural behavior exists worldwide. Therefore, reference will also be made to the state of the art of storage

Fig. 4 Cross-aisle and down-aisle directions (AS 4084 2012)

rack systems, which represents the structural basis of all Rack Buildings. Different rack types were introduced by Pekoz and Winter (1973). Racking is constructed from steel components including upright frames, beams, and decking. Special beam to column (so-called upright) connections and bracing systems are used, in order to achieve a three-dimensional steel “sway” or “braced” structure. These structures mainly consist of cold-formed steel members. There are two principal directions of the structure—the cross-aisle (CA) and the down-aisle (DA); they are indicated in Fig. 4 (AS 4084 2012).

In the cross-aisle direction, two (often perforated) upright sections linked together by a system of bracing members to provide lateral stability of the structure in this direction. Bracings are usually X, D, Z and K type. In the down-aisle direction, the main lateral force resisting system is provided by vertical bracing. Diagonal bracing members have pinned or welded connection to the uprights. The beam is connected to upright through a specific connector, which behavior is semi-rigid in flexure. These joints can also provide lateral stability in the down-aisle direction in the absence of a vertical bracing system (Castiglioni 2016). Numerical and experimental research performed on the seismic behavior of pallet rack structures were mainly focused on structural components. Such as beam-to-upright connection, base plate connection, compression and tension tests on structural elements (Zhao et al. 2004; Bernuzzi and Castiglioni 2001; Aguirre 2005; Beattie 2006; Gibert and Rasmussen 2009; Simoncelli et al. 2020). Also, several full-scale shake table tests were performed on these structures (Filiatrault et al. 2006; SEISRACKS 2004, and SEISRCAKS II 2011). Nowadays, there are only a few studies about the seismic behavior of clad rack warehouses in the literature. Kilar et al. (2011) performed research regarding high-rack steel structures (it is not an ARSW, but its structural behavior is close to clad racks). He concluded that the proposal of EN 1998-1 (European Committee for Standardization 2005), regarding 5% of center of mass eccentricity for typical buildings is not applicable for high-rack structures where the eccentricity can reach up to 10% in specific loading conditions. Therefore, the critical condition during seismic event has been reached when rack was loaded between 55 and 80%. However, EN 16681 (2016) states that the critical condition during the seismic event is reached when rack is 100% loaded. Rafiqul Haque (2012) performed a specific research on clad rack structures. He investigated only frames in a down-aisle direction in moment resisting configuration using incremental dynamic time history analysis. Selected frames had 4, 6, 8 and 10 stories. It was concluded that moment-resisting frame configuration is not appropriate as a lateral load resisting system for high-rise clad racks. It has much higher flexibility comparing with traditional steel moment-resisting frames due to the limited energy dissipation capacity of the beam-to-upright connector. In addition, their inter-storey drift ratios were two times higher than of traditional steel structures. As a consequence, the bracing system in the down-aisle direction is needed. Caprili et al. (2017, 2018) concluded that ARSWs cannot be treated as storage rack systems and therefore, they have to be studied as a “building-like” structure. Numerical studies were performed using pushover analysis on not dissipative and dissipative concepts, in accordance with EN 1998-1 (Kilar et al. 2011) and Italian building code NTC (2018). A comparison between two design concepts showed higher diffusion of the plastic hinges in the dissipative case. The main outcome of this research is that standards for typical buildings are not applicable for ARSWs and specific code for these structures is required.

AMSWs are complex structures, and their structural behavior is hard to predict analytically. The research made up to now is mainly limited to steel storage racks which are a much smaller scale of automated warehouses. Automated storage systems, which will probably be the future of the warehouse sector, have not been investigated to such an extent so far. In a few research works that were done on clad racks, some shortcomings of current

design provisions have been noted. AMSW is treated as a structure in between typical storage racks and multi-story buildings without its own design code, which causes a lot of doubts for structural engineers all over the world during the design of these structures. Therefore, extensive research is needed to fully describe AMSW's structural behavior and draft a new design code on it.

In this work, to quantify the structural performance of Automated Multi-Depth Shuttle Warehouses (AMSWs) in the context of low-to-moderate seismicity, The Authors collected 5 different AMSW configurations designed by 5 European rack producers according to current standards [EN 15512 (2009), EN 16681 (2016)]. As site location, Montopoli (Pisa, Italy) has been chosen, which is characterized by a peak ground acceleration (PGA) of 0.13 g. The Authors produced 15 accelerograms compatible with the chosen location and performed 150 time-history analysis by direct integration including P-Delta effects using a commercial numerical analysis software (SAP2000 2020). Because of the complexity and dimension of such structures, the efficiency of different yielding patterns has been assessed by performing numerical analyses with the simplified models (without warping effects). Warping deformations (nonuniform out-of-plane displacements in the cross-section that arise as a result of torsional effects) has not been considered during the structural analysis by SAP2000 (2020) software introducing therefore a simplification in the model and, possibly, an underestimation of the internal forces in the upright, especially in the external ones (Bernuzzi and Simoncelli 2014). The warping properties of cross-sections were considered only on the post-processing phase during the structural members check. Based on the results:

- The most stressed members and corresponding failure mechanisms of AMSWs during low-to-moderate seismic actions have been obtained.
- Proposals to enhance the structural behavior of AMSWs during earthquake events in low-to-moderate seismicity regions have been made.

2 Design approach and methodology

Based on the design and experimental test reports of the AMSWs under investigation (received from the racking companies involved in the research), the structures have been modeled and analyzed using time-history analysis by direct integration with Hilber-Hughes-Taylor method (SAP2000 2020) including P-Delta effects, then calculations of each structural member's resistance and demand/capacity ratio have been performed, and finally the obtained failure mechanisms and their hierarchy have been discussed.

The seismic combination was calculated according to Eq. 1 (EN 16681 2016):

$$G_1 + G_2 + Q + E \quad (\text{EN 16681 (2016)}) \quad (1)$$

where “ G_1 ” identifies the permanent structural loads. “ G_2 ” represents the non-structural permanent loads (the weight of cladding panels). It is taken as 20 kg/m² for all the structures. “ Q ” identifies the unit loads. “ E ” represents the seismic action.

Table 1 shows the design input. All the data has been taken in accordance with EN 16681 (2016). The snow load has not been considered. Equal assumptions for the modeling of all the case studies have been done, and one of these assumptions was to neglect the influence of the supplemental damping induced by the pallet sliding. Different choice of damping coefficient can modify the values of the capacity/demand ratios, but since the

Table 1 Design information

Importance class	I (warehouses with fully automated storage operations)
Design life	50 years
Warehouse type	Automated warehouse
Unit load weight specifications	Shown in Table 3
R_f , rack filling reduction factor	Specified for each AMSW
Storage environment	Standard
Type of load made up accessory	Wooden pallet
Class of stored goods	A (compact constrained)
Seismic sway of the building	Non-sway
Structural damping	3%
Characterization of base anchoring slab	C30/37 with a thickness of 80 cm

main goal of the study is to evaluate the hierarchy of criticalities, the particular value of damping coefficient does not play the crucial role if it is taken equal for all of the considered structures.

The site is located in Montopoli (Italy) with coordinates of [43.67 °N, 10.74 °E] on a soil with $V_{s30} = 270$ m/s represented by soil type C in EN 1998-1 (European Committee for Standardization 2005). The seismic hazard and disaggregation computations have been performed with the OpenQuake (Pagani 2014) software, based on the SHARE (Giardini 2013) area source model and using the ground motion prediction equation proposed by Boore and Atkinson (2008). Average spectral acceleration, Avg_{SA} (0.3:0.1:3.0) is used, where 0.3:0.1:3.0 denotes a set of periods that discretizes the interval [0.3, 3.0] with a step of 0.1 s, to link the hazard and the selected record set. Following the Conditional Spectra record selection approach (Kohrangi et al. 2017), a set of 15 pairs of ground motions corresponding with the 10% probability of occurrence in 50-year return period were selected and scaled. The number of seismic records has been chosen considering a balance between accurate assessment of structural behavior and adequate computational time and burden. Figure 5a shows the disaggregation bar charts and Fig. 5b shows the target CS (i.e., mean ± 2 standard deviation) and the geometric mean of the 15 selected pairs of records. Table 2 shows the name and corresponding peak ground acceleration (PGA) for each accelerogram. In total 150 time history analyses have been performed (15 simulations for each 5 ARSW configurations in each principal direction—the cross-aisle and the down-aisle).

All considered structures are regular in plan, and the linear modal analysis of the whole 3D structure highlights that the behavior in the cross-aisle (CA) and down-aisle (DA) directions are practically independent. Therefore, 2D analyses have been carried out in each case separately in 2 directions (the DA and the CA). Moreover, there is a huge difference in terms of computational cost between 2 and 3D models and the increment of 3D model's precision is not expected to change the hierarchy of criticalities found within the research. The modal analysis started from a deformed condition obtained by the execution of a non-linear analysis with only vertical loads acting, including P-Delta effects. The linear elastic material behavior have been used for all the structural members. Regarding failure mechanisms, calculations of the resistance of each structural component have been performed. Based on them and applied forces it

Fig. 5 Seismic hazard data used in the analysis

(b) Selected records and the 2.5th/50th/97.5th percentiles using *AvgSA* as the conditioning *IM* corresponding to 10% in 50 years return period.

was possible to identify which failure mode will be critical for each structural member/connection. Table 3 provides pallet and unit load dimensions, and the maximum weights with a correspondence to the load levels for all the structures. These loads were modelled as a point load in the down-aisle direction (acting on the points of application—Z-shape rails) and as a distributed load in the cross-aisle direction (acting on Z-shape rails as well). For the seismic analysis, the rack was considered 100% loaded, which is the most critical loading condition during a seismic event according to EN 16681 (2016).

Table 2 Accelerograms' data

Name of accelerogram	Peak ground acceleration (g)
Hector Mine 1999 10 16 0946, Morongo Valley F.S	0.09
Northridge aftershock 01/17/94 CASTAIC—old ridge route	0.41
Santa Cruz MTNS 10/17/1989 lower crystal springs dam chan	0.11
Chi-Chi Aftershock 09/20/99 1803, TCU100	0.10
Loma Prieta 10/18/89 00:05, Dublin fire station	0.08
Chi-Chi 09/20/99, CHY029	0.13
Hector Mine SAT OCT 16, 1999 02:47, Newhall—country fire station	0.09
Northridge 1/17/94 12:31, Neenatch—Sacatara CK	0.07
Chi-Chi Aftershock 09/20/99 2146, TCU039	0.08
Landers 06/28/92 1158, LA—Obregon Park	0.06
Chi-Chi 09/20/99, HWA035	0.16
Chi-Chi 09/20/99, TTN010	0.05
Chi-Chi Aftershock 09/20/99 2146, CHY065	0.10
Manjil 06/20/90 2100, Manjil, Iran longitudinal comp	0.16
Chi-Chi 09/20/99, CHY046	0.09

Table 3 Pallet properties

Type	Pallet dimensions Width×Depth (mm)	Load dimensions Width×Depth×Height (mm)	Maximum weight (kg)	Load levels
1	1219×1050	1300×1100×2100	1000	1–2
2			800	3–5
3			600	6–9

AMSWs have been designed using seismic action reduction parameters for response spectrum analysis (q and K_d factors, where K_d is introduced to account for the dissipative phenomena typical of the dynamic behavior of racking structures under seismic actions), mass reduction by means of rack filling grade reduction factor (R_f) and the reduction of lateral stiffness in the cross-aisle direction to take into account realistic shear stiffness of the upright frame have been taken in accordance to EN 16681 (2016) (Table 4). Regarding behavior factors q for a low dissipative design, EN 16681 (2016) suggests a value up to 1.5 for frames with tension and compression diagonals, and a value up to 2 for frames with only active tension diagonals, provided that the design effect due to seismic action in all bracing members and their connections is increased by the value of q (valid for all types of bracing typologies in the CA direction and only for tension and compression diagonals without horizontal compression element bracing

Table 4 Design approaches for all structures

AMSW	Direction	Seismic action reduction	Unit load mass reduction	Lateral stiffness reduction
1	CA	−46% ($q=1.5$, $K_d=0.8$)	−20% ($R_F=0.8$)	Yes (axial spring)
	DA	−46% ($q=1.5$, $K_d=0.8$)	−20% ($R_F=0.8$)	–
2	CA	−46% ($q=1.5$, $K_d=0.8$)	−20% ($R_F=0.8$)	Yes (reduced area of diagonal's cross-section)
	DA	−60% ($q=2$, $K_d=0.8$)	−20% ($R_F=0.8$)	–
3	CA	−46% ($q=1.5$, $K_d=0.8$)	NO ($R_F=1$)	Yes (axial spring)
	DA	−60% ($q=2$, $K_d=0.8$)	−20% ($R_F=0.8$)	–
4	CA	−46% ($q=1.5$, $K_d=0.8$)	−20% ($R_F=0.8$)	No
	DA	−60% ($q=2$, $K_d=0.8$)	−20% ($R_F=0.8$)	–
5	CA	−34% ($q=1.5$, $K_d=1$)	−20% ($R_F=0.8$)	No
	DA	−34% ($q=1.5$, $K_d=1$)	−20% ($R_F=0.8$)	–

typology in the DA direction). Table 4 shows that in majority of cases, seismic action is reduced almost by half for response spectrum analysis and the reduction in the down-aisle (DA) direction is even larger than in the cross-aisle (CA) direction (AMSW types 2, 3 and 4), as allowed by the code. Since each racking company followed its own design practice, the values of coefficients vary between AMSWs. Since the Authors have performed time history analyses, q and K_d parameters have not been taken into account, but all other design assumptions (R_F , lateral stiffness reduction in the CA direction) are valid for time history analyses and reported for each AMSW in Table 4.

After structural analyses, resistance values for different buckling failure modes have been calculated for each cross-section. The lowest buckling resistance among all others corresponds to a critical buckling failure mechanism for each cross-section. Then, utilization factors (demand/capacity ratios) for each element with corresponding critical failure mode have been calculated using Matlab (2020). EN 1993-1-1-2005/Eurocode 3 part 1 (EN 1993-1-1 2005), EN 1993-1-3-2007/Eurocode 3 part 3 (EN 1993-1-3 2007), EN 15512/Specific regulations for steel racks (EN15512 2009), and EN 16681/Regulation for steel racks design in seismic situations (EN 16681 2016) have been used for the verification checks of elements. For the verification of the connections EN1993-1-8 (2005) has been used. Base plates with bonded anchors verifications have been done according to the EOTA TR29 (2010), EOTA TR45 (2013), and EOTA TR49 (2016). Each base plate resistance has been obtained in Hilti PROFIS Anchor software (Hilti 2020). Table 5 reports the material factors used in the checks in accordance with NTC (2018). For the consistency, the same material factors have been used for all structures.

Table 5 Material factors

Type of verification	Safety factor	Value
Resistance of cross-sections	γ_{m0}	1.05
Stability check of members	γ_{m1}	1.05
Connections checks	γ_{m2}	1.25

Fig. 6 Analysed AMSWs' frames

3 The geometry of structures

Figure 6 shows the analyzed frames in this article in the cross-aisle (CA) and the down-aisle (DA) directions.

Figure 7 shows the schematic plan view of the warehouse with the following distinction:

- “Pallet” frames, that support pallets and unit loads.

Fig. 7 Plan view of the warehouse

- “Tower” frames, that are located in the bracing towers.

Bracing towers are specific zones where the diagonal braces for the DA direction are located. In these zones, upright columns are different in terms of cross-sections and steel grades. “Aisle 1” and “Aisle 2” correspond to shuttle tunnels. Structural behavior in stiffness and resistance along the CA direction is mainly represented by “pallet” CA frames. Torsional effects due to the interaction between “pallet” and “tower” CA frames have not been considered significant since modal analysis of 3D structure revealed uncoupled modes in the CA and the DA directions. Therefore, it was decided to consider “pallet” CA frames in the analysis.

The numerical models have been calibrated using the following experimental campaign:

- Stub column tests on uprights (to evaluate effective area of the cross-section) in accordance with EN 15512, Annex A.2.1 (EN 15512 2009).
- Test for the shear stiffness of upright frames (to obtain realistic structural behavior in the CA direction) in accordance with EN 15512, Annex A.2.8 (EN 15512 2009).

Table 6 Structural geometries in the CA direction among all structures

AMSW	W_f (m)	W_s (m)	W_{sh} (m)	L_b (m)	W_b (m)	Total number of uprights	L (m)	Bracing type
1	1.44	1.32	1.70	15.27	0.50	48	64.99	“K-shaped”
2	1.20	1.15	1.70	15.27	0.50	56	64.98	“X-shaped”
3	1.20	1.15	1.70	15.27	0.38	56	64.86	“K-shaped” and “D-shaped” combined
4	1.20	1.15	1.70	15.27	0.50	56	64.98	“D-shaped”
5	1.20	1.15	1.70	15.27	0.50	56	64.98	“D-shaped”

Fig. 8 Non-dimensional geometry in the CA direction

- Bending tests on beam end connector (to evaluate its rotational stiffness) in accordance with EN 15512, Annex A.2.4 (EN 15512 2009).
- Tests on floor connections (to evaluate their rotational stiffness in the DA direction) in accordance with EN 15512, Annex A.2.7 (EN 15512 2009).

The main structural dimensions in the CA direction for all the structures are presented in Table 6 with reference to Fig. 8, where:

- W_f is single frame width.
- W_s is the spacer length between single frames.
- W_{sh} is the width of the shuttle tunnel.
- L_b is the width of one block.
- W_b is the space between two blocks in the middle.
- L is the overall width of the frame in the CA direction as a distance between end uprights' back.

Regarding the DA direction, the main structural dimensions for all the structures are presented in Table 7 with reference to Fig. 9, where:

Table 7 Structural geometries in the DA direction among all structures

AMSW	W_{br} (m)	W_p (m)	Total number of uprights	L (m)	Total amount of bracing towers
1	3.12	1.54	45	71.04	2
2	3.00	1.58	45	72.19	2
3	3.04	1.52	47	76.18	4
4	3.10	1.51	45	69.80	2
5	2.50	1.57	45	71.05	2

Fig. 9 Non-dimensional geometry in the DA direction**Table 8** Load level heights among all structures

LL	Height (m)				
	AMSW-1	AMSW-2	AMSW-3	AMSW-4	AMSW-5
1	0.55	0.28	0.33	0.55	0.55
2	3.17	2.93	3.00	3.22	3.22
3	5.80	5.59	5.67	5.89	5.89
4	8.42	8.24	8.34	8.56	8.56
5	11.05	10.89	11.01	11.23	11.23
6	13.67	13.52	13.68	13.90	13.90
7	16.30	16.17	16.35	16.57	16.57
8	18.92	18.83	19.02	19.24	19.24
9	21.55	21.48	21.69	21.91	21.91

- W_{br} is a width between upright's axis in the bracing tower.
- W_p is a width between pallet upright's axis
- L is the overall width of the frame in the DA direction as a distance between end uprights' back.

In all the structures “X-shaped” bracing configuration was used in bracing towers. “L” indicates a load level, where the unit load is stored (Fig. 9). Table 8 shows the height of each load level among all structures. The floor zero matches the top of the concrete base slab, level height does not account for satellite rails height.

Table 9 presents the structural heights of all the analysed warehouses, in particular the height of the structure under roof truss (H, Figs. 8, 9) and its overall height (H tot, Figs. 8, 9).

Table 10 presents elements’ connection properties. Their typology and stiffness have been taken in accordance with laboratory tests. The distinction is made for “pallet” upright columns, which support pallets and unit loads and “tower” upright columns that are located in bracing towers. The distinction is also made for the CA and the DA directions of the warehouse. The connections have all been represented by linear springs, if they could be classified as “semi-rigid” following the indications of EN1993-1-8 (2005), or as perfect hinges in the case they could be classified as “hinges” following the same code. Therefore, the hysteretic behavior of these connections was neglected but, considering the very limited amount of energy dissipated by these elements, this simplification is not expected to significantly modify the response of the analysed buildings. The non-dimensional rotational and axial stiffnesses of all semi-rigid connections is present in Table 11, where “k” represents the lowest rotational/axial stiffness value among all case studies for a given element and 0 is assigned to pinned connections. Eccentricities of the lacings in the CA direction and bracings in the bracing towers in the DA directions have not been considered in the structural analysis. These connections are schematically shown in Fig. 10 for the CA frames and in Fig. 11 for the DA frames. In AMSW-3, additional base connections have been used (Fig. 12), which were modelled as fixed according to the racking company’s design practice (modelling such connections as fixed, which is not a common assumption in this kind of structures, is a consequence of a particular connection’s development of racking company that cannot be discussed in detail due to the non-disclosure agreement).

In the DA model both compression and tension diagonals have been simulated. Taking into account that diagonal in compression is going to buckle, only the half of cross-section area has been used to reduce the stiffness of the diagonal members. For the checks, the axial forces acting in the diagonals were doubled, as the one in compression is going to buckle. Therefore, only tensile forces have been considered in the diagonals check. Figure 13 shows the cross-section shapes and characteristic for upright columns, diagonals and pallet beams.

All cold-formed cross-sections have been non-dimensionalised due to a non-disclosure agreement with racking companies. All the upright columns have a Ω -shape cross-section, however for beams and diagonals both U-shape and Boxed C-shape sections have been used (Fig. 13). Parameters for all these shapes are presented in Tables 12, 13, 14, 15, 16, and 17 in terms of:

Table 9 Structural heights for all structures

AMSW	H (m)	H tot (m)
1	24.00	25.88
2	24.27	26.41
3	24.35	26.31
4	24.35	26.00
5	24.35	26.90

Table 10 Elements' connections

Connection location	Direction	AMSW-1	AMSW-2	AMSW-3	AMSW-4	AMSW-5
“Pallet” upright columns’ base	CA	Pinned in all structures				
	DA	Semi-rigid	Semi-rigid		Pinned	Semi-rigid
	DA	Semi-rigid	Pinned		Pinned	Semi-rigid
	All	Continuous from the base to the top, pinned at the roof beam				
Upright column						
Diagonal-to-upright column	CA	Pinned with an axial spring	Pinned	Pinned with an axial spring	Pinned	Pinned
Horizontal-to-upright column	CA	Pinned in all structures				
Diagonal-to-upright column	DA	Pinned in all structures				
Beam-to- “bracing” upright column	DA	Semi-rigid	Pinned	Pinned	Pinned	Semi-rigid
Beam-to- “pallet” upright column	DA	Semi-rigid	Semi-rigid	Pinned	Semi-rigid	Semi-rigid

Table 11 Adimensional rotational/axial stiffnesses of connections

Connection location	Type of stiffness	AMSW-1	AMSW-2	AMSW-3	AMSW-4	AMSW-5
“Pallet” upright columns’ base (DA)	Rotational	1.11 k	2.03 k	0	0	k
“Tower” upright columns’ base (DA)	Rotational	1.11 k	0	0	0	k
Diagonal-to-upright column (CA)	Axial	k	0	2.17 k	0	0
Beam-to- “bracing” upright column (DA)	Rotational	k	0	0	0	2.41 k
Beam-to- “pallet” upright column (DA)	Rotational	k	1.15 k	0	5.25 k	3.28 k

a AMSW-1 and AMSW-3 **b** AMSW-2 **c** AMSW-4 and AMSW-5**Fig. 10** Elements’ connections in CA direction among all the structures **a** AMSW-1 **b** AMSW-2 **c** AMSW-3 **d** AMSW-4 **e** AMSW-5**Fig. 11** Elements’ connections in DA direction among all the structures

Fig. 12 Particular base connection in AMSW-3 in the DA direction

Fig. 13 Cross-sections' shapes characteristics of upright columns, beams and diagonals

Table 12 Key geometric parameters of “pallet: upright columns’ cross-section

	AMSW-1		AMSW-2		AMSW-3		AMSW-4		AMSW-5
	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	
Material	S350 GD	S350 GD	S420 MC	S420 MC	S355	S355	S355 MC	S500 MC	S500 MC
A	50.00	33.33	30.00	30.00	29.20	33.20	50.50	40.80	25.20
B	60.00	40.00	60.00	60.00	40.00	40.00	34.50	49.28	23.00
A_{eff}/A	0.81	0.86	0.82	0.85	0.92	0.80	0.90	0.83	0.95
I_z/I_y	2.27	2.19	5.18	4.97	2.15	1.76	0.39	1.35	1.12
ρ_z/ρ_y	1.51	1.48	2.28	2.23	1.46	1.32	0.62	1.16	1.06
$W_z/W_{y, left}$	1.17	1.19	2.10	1.92	1.33	1.28	0.44	0.99	1.02
$W_z/W_{y, right}$	2.32	2.32	2.78	2.78			0.79	1.28	1.43
y_0/d	2.28	2.31	1.83	1.75	2.08	2.11	2.12	2.15	2.20

- The steel grade.
- The ratio between the second moments of area (I_z/I_y).
- The section moduli (W_y/W_z).
- Radii of gyration (ρ_y/ρ_z).

Table 13 Key geometric parameters of “tower” upright columns’ cross-section

	AMSW-1			AMSW-2		AMSW-3	AMSW-4	AMSW-5
	Upper	Lower	Lower reinforced	Upper	Lower			
Material	S350 GD	S350 GD	S350 GD	S420 MC	S420 MC	S355	S500 MC	S500MC
A	48.00	30.00	15.00	30.00	50.00	35.00	28.57	49.00
B	58.00	36.25	18.13	60.00	60.00	35.00	40.00	23.00
A_{eff}/A	0.75	0.85	0.87	0.82	0.93	0.91	0.78	1.00
I_z/I_y	1.04	1.02	1.33	5.18	1.47	1.24	1.66	0.37
ρ_z/ρ_y	1.02	1.01	1.16	2.28	1.21	1.11	1.29	0.61
W_z/W_y , left	0.94	0.90	0.93	2.10	0.98	1.08	1.06	0.83
W_z/W_y , right	1.32	1.32	1.79	2.78	1.35	1.08	1.35	0.82
y_0/d	2.29	2.28	1.90	1.83	1.92	2.10	2.16	1.50

Table 14 Key geometric parameters of diagonals in CA direction cross-section

	AMSW-1		AMSW-2	AMSW-3	AMSW-4		AMSW-5
	Upper	Lower			Upper	Lower	
Material	S350 GD	S350 GD	S280 GD	S250	S250 GD	S355 MC	S275 JR
Cross-section	U-shape	U-shape	CHS 30×1	U-shape	U-shape	U-shape	SHS 50×2
A	42.67	32.00		26.67	46.67	16.00	
B	33.33	25.00		26.67	20.00	10.00	
I_z/I_y	2.12	2.14		1.85	8.52	3.22	
ρ_z/ρ_y	1.46	1.46		1.36	2.92	1.79	
W_z/W_y , up	1.24	1.27		1.73	1.87	1.51	
W_z/W_y , bottom	2.00	2.06		1.73	5.43	2.51	
y_0/d	2.28	2.25		1.49	1.79	2.11	

In several cases hot-rolled profiles have been used in the diagonals and beams. In these situations, only cross-section names are gathered, since their mechanical characteristics are well known. In case of upright columns also following parameters are given:

- The ratio between effective and gross area (A_{eff}/A)
- The y_0/d ratio, i. e. the distance between the shear centre and centroid (y_0) and the distance between the shear centre and the web (d). This parameter is also indicated in U-shape diagonals.

For upright columns and diagonals, the distinction has been made for upper and lower parts of the structure to obtain more efficient use of a material. For upright columns’, diagonals’, and beams’ cross-section parameters following tables are shown:

Table 15 Key geometric parameters of diagonals in DA direction cross-section

	AMSW-1		AMSW-2	AMSW-3		AMSW-4	AMSW-5
	Upper	Lower		Upper	Lower		
Material	S355 JR	S355JR	S280 GD	S235	S355	S355 MC	S275 JR
Cross-section	L66×33×3	L66×33×4	U-shape	L70×70×6	L80×80×8	Railing Sect. 71×5 (×2)	RHS 140×80×10
A			35.00				
B			17.50				
I_z/I_y			3.76				
ρ_z/ρ_y			1.94				
W_z/W_y , up			1.57				
W_z/W_y , bottom			2.20				
y_0/d			1.31				

Table 16 Key geometric parameters of pallet beams cross-section

	AMSW-1	AMSW-2	AMSW-3	AMSW-4			AMSW-5
				LLs1-2	LLs3-5	LLs6-9	
Material	S355 JR	S275 JR	S350	S355	S275 JR	S275JR	S275 JR
Cross-section	U-shape	Boxed C-shape	Boxed C-shape	Boxed C-shape	Boxed C-shape		
A	33.33	60.00	60.00	100.00	53.33	40.00	75.00
B	13.33	30.00	20.00	33.33	33.33	33.33	25.00
I_z/I_y	8.12	3.91	13.15	7.60	2.16	1.38	8.24
ρ_z/ρ_y	2.85	1.98	3.63	2.76	1.47	1.17	2.87
W_z/W_y , up	2.29	1.95	5.73	2.63	1.31	1.15	2.77
W_z/W_y , bottom	4.43						

Table 17 Key geometric parameters of bracing beams cross-section

	AMSW-1	AMSW-2	AMSW-3	AMSW-4	AMSW-5
Material	S275 JR	S280 GD	S235	S355 MC	S275 JR
Cross-section	HEA100	U-shape	SHS 90×3	U-shape	Boxed C-shape
A		50.00		22.22	75.00
B		30.00		13.89	25.00
I_z/I_y		4.47		3.09	8.24
ρ_z/ρ_y		2.11		1.76	2.87
W_z/W_y , up		1.17		1.46	2.77
W_z/W_y , bottom		2.92		2.40	

- Table 12 indicates parameters for “pallet” upright columns, that support pallets and unit loads.
- Table 13 shows parameters for “tower” upright columns that are located in bracing towers.
- Table 14 presents parameters for diagonals in the CA direction.
- Table 15 gathers parameters for the diagonals in the bracing towers in the DA direction.
- Table 16 presents parameters for “pallet” beams that support pallets and unit loads. In AMSW-4 different beams’ cross-sections were used for the different load levels (LLs).
- Table 17 indicates parameters for bracing beams located in bracing towers.

Figure 14 shows non-dimensional (not in scale) parameters about the shape and characteristics of the base connections, where “t” is the thickness of the base plate. All the geometric parameters and characteristics of anchors for upright columns’ base plates are reported in following tables:

- Table 18 shows geometric parameters and anchors characteristics for “pallet” upright columns’ base plates.
- Table 19 gathers geometric parameters and anchors characteristics for “tower” upright columns’ base plates.
- Table 20 presents geometric parameters and anchors characteristic for bracing diagonals’ base plates in the DA direction. This information is gathered only for AMSW types 2, 4, and 5, where these base connections were used.

Equation 2 (EN 16681 2016) indicates the combination of vertical loads used to define the seismic mass W_E in both the CA and the DA directions:

$$W_e = G_1 + G_2 + R_f \cdot Q \quad (\text{EN 16681 (2016)}) \quad (2)$$

Fig. 14 The base plate connection geometry (t: thickness of the base plate)

Table 18 “Pallet” upright columns’ base plates parameters

	AMSW-1	AMSW-2	AMSW-3	AMSW-4	AMSW-5
A	20.00	16.67	16.67	20.53	16.40
B	13.33	10.00	10.00	13.33	14.40
C	13.33	13.33	13.33	14.67	6.40
D	—	—	—	—	10.80
Anchors	Post-installed, 2M16 C8.8, L= 150 mm	Post-installed, 2M16 C8.8, L= 200 mm	Post-installed, 2M16 C8.8, L= 120 mm with gap filling	Post-installed, 2M20 C8.8, L= 92 mm	Post-installed, 6M12 C8.8, L= 240 mm

Table 19 “Tower” upright columns’ base plates parameters

	AMSW-1	AMSW-2	AMSW-3	AMSW-4	AMSW-5
A	61.67	16.67	13.20	15.40	16.40
B	23.33	16.67	13.20	10.00	14.40
C	18.33	13.33	10.00	11.00	6.40
D	16.67	13.33	10.00	–	10.80
Anchors	Post-installed, 8M24 C8.8, L= 375 mm	Post-installed, 4M16 C8.8, L= 300 mm	Post-installed, 4M24 C5.8, L= 400 mm with gap filling	Post-installed, 2M20 C8.8, L= 372 mm	Post-installed, 6M12 C8.8, L= 240 mm

Table 20 “Diagonal bracings” base plates parameters

	AMSW-2	AMSW-4	AMSW-5
A	18.50	50.00	8.80
B	12.50	10.00	8.80
C	7.50	37.50	5.60
D	10.00	–	5.60
Anchors	Post-installed 6M16 C8.8, L = 200 mm	Post-installed 2M20 C8.8, L = 249 mm	Post-installed 4M24 C8.8, L = 110 mm

Table 21 Structural weight (kN)

	CA				DA			
	G1 (kN)	G2 (kN)	Q*R _f (kN)	W _e (kN)	G1 (kN)	G2 (kN)	Q*R _f (kN)	W _e (kN)
AMSW-1	152.47	17.60	2830.00	3000.07	116.37	33.03	2285.00	2434.40
AMSW-2	163.70	19.35	2830.00	3013.05	97.05	17.03		2399.08
AMSW-3	163.70	19.80	3537.00	3720.50	153.00	17.86		2455.86
AMSW-4	171.00	19.66	2830.00	3020.66	101.26	17.80		2404.06
AMSW-5	315.00	19.53	2830.00	3164.53	292.94	16.66		2594.60

Table 22 Contribution of each load in a total weight of the structure

	CA			DA		
	G1 (%)	G2 (%)	Q*R _f (%)	G1 (%)	G2 (%)	Q*R _f (%)
AMSW-1	5.08	0.59	94.33	4.78	1.36	93.86
AMSW-2	5.43	0.64	93.92	4.05	0.71	95.24
AMSW-3	4.40	0.53	95.07	6.23	0.73	93.04
AMSW-4	5.66	0.65	93.69	4.21	0.74	95.05
AMSW-5	9.95	0.62	89.43	11.29	0.64	88.07

(EN 16681 2016) where G_1 identifies the structural permanent loads. G_2 represents the non-structural permanent loads. Q identifies the unit loads. $R_f=0.8$ in both the CA and the DA direction for all the structures, except AMSW-3 in the CA direction (Table 4).

Table 21 presents the information about seismic mass W_E and all its components from Eq. 2 (EN 16681 2016) for each AMSW. Distinction is made for the CA and the DA directions. The unit loads provide significant contribution to the total seismic mass of the structures. Table 22 shows, that in all the structures unit loads provide around 90% of the total seismic mass of the considered AMSWs.

Table 23 Relevant modes and corresponding participant mass for all the structures

AMSW	Main direction	Period (s)	Participant mass (%)	Mode
1	CA	1.78	67	1
	DA	1.85	60	1
	DA	0.67	19	2
	CA	0.60	16	3
2	CA	1.78	67	1
	DA	1.87	61	1
	DA	0.60	18	2
	CA	0.58	15	3
3	CA	1.60	62	1
	DA	0.90	57	1
	DA	0.39	19	2
	CA	0.32	23	3
4	CA	1.22	59	1
	DA	1.82	57	1
	DA	0.30	20	2
	CA	0.53	21	5
5	CA	0.97	62	1
	DA	1.42	55	1
	DA	0.35	17	2
	CA	0.23	18	3

Table 24 Critical load multipliers

AMSW	Main direction	Critical load multiplier
1	CA	8.91
	DA	11.64
2	CA	10.07
	DA	12.20
3	CA	13.80
	DA	6.90
4	CA	9.78
	DA	11.88
5	CA	38.52
	DA	12.45

4 Results

4.1 Modal analysis

Regarding modal analysis, 12 modes were sufficient to include in both the CA and the DA directions for all the structures to obtain a cumulative mass percentage bigger than 90%, which is required by EN 1998-1 (Kilar et al. 2011). Table 23 illustrates the relevant modes,

Fig. 15 AMSW-1 CA Mode 1 deformed shape

Fig. 16 AMSW-1 CA Mode 3 deformed shape

Fig. 17 AMSW-1 DA Mode 1 deformed shape

Fig. 18 AMSW-1 DA Mode 2 deformed shape

corresponding period, and participant mass for considered structures. In AMSW types 1, 2 and 3, where lateral stiffness reduction has been taken into account to use the realistic shear stiffness of upright frames (Table 4), fundamental structural periods in the CA direction were significantly higher than in AMSW types 4 and 5. Table 24 shows critical load multipliers for gravity load SLS combination.

Deformed shapes obtained during modal analysis were similar for all the structures (Figs. 15, 16, 17, 18). Therefore, they are graphically represented only for AMSW-1. Two translational modes have been considered as relevant due to the high mass participation percentage in both directions.

Fig. 19 CA axial force diagram (The time step when maximum axial forces acting on upright columns during time history 1 for the AMSW-1 is presented)

Table 25 Demand/capacity ratios for the critical elements in the CA directions

AMSW	Upright	Diagonal to upright connection	Diagonal	Upright's base connection
1	1.09	0.47	0.52	1.01
2	0.71	0.37	0.86	0.82
3	1.21	0.57	0.58	2.01
4	1.14	0.54	0.91	0.74
5	0.40	0.53	0.31	0.27

Fig. 20 Variability of utilization factor in the CA direction among all structures

4.2 Internal actions in the CA direction

Taking into consideration that among all the structures different cross-sections, bracing schemes and connections have been used, results and conclusions for each AMSW slightly vary. However, several similarities in terms of structural behavior of AMSWs among all structures have been found. They are presented separately for the CA and the DA directions. As shown in Fig. 19, in the CA direction axial forces acting in the bottom part of the frame were significantly higher than in the top part of the frame.

The maximum demand/capacity ratios for each critical structural member as the average of 15 time histories are shown in Table 25. Figure 20 shows the variability of normalized utilization factor (demand/capacity ratio) for each member in the CA direction among 15 earthquake records (the values are normalized by the most stressed member):

- The value “1” represents that the member has the highest demand/capacity ratio among all others. All other member’s utilization factors normalized to this one.
- The “red line” identifies the most frequent value along all the 15 time histories for each member.
- “Rectangle with blue outline” shows a range of other frequent utilization factors obtained.
- “The red cross” indicates unitary occasions which are out of the range of frequent values obtained.

These graphs show only the location of criticality and correspond to the most critical failure mode in it (“Diagonal” corresponds to the biggest utilization factor between diagonal member and diagonal-to-upright connection). When different sections have been used in the bottom and upper parts of a structure, the same distinction has been applied in upright’s and diagonal’s labels.

Figure 21 indicates the most stressed elements in the CA direction, which presents the variability of the most frequent utilization factor among 5 considered structures (in this case, utilization factor also means normalized value, not an absolute one). The most stressed members are in the following order: upright column’s base plate, upright column, diagonal member, and horizontal members (almost unstressed therefore not critical during the excitation). Regarding the diagonals and upright columns, high variability has been observed among structures. Therefore, assessment of each critical member in each AMSW and its failure mode has been performed to evaluate the development of different energy dissipation mechanisms. Overall, the system of upright frames connected by diagonal bracing acted as a main lateral force resisting system in the CA direction as also indicated by EN 16681 (2016).

4.3 Internal actions in the DA direction

Comparing results in the CA and the DA directions, it has been observed that “pallet” upright columns were much less stressed in the DA direction. They do not play as important role in the earthquake resistance of AMSW in the DA direction as in the CA direction. Figure 22 illustrates this phenomenon for AMSW-1, but the same observation has been

a Variability of utilization factor in the CA direction for AMSW-1.

b Variability of utilization factor in the CA direction for AMSW-2.

c Variability of utilization factor in the CA direction for AMSW-3.

d Variability of utilization factor in the CA direction for AMSW-4.

e Variability of utilization factor in the CA direction for AMSW-5.

Fig. 20 (continued)

made for all other structures. Even the variability of axial force is much lower in “pallet” upright columns in the DA direction since they are mainly stressed by vertical loads that are the same for all the time histories. Therefore, it can be concluded that “pallet” upright columns were mainly stressed in the CA direction during the earthquake excitation and “tower” upright columns were mainly stressed in the DA direction. Higher axial forces in absolute values acted on “tower” upright columns, therefore the use of larger cross-sections in “tower” upright columns comparing with “pallet” ones was reasonable.

The same procedure of evaluating the most stressed members as in the CA direction has been applied also for the DA direction. The maximum demand/capacity ratios for each critical structural member as the average of 15 time histories are shown in Table 26. The variability of normalized utilization factor for each member in the DA direction among 15 earthquake records is gathered in Fig. 23. These graphs show only the location of criticality for the most critical failure mode in it (“Diagonal” corresponds to the biggest utilization

Fig. 21 The most stressed members in the CA direction

Fig. 22 Comparison of axial forces acting in “pallet” upright columns in the CA and the DA directions

Table 26 Demand/capacity ratios of the most critical structural members in the DA direction

	AMSW	Upright	Diagonal to upright connection	Diagonal	Upright's base connection
1		1.10	1.43	1.28	0.82
2		1.28	1.37	1.15	1.41
3		1.00	0.72	0.48	0.89
4		2.60	2.02	0.78	2.52
5		0.43	1.30	0.12	3.02

Fig. 23 Variability of utilization factor in DA direction among all structures

factor between diagonal member and diagonal-to-upright connection). “Pallet” upright columns and their base plates are not shown, because their utilization factors were relatively low in all the cases (lower than 0.20). The same governs for beams in bracing towers, their utilization factor was higher than 0.20 only in AMSW-4 (Fig. 23d). Utilization factors for diagonals’ base plates are presented in cases when they have been used in the AMSW.

Figure 24 represents the variability of normalized utilization factors among 5 considered structures. It was concluded that in the DA direction the stiff bracing towers resisted seismic forces almost entirely, leaving the “pallet” upright columns and pallet beams unstressed in all the structures. This is also visible from axial forces diagram (Fig. 25). Among all the members in a bracing tower, the bracing beam was the least stressed. Regarding diagonals and bracing upright columns, high variability among structures has been observed. Therefore, assessment of each critical member in each AMSW and its failure mode has been performed to evaluate the development of different energy dissipation mechanisms.

5 Assessment of AMSW’s components prone to failure

This section compares the results of all the analyzed structures, focusing on each critical member such as diagonals, diagonal-to-upright connections, uprights and their base connections.

5.1 Diagonals

The hierarchy of criticalities has been obtained between diagonal-to-upright connection and diagonal member to understand the failure modes of the diagonal-to-upright connection in each AMSW and their relation to the buckling/yielding of the diagonal member. The hierarchy of criticalities reported in Tables 27 and 28 shows the failure modes of the diagonal-to-upright connections in each AMSW and their relation to the buckling/yielding of the diagonal member.

In AMSW types 1, 3 and 4 the most critical buckling type of the CA diagonals was the flexural–torsional buckling (Table 27) since the shear centre was not coincident with the centroid of cross-section in these cases (Fig. 13, Table 14). Therefore, warping effects have a big influence on the performance capacity of diagonals. However, in AMSW types 2 and 5 where CHS and SHS cross-sections have been used, the most critical failure mode in diagonal members has been flexural buckling. Only in AMSW-5, connection failure happened before buckling of the diagonal. Bearing failure–ovalization in the diagonal side has been the most critical failure mode in the diagonal-to-upright connection. In all the considered connections bolts had 2 shear planes.

Considering that in the DA direction diagonals have been checked only in tension, the most critical failure mode for all the structures was yielding (Table 28). Diagonal-to-upright connection failure preceded yielding of the diagonal member due to tensile forces in all the cases. This means that vertical bracing’s diagonals in the DA direction cannot be considered as an energy dissipation zones in given structures.

a Variability of utilization factor in the DA direction for AMSW-1.

b Variability of utilization factor in the DA direction for AMSW-2.

c Variability of utilization factor in the DA direction for AMSW-3.

d Variability of utilization factor in the DA direction for AMSW-4.

e Variability of utilization factor in the DA direction for AMSW-5.

Fig. 23 (continued)

5.2 Upright columns

Upright columns played a crucial role in both the CA and the DA directions. Flexural–torsional buckling mode was the most critical for all the cross-sections (due to shear center eccentricity, Fig. 13a). Distortional buckling failure mode has not been considered due to the absence of tests performed on real-height cross-sections. In all the case studies, where distortional buckling tests have been performed on uprights (Table 29), its effect on effective cross-section area has been determined according to the Sect. 9.7.2 of EN 15512 (2009). The calculations revealed that flexural–torsional buckling was more critical than distortional buckling in these case studies. Furthermore, since distortional buckling is controlled by bracing pattern of the upright frames that has a high density in AMSWs, flexural–torsional buckling was assumed to be the most critical. Table 29 shows the following parameters that are relevant to compare and possibly optimize upright columns’

Fig. 24 The most stressed members in the DA direction among all structures

Fig. 25 DA axial force diagram (The time step when maximum axial forces acting on upright columns during time history 1 for the AMSW-1 is presented)

Table 27 Diagonals criticalities for the CA direction

AMSW	Critical failure mechanism of diagonal member	Critical failure mechanism of connection	First to be failed
1	Flexural–torsional buckling	Bearing failure	Diagonal member
2	Flexural buckling	Bearing failure	Diagonal member
3	Flexural–torsional buckling	Bearing failure	Diagonal member
4	Flexural–torsional buckling	Bearing failure	Diagonal member
5	Flexural buckling	Bearing failure	Connection

Table 28 Diagonals criticalities for the DA direction

AMSW	Critical failure mechanism of diagonal member	Critical failure mechanism of connection	First to be failed
1	Yielding	Net section failure	Connection
2	Yielding	Net section failure	Connection
3	Yielding	Bearing failure	Connection
4	Yielding	Bearing failure	Connection
5	Yielding	Bolt shear failure	Connection

performance (only upright columns in lower parts of the frames are considered since they were the most stressed ones):

- Steel grades.
- Availability of experimental distortional buckling tests.
- The effective cross-section areas A_{eff} normalized to the cross-section with the lowest value of it among all the structures (A).
- The slenderness of the profile, which corresponds to the most critical buckling mode. Here reported in the non-dimensional form.
- The ratio between the effective and the gross area (to measure the current effectiveness of section).
- The effective (or elastic) section modulus W_{el} normalized to the cross-section with the lowest value of it among all the structures (W).

It is concluded that:

- All the cross-sections have good performances in terms of $A_{\text{eff}}/A_{\text{tot}}$ ratio.
- Non-dimensional slenderness is high in comparison with columns in typical steel structures, especially in a AMSW-4.

“Pallet” upright columns were mainly stressed in the CA direction (Fig. 22). Therefore, the check on “pallet” uprights in the CA direction have been performed only based on forces acting in the CA direction i.e., without taking 30% contribution from the DA direction into account. Demand/capacity ratios for the most stressed “pallet” upright columns in the CA direction among all structures are reported (Fig. 26a). Average value among 15 time histories is presented for each AMSW. The lowest Demand/Capacity ratio has been obtained in AMSW-5, where cross-section with the largest area and highest steel grade has been used compared to other cases (Table 29). Only in AMSW types 2 and 5 upright columns failed after a failure in the diagonal (Fig. 20b), which is favourable condition during the earthquake excitation for the development of energy dissipation mechanisms. Furthermore, in AMSW types 2 and 5 average utilization factor was lower than 1 which shows that uprights are unlikely to buckle during the earthquake excitation in these warehouses. Regarding bending moments, following conclusions have been made:

- Bending moments acting on upright columns in the CA direction were very low, comparing to typical buildings. Their average values among all time histories were up to 4 kNm

Table 29 Upright columns' characteristics comparison

AMSW	Upright column	Steel grade	Experimental tests	A_{eff}	λ_{max}	A_{eff}/A_{tot}	W_{el}
1	"Tower" reinforced	S350GD	NO, data extrapolated from tests on similar sections	$4.67 \times A$	0.93	0.87	$8.3 \times 9 W$
	"Tower" not reinforced	S350GD	NO, data extrapolated from tests on similar sections	$2.28 \times A$	0.83	0.85	$4.07 \times W$
	"Pallet"	S350GD	NO, data extrapolated from tests on similar sections	$1.37 \times A$	0.85	0.86	$2.14 \times W$
2	"Tower"	S420MC	NO	$2.55 \times A$	1.41	0.93	$2.88 \times W$
	"Pallet"	S420MC	NO	$1.28 \times A$	1.03	0.85	$1.58 \times W$
3	"Tower"	S355	NO, data extrapolated from tests on similar sections	$2.71 \times A$	0.97	0.92	$3.64 \times W$
	"Pallet"	S355	NO, data extrapolated from tests on similar sections	$1.00 \times A$	1.40	0.8	$1.00 \times W$
4	"Tower"	S500MC	NO	$1.76 \times A$	1.42	0.78	$2.26 \times W$
	"Pallet"	S500MC	NO	$1.35 \times A$	1.44	0.83	$1.35 \times W$
5	"Tower"	S500MC	NO	$5.03 \times A$	0.93	1.00	$10.19 \times W$
	"Pallet"	S500MC	NO	$2.91 \times A$	1.04	0.95	$4.65 \times W$

Fig. 26 Assessment on the behavior of "pallet" upright columns in the CA direction among all structures

Fig. 27 AMSW-1 CA moment participation to upright check

- Figure 26b shows that upright columns' cross-sections had a very limited bending resistance for AMSW types 1, 2, and 3 (considering upright columns in the bottom part of the structure, where the highest bending moments were acting).
- In AMSW-5 upright columns' section modulus was much higher, comparing to others (Table 29). Consequently, its bending capacity was significantly higher.

The overall resistance of upright columns is a combination of its moment and axial resistances. Therefore, the discussion of moment participation in overall upright columns' resistance has been done. Figure 27 presents the results for AMSW-1 among all the time histories. Values of moment participation in overall upright resistance varied from one AMSW to another but in AMSW types 1, 2, and 3 for all time histories they were higher than design ones. For example, 20% moment participation means that 20% of overall demand/capacity ratio for the particular upright comes from bending moment.

Fig. 28 Assessment on the behavior of "tower" upright columns in the DA direction among all structures

Moment participation can be significant in the CA direction during the earthquake event and its influence in upright check can be up to 30%.

Regarding the DA direction, Fig. 28a reports the demand/capacity ratios for the most stressed "tower" upright columns among all structures. Results are presented in the DA direction where "tower" upright columns were mainly stressed. Regarding 30% contribution from the CA direction, it has not been considered for "tower" uprights since only "pallet" CA frames have been considered for the 2D analysis in the CA direction i.e., internal actions for "tower" uprights in the CA direction have not been calculated. Therefore, it can be assumed that actions in the "tower" upright in the DA direction have been slightly underestimated during the design checks, but it is not changing the main conclusions of the research and hierarchy of failures since they were already one of the most stressed members. The lowest Demand/Capacity ratios have been obtained in AMSW-5, where the used upright column's cross-section had larger area than in all others structures (Table 29). In AMSW types 1, 2 and 5 failure of the diagonals preceded the buckling of the upright column (Fig. 23a, b, e), which is favourable condition during the earthquake excitation for the development of energy dissipation mechanisms. Furthermore, in AMSW-5, the upright column's section modulus was bigger compared to the others (Table 29), and its bending capacity was significantly higher as a consequence (Fig. 28b). In the DA direction, moment participation in the upright columns has been checked. Figure 29 shows the results for AMSW-1, which are similar for all other structures. It has been concluded that:

- Moment participation did not play a significant role in the "tower" upright columns' check.
- Bending moments acting on "tower" upright columns in the DA direction were very low, comparing with typical buildings.
- Upright columns' cross-sections have been much stronger in "tower" upright columns comparing to "pallet" ones (Table 29).

Fig. 29 AMSW-1 DA moment participation to upright check

a Participation of axial and shear forces in "pallet" upright columns' base connection check in the CA direction

b Combined pullout-concrete cone failure capacity in "pallet" upright columns' base connections.

Fig. 30 Assessment on the behavior of "pallet" upright columns' base connections in CA direction among all structures

5.3 Upright columns' base connections

Upright columns' base connections played a crucial role in both the CA and the DA directions. They were subjected to mainly axial and shear forces, meanwhile bending participation was limited. Therefore, the combined axial and shear forces check has been performed. The failure mechanisms were brittle, and the most common failure mode was a concrete cone and pull-out failure due to axial forces combined with anchor failure in shear. "Pallet" upright columns were mainly stressed in the CA direction (Fig. 22), meanwhile "tower" upright columns were mainly stressed in the DA direction. Since failure of the base connections was induced by the combined action of axial and shear forces, assessment on participation of each force has been done in a similar way as it has been done for upright columns. In the CA direction for "pallet" uprights, the results are shown in Fig. 30a. In all the cases participation of the shear force was

a Participation of axial and shear forces in "tower" upright columns' base connection check in the DA direction

b Combined pullout-concrete cone failure capacity in "tower" upright columns' base connections.

Fig. 31 Participation of forces in base plate checks in the DA

lower than 20%. In the base connections check, the axial forces were dominant, and the combined pullout-concrete cone was the most critical failure mode. Figure 30b shows the comparison of combined pullout-concrete cone failure capacity among all the structures. Following conclusions have been obtained:

- In AMSW types 1 and 3 the brittle failure of the upright columns' base connections preceded the failure in the diagonals (Fig. 20a, c).
- In AMSW types 2, 4, and 5 the failure in the diagonals preceded the brittle failure of the upright columns' base connections (Fig. 20b, d, e), which is a favorable condition for the development of energy dissipation mechanism.
- AMSW-5 was the only one with more than 2 anchors in a "pallet" upright column base plate (Table 18). As a consequence, this connection was by far the strongest one among all the structures (Fig. 30b).

Figure 31a shows the participation of axial and shear forces among all structures in the DA direction for "tower" upright columns' base connections. The participation rate of shear force was similar to the "pallet" uprights' base connections in the CA direction (around 20% in AMSW types 1, 2, 4 and 5). In AMSW-3 shear force participation was equal to 0 due to an additional base connection, which attracted all the shear forces, leaving upright's base connection subjected only to axial forces (Fig. 12). Figure 31b shows combined pullout-concrete cone failure capacities among all structures for "tower" uprights' base connections. Following conclusions has been obtained:

- The base connection used in AMSW-1 had higher capacity comparing to other structures because 8 anchor bolts have been used in the base plate that was the highest number among all structures (Table 19).
- Only in AMSW-1 obtained hierarchy of criticalities was satisfactorily for the development of energy dissipation mechanism: upright columns' base connections failure happened after yielding of diagonals (Fig. 23a).

Table 30 Possibility of the development of the energy dissipation mechanism among all structures

AMSW	Direction	First element to be failed	Failure mode	The development of energy dissipation mechanism	Demand/capacity
10% probability in 50 years earthquake					
1	CA	Upright column	Brittle (flexural–torsional buckling)	No	1.09
2	DA	Diagonal-to-upright connection	Brittle (net section failure)	No	1.43
	CA	Diagonal	Brittle (flexural–torsional buckling)	No	0.86
3	DA	Base connection	Brittle (combined pullout-concrete cone failure)	No	1.41
	CA	Base connection	Brittle (combined pullout-concrete cone failure)	No	2.01
4	DA	Upright column	Brittle (flexural–torsional buckling)	No	1.00
	CA	Upright column	Brittle (flexural–torsional buckling)	No	1.14
	DA	Upright column	Brittle (flexural–torsional buckling)	No	2.60
5	CA	Diagonal-to-upright connection	Ductile (bearing failure)	Yes (limited)	0.53
	DA	Base connection	Brittle (combined pullout-concrete cone failure)	No	3.02
2% probability in 50 years earthquake					
1	CA	Base connection	Brittle (combined pullout-concrete cone failure)	No	2.35
	DA	Diagonal-to-upright connection	Brittle (net section failure)	No	2.36
2	CA	Base connection	Brittle (combined pullout-concrete cone failure)	No	1.36
	DA	Diagonal-to-upright connection	Brittle (net section failure)	No	2.85
3	CA	Base connection	Brittle (combined pullout-concrete cone failure)	No	3.96
	DA	Upright column	Brittle (buckling)	No	1.87
4	CA	Base connection	Brittle (combined pullout-concrete cone failure)	No	1.97
	DA	Base connection	Brittle (combined pullout-concrete cone failure)	No	4.31
5	CA	Diagonal-to-upright connection	Ductile (bearing failure)	Yes (limited)	1.06
	DA	Base connection	Brittle (combined pullout-concrete cone failure)	No	3.80

Fig. 32 Brittle failure of diagonals in bracing towers (Konkong 2017)

6 Discussion of the results and design proposals

In majority of the considered structures, brittle failure of upright columns and their base connections preceded failure of the diagonal members in both the cross-aisle (CA) and the down-aisle (DA) directions (Table 30). In particular:

- In the CA direction only in AMSW type 5 the energy dissipation mechanism could be achieved. However, bearing failure of the connection can provide only limited energy dissipation for a few load cycles (Bernuzzi and Castiglioni 2001).
- Demand/capacity ratio of the most stressed member is lower than 1 in the CA direction only in AMSW types 2 and 5.
- In AMSW types 1, 3, and 4 brittle failure of the upright columns and their base connections occurred before the activation of any energy dissipation mechanism in the CA direction with demand/capacity ratio higher than 1.
- In the DA direction in all AMSWs brittle failure of either diagonal-to-upright connections, “tower” upright columns or their base connections happened before the yielding of the bracing diagonal with demand/capacity ratio higher than 1.

Since seismic hazard has a probabilistic nature, sometimes earthquakes that are more severe than designed ones could happen. Therefore, structural assessment for earthquakes with 2% probability of occurrence has been performed. The same structures and design assumptions have been used as for the seismic event with 10% probability of occurrence. The only difference was a scale factor applied to the accelerograms. The results are indicated on Table 30 and show the importance of having adequate hierarchy of criticalities and development of energy dissipation mechanisms in AMSWs.

6.1 Diagonals, and their connections to the upright columns

The main issue in the diagonals is that diagonal-to-upright connections’ failure preceded the yielding of the diagonal members. The connection failure was the first to occur in all the DA frames and in 1 CA frame of the considered structures. In 2 of the DA frames and all the CA frames diagonal-to-upright connection bearing failures has been observed. Furthermore, in the DA direction, either net section or bolt shear failures happened before the yielding of the diagonal. These failure modes are considered as brittle [Fig. 32 (Konkong 2017)], and they should be avoided to appear before ductile

Fig. 33 Example of upright reinforcement

failure modes in a seismic loading condition. Although the bearing failure is more ductile than the net-section and the bolt-shear failures, such yielding phenomena still relies on the local plasticity around the bolt holes (Bernuzzi and Castiglioni 2001), and in case of cyclic loading, its contribution is strongly influenced by the sequence of the loading. To achieve a global ductility, it is recommended to prioritize the bracing member yielding to the connection failure of any type.

Following proposals have been made for strengthening this connection to avoid its failure before yielding of the diagonal member:

- Use an additional safety factor to calculate the diagonal-to-upright connection thickness.
- Use an additional safety factor to calculate the number of bolts in the diagonal-to-upright connection.

In both the DA and the CA directions brittle failure of other structural elements preceded the development of energy dissipation mechanism in the diagonals. Therefore, during the amplification of the seismic action by the value of q in all diagonal bracing members and their connections in the CA direction, and in the DA direction (when tension and compression diagonals are used without horizontal compression element as suggested by EN 16681 (2016)], attention must be paid to avoid the brittle failure of other structural components preceding the diagonal failure.

6.2 Upright columns

Upright columns failure due to the combined action of compression and bending forces preceded the diagonals failure in the majority of considered structures both in the DA and the CA directions. In particular, “pallet” upright columns in the CA direction buckled after diagonals failure, only in AMSW types 2 and 5, where the strongest upright column’s cross-section has been used (Table 29). It was mainly achieved due to a larger upright’s thickness comparing to other structures. In addition, buckling of “tower” upright columns in the DA direction happened after yielding of the diagonals only in AMSW-1, where upright’s cross-sections have been reinforced. Following proposals have been made to develop the energy dissipation mechanisms of AMSWs:

Fig. 34 Concrete cone failure

- It is necessary to design upright columns with a sufficient overstrength to develop the formation of energy dissipation mechanisms in the diagonal members.
- Reinforce upright columns' cross-section: Reinforcing can be done by welding a steel plate to it, which increases its thickness and therefore resistance (Fig. 33).
- Develop a limit for non-dimensional slenderness in upright columns' cross-sections: Non-dimensional slenderness was high in all the cases, comparing to the columns in typical steel structures. The development of more compact sections can increase upright columns' seismic performance and their buckling resistance.
- Perform real scale tests on all the upright columns' cross-sections: It can help to accurately evaluate the role of the distortional buckling mode in a structural behavior.
- Develop an amplification factor for acting seismic forces in the upright columns: It can encourage producers to use stronger cross-sections for upright columns to obtain their failure after the yielding of the diagonals, which can help a correct energy dissipation mechanism to develop.

6.3 Upright columns' base connections

Base connections for both “pallet” and “tower” upright columns were one of the most critical structural elements in the CA and the DA directions, which must be designed with a sufficient overstrength to avoid their premature failure in a seismic loading condition. Especially, recalling the fact that their failure is brittle due to combined concrete cone and pullout failure (Fig. 34).

Following proposals have been made:

- Develop an amplification factor for acting seismic forces in the base connection. It can encourage producers to use stronger base connections to obtain their failure after the yielding of the diagonals, which can help a correct energy dissipation mechanism to develop.
- The use of the same base connections for both “pallet” and “tower” upright columns is questionable since internal actions were significantly bigger in the later ones. Therefore, it can lead to either “pallet” upright's base connection overdesign or “tower” upright's base connection under design.

7 Conclusions

In this article, the behavior of Automated Multi-Depth Shuttle Warehouses (AMSWs) has been investigated using time-history analysis by direct integration including P-Delta effects, in the context of low-to-moderate seismicity. The Authors collected 5 different AMSW configurations designed by 5 European rack producers according to current standards [EN 15512 (2009), EN 16681 (2016)] for a site located in Montopoli (Pisa, Italy, characterized by low-to-moderate seismicity with PGA 0.13 g). The outcomes of this research have been obtained with the following assumptions and considerations:

- The structural damping has been taken as 3% for all the structures without taking into account the sliding effects of the pallets on the beams.
- Each structure has been analyzed by means of 2 2D models (the cross-aisle (CA) and the down-aisle (DA) directions).
- The warping has not been considered during the structural analysis; it has been considered during performance verifications.
- Distortional buckling has not been explicitly taken into account in upright columns' check since during the laboratory tests it was less critical than flexural–torsional buckling.
- A moderate number of ground motion recordings (15) has been used.

These simplifications were included considering the necessity of limiting the computational efforts of the complex AMSW models, therefore the obtained results require further confirmations. However, the low variability within the results, and the observation of common aspects in all the performed analyses allowed us to obtain some relevant and interesting conclusions. The results, indeed showed, the dominance of the following brittle failure mechanisms in all the structures: concrete cone and pullout failure of upright columns' base connections, flexural torsional buckling of uprights, net-section and bolt shear failure of the diagonal-to-upright connections, and the lack of energy dissipation mechanism activation as a consequence. Regarding the structural behavior in the CA direction, the following conclusions can be made:

- Structural behavior can be enhanced by enforcing capacity rules on the upright frames, securing that the diagonals fail before any brittle failure mechanism in upright columns, and their base connections is activated.
- Since the bending moment participation in upright columns was high (up to 30%), “pallet” upright column's safety depended on its bending capacity, and on its buckling capacity.

Regarding structural behavior in the DA direction (DA), the following conclusions can be made:

- Structural behavior can be enhanced by enforcing capacity rules on the bracing towers, securing that the diagonals yield before any brittle failure mechanism in the diagonal-to-upright connection, uprights, and their base connections.
- Since bending moment participation in upright columns was low (less than 10%), “tower” upright column's safety depended mainly on its buckling capacity.

Based on the above conclusions, the Authors can make the following design proposals to enhance the structural behavior of AMSWs in the low-to-moderate seismicity context:

- Develop safety coefficients for diagonal-to-upright connection calculations.
- Amplify the seismic action by the value of q in all diagonal bracing members and their connections in the CA direction, and in the DA direction (when tension and compression diagonals are used without horizontal compression element), being aware that the premature failure of upright columns and their base connection is avoided.
- Specify a limit value for the non-dimensional slenderness of the upright columns' cross-sections.
- Perform real scale tests on the upright columns' cross-sections to calibrate the models and design verifications.
- Guarantee a sufficient overstrength on the upright columns and at their base connections by using an amplification factor for the acting seismic forces so that their brittle failure does not precede the failure of other AMSW's components.

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Availability of data and material The data that support the findings of this study are subjected to confidentiality restrictions imposed by the supplying racking companies, and cannot thus be rendered publicly available.

Code availability All code data used to generate results that are reported in the article and central to its main claims are available from the Authors upon reasonable request and with permission of racking companies.

Declarations

Conflict of interest The Authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper. The Authors also declare that they have no conflicts of interests to disclose with participated racking companies.

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