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.

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. Reduced running costs, automation, and larger sizes are the future trends of the warehouse sector. 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 [1-2]. 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 [3] 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 [4] which provides principles for the structural design of pallet racking systems and EN 16681 [5] 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 [6-7]. 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 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 2.a [8] 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. The connection of the frames with the foundation was also very weak as the vertical elements were observed to fail at the vertical element-to-foundation interface [9] as indicated in Figure 2.b [9].

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 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 [3]. The main elements of storage distribution in AMSWs are (Figure 3 [10]):

- 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 rack systems, which represents the structural basis of all Rack Buildings. Different rack types were introduced by Pekoz (1973) [11]. 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 – cross-aisle and down-aisle; they are indicated in Figure 4 [12].

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 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 [12]. 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 [13-18]. Also, several full-scale shake table tests were performed on these structures [19-21]. Nowadays, there are only a few studies about the seismic behavior of clad rack warehouses in the literature. Kilar (2011) [22] 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 [23], 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, EN16681[5] states that the critical condition during the seismic event is reached when rack is 100% loaded. Haque (2012) [24] 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 that of traditional steel structures. As a consequence, the bracing system in the down-aisle direction is needed. Caprili (2017, 2018) [25-26] 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 [23] and Italian building code NTC 2018 [27]. 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 our work, to quantify the structural performance of Automated Multi-Depth Shuttle Warehouses (AMSWs) in the context of low-to-moderate seismicity, we collected 5 different AMSW configurations designed by 5 European rack producers according to current standards (EN 15512 [4], EN 16681 [5]). As site location, Montopoli (Pisa, Italy) has been chosen, which is characterized by a peak ground acceleration (PGA) of 0.13g. We 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 (SAP 2000 [28]). To limit 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). 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 [28] 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 [5]:

\[
G_1 + G_2 + Q + E
\]

**EQ.1 [5]**

- “\( G_1 \)” identifies the permanent structural loads.
- “\( G_2 \)” represents the non-structural permanent loads (the weight of cladding panels). It is taken as 20 kg/m\(^2\) 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 [5]. The snow load has not been considered.
The site is located in Montopoli (Italy) with coordinates of [43.67°N, 10.74°E] on a soil with $V_{s30}=270\text{m/s}$ represented by soil type C in EN 1998-1 [23]. The seismic hazard and disaggregation computations have been performed with the OpenQuake [29] software, based on the SHARE [30] area source model and using the ground motion prediction equation proposed by Boore and Atkinson (2008) [31]. 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 0.1s, to link the hazard and the selected record set. Following the Conditional Spectra record selection approach [32], a set of 15 pairs of ground motions corresponding with the 10% probability of occurrence in 50-year return period were selected and scaled. Figure 5.a shows the disaggregation bar charts and Figure 5.b 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, therefore, 2D analyses have been carried out in each case separately in 2 directions (the down-aisle and the cross-aisle). 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. 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 [5].

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 [5] (Table 4). Regarding behavior factors q for a low dissipative design, EN16681 [5] 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 typology in the DA direction). Table 22 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 we have performed time history analyses, q and $K_d$ parameters have not been taken into account, but all other design assumptions
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 [33]. EN 1993-1-1-2005 / Eurocode 3 part 1 [34], EN 1993-1-3-2007 / Eurocode 3 part 3 [35], EN 15512 / Specific regulations for steel racks [4], and EN 16681/ Regulation for steel racks design in seismic situations [5] have been used for the verification checks of elements. For the verification of the connections, EN1993-1-8-2005 [36] has been used. Base plates with bonded anchors verifications have been done according to the EOTA TR29 [37], EOTA TR45 [34], and EOTA TR49 [35]. Each base plate resistance has been obtained in Hilti PROFIS Anchor software [38]. Table 5 reports the material factors used in the checks in accordance with NTC2018 [27]. For the consistency, the same material factors have been used for all structures.

### 3 Structures Definition

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.
- “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. 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 [4].
- 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 [4].
- Bending tests on beam end connector (to evaluate its rotational stiffness) in accordance with EN 15512, Annex A.2.4 [4].
- Tests on floor connections (to evaluate their rotational stiffness in the DA direction) in accordance with EN 15512, Annex A.2.7 [4].
The main structural dimensions in the CA direction for all the structures are presented in Table 6 with reference to Figure 8, where:

- A is single frame width.
- B is the spacer length between single frames.
- C is the width of the shuttle tunnel.
- D is the width of one block.
- E 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 Figure 9, where:

- A is a width between upright’s axis in the bracing tower.
- B 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 (Figure 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, Figure 8 and Figure 9) and its overall height (H tot, Figure 8 and Figure 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. These connections are schematically shown in Figure 10 for the CA frames and in Figure 11 for the DA frames.

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 12 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 (Figure 12).
Parameters for all these shapes are presented in Table 11, Table 12, Table 13, Table 14, Table 15, and Table 16 in terms of:

- 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 ($\rho_y/\rho_z$).

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 ($A_{\text{eff}}/A$)
- 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 11 indicates parameters for “pallet” upright columns, that support pallets and unit loads.
- Table 12 shows parameters for “tower” upright columns that are located in bracing towers.
- Table 13 presents parameters for diagonals in the CA direction.
- Table 14 gathers parameters for the diagonals in the bracing towers in the DA direction.
- Table 15 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 16 indicates parameters for bracing beams located in bracing towers.

Figure 13 shows 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 17 shows geometric parameters and anchors characteristics for “pallet” upright columns’ base plates.
- Table 18 gathers geometric parameters and anchors characteristics for “tower” upright columns’ base plates.
- Table 19 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.
EQ.2 [5] indicates the combination of vertical loads used to define the seismic mass $W_E$ in both the CA and the DA directions:

$$W_E = G_1 + G_2 + R_f \cdot Q \quad EQ.2 \ [5]$$

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 20 presents the information about seismic mass $W_E$ and all its components from EQ.2 [5] 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 21 shows, that in all the structures unit loads provide around 90% of the total seismic mass of the considered AMSWs.

4 Results

4.1 Modal analysis

Regarding modal analysis, following conclusions have been obtained:

- 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 [23].
- Modes along the CA and the DA direction were separated, the most relevant modes were translational only in one of the principal structural directions. This was expectable, due to 2D analyses of the structures.

Table 22 illustrates the relevant modes, corresponding period, and participant mass for considered structures.

Deformed shapes obtained during modal analysis were similar for all the structures (Figure 14, Figure 15, Figure 16, Figure 17). 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.

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 Figure 18, 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.

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 (Figure 19.a). Stiffer structures attracted higher seismic forces, therefore in AMSW types 4 and 5:

- Structural base shears were higher than in AMSW types 1, 2, and 3 (Figure 19.b).
- Maximum axial forces in “pallet” upright columns were higher than in AMSW types 1, 2, and 3 (Figure 19.c).
- Maximum bending moments in upright columns were higher than in AMSW types 1, 2, and 3 (Figure 19.d).

Maximum internal actions in “pallet” upright columns are presented as an average value among 15 time histories for all the structures. Following observations are valid for all the structures:

- Maximum tensile forces were lower than maximum compressive forces (Figure 19.c) since upright columns were compressed by vertical loads in addition to the horizontal seismic action. It points out the importance of the buckling phenomena in the upright columns in the CA direction.
- Values of maximum positive moment (sagging) and maximum negative moment (hogging) were very close to each other (Figure 19.d).

In the diagonals that connect upright columns with each other, the lowest axial forces have been obtained in AMSW-2 (Figure 19.e) since only in this AMSW X-shaped bracing was used (Table 6), which attracted the lowest axial forces per each diagonal member. Maximum positive (tension) and negative (compression) axial forces values were close to each other.

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. 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, 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 EN16681 [5].

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. Figure 22 illustrates this phenomenon for AMSW-1, but the same observation has been made for all other structures. 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 (Figure 19.c and Figure 25.c).

Two different bracing towers distributions in the plan have been used among all the structures in the DA direction:

- One bracing tower in each edge (AMSW types 1, 2, 4, and 5, Figure 23.a).
- Two bracing towers in each edge (AMSW-3, Figure 23.b).

In AMSW-3, additional base connections have been used (Figure 24). Obtained structural behavior did not differ between AMSW-3 and other structures in terms of structural criticalities and their hierarchy. The main obtained differences are:

- In structural periods. In AMSW-3 the lowest fundamental period in the DA direction has been obtained (0.90 s) among all the structures (Figure 25.a). It was expected, since 4 bracing towers have been used in AMSW-3 and the structure was the stiffest one among all the structures as a consequence.
- In base shear. The highest value of base shear has been obtained in AMSW-3 (Figure 25.b). The average values among 15 time histories for each AMSW are shown.
In terms of axial forces in “tower” upright columns following observations have been made (Figure 25.c, maximum internal actions are presented as an average value among 15 time histories for all the structures):

- The highest axial forces were acting in AMSW-5.
- As well as in the CA direction, maximum positive axial force (tension) was lower than maximum negative axial force (compression). It happened because upright columns were compressed by vertical loads in addition to the horizontal seismic action, which points out the importance of the buckling phenomenon in the uprights in the DA direction.

Assessment has been done on “tower” upright columns, since they have been much more stressed in DA direction comparing to “pallet” upright columns. Following observations have been made regarding bending moments in “tower” upright columns (in Figure 25.d, maximum internal actions are presented as an average value among 15 time histories for all the structures):

- Acting bending moments were low comparing to typical buildings.
- In AMSW-5, the highest values of bending moments were reached.
- In AMSW-3, bending moment values were very low comparing to other structures due to the additional base connections used in bracing towers (Figure 24). They have a fixed support and attracted majority of the moments acting on the structure leaving upright columns almost unstressed in terms of bending moment.

Self-weight of the structure in AMSW-5 is twice higher comparing to all other structures (Table 20), since larger and more resistant cross-sections have been used. It explains, why internal actions in “tower” upright columns in AMSW-5 were higher than in AMSW types 1, 2, and 4. The internal actions in each “tower” upright column were higher in AMSW-5 since the seismic action was shared between 6 and 4 “tower” upright columns respectively in AWSW types 3 and 5 (Figure 23.b).

The same procedure of evaluating the most stressed members as in the CA direction has been applied also for the DA direction. The variability of normalized utilization factor for each member in the DA direction among 15 earthquake records is gathered in Figure 26. These graphs show only the location of criticality for the most critical failure mode in it. “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 (Figure 26.d). Utilization factors for diagonals’ base plates are presented in cases when they have been used in the AMSW.

Figure 27 represents the variability of generalized 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 (Figure 28). Among all the members in a bracing tower, the bracing beam was
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 Table 23 and Table 24 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 23) since the shear centre was not coincident with the centroid of cross-section in these cases (Figure 12, Table 13). 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 24). 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.

5.2 Upright columns

Upright columns played a crucial role in both the CA and the DA directions. Torsional flexural buckling mode was the most critical for all the cross-sections (due to shear center eccentricity, Figure 12.a). Distortional buckling failure mode has not been considered due to the absence of tests performed on real-height cross-sections. Table 25 shows the following parameters that are relevant to compare and possibly optimize upright columns’ performance (only upright columns in lower parts of the frames are considered since they were the most stressed ones):

- Steel grades.
- Availability of experimental tests.
- The effective cross-section areas $A_{\text{eff}}$ 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 $W_{\text{el}}$ 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 (Figure 22). Therefore, demand/capacity ratios for the most stressed “pallet” upright columns in the CA direction among all structures are reported (Figure 29.a). 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 25). Only in AMSW types 2 and 5 upright column failed after a failure in the diagonal (Figure 20.b, and Figure 20.e), which is favourable condition during the earthquake excitation for the development of energy dissipation mechanisms. 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 (Figure 19.d).
- Figure 29.b 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 25). 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 30 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. 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, Figure 31.a. 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. The lowest Demand/Capacity ratios have been obtained in AMSW types 5
and 1, where the used upright columns’ cross-sections had much larger areas than in all others structures (Table 25). In AMSW types 1, 2 and 5 failure of the diagonals preceded the buckling of the upright column (Figure 26.a, Figure 26.b, and Figure 26.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 25), and its bending capacity was significantly higher as a consequence (Figure 31.b). In the DA direction, moment participation in the upright columns has been checked. Figure 32 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 25).

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 (Figure 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 Figure 33.a. In all the cases participation of the shear force was 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 33.b shows the comparison of combined pullout-concrete cone failure capacity among all the structures. Following conclusions have been obtained:

- In AMSW types 1, 2, 3, and 4, the brittle failure of the upright columns’ base connections preceded the failure in the diagonals.
- Only in AMSW-5, the failure in the diagonals preceded the brittle failure of the upright columns’ base connections (Figure 20.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 17). As a consequence, this connection was by far the strongest one among all the structures (Figure 33.b).

Figure 34.a. 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 (Figure 24). Figure 34.b 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 in AMSW-1, 8 anchor bolts have been used in the base plate that was the biggest number among all structures (Table 18).
- 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 (Figure 26.a).
- The use of the same base connection for both “pallet” and “tower” upright columns (as it was done in AMSW-5) is questionable. Maximum axial forces in “tower” uprights were much higher than in “pallet” ones.

Figure 33.b and Figure 34.b show how combined pullout-concrete cone failure capacity in AMSW-5 from the highest among all structures for “pallet” upright columns becomes the lowest among all structures for “tower” upright columns.

6 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 26).

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 (Figure 35 [41]), and they should be avoided to appear before ductile 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 [17], 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[5]), 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-5, where the strongest upright column’s cross-section has been used (Table 25). It was mainly achieved due to a larger upright’s thickness (2.5t) comparing to other structures (t). 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:

- 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 (Figure 36).
- 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 evaluate the role of the distortional buckling mode in a structural behavior, which has not been considered now due to absence of any such tests.
- 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 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 (Figure 37).

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. We collected 5 different AMSW configurations designed by 5 European rack producers according to current standards (EN 15512 [4], EN 16681 [5]) for a site located in Montopoli (Pisa, Italy, characterized by low-to-moderate seismicity with PGA 0.13g).

The results 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 cross-aisle direction (CA), the following conclusions can be made:

- Structural behavior can be enhanced by enforcing capacity rules on the upright frames, securing that the diagonals yield before any brittle failure mechanism in upright columns, and their base connections.
- 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 down-aisle 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, we 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.

Declarations

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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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Conflict of interest:

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no conflicts of interests to disclose.

**Availability of data and material:**

The data that support the findings of this study are available from racking companies but restrictions apply to the availability of these data, which were used under license for the current study, and so are not publicly available. Data are however available from the authors upon reasonable request and with permission of racking companies.

**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.

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Tables

Table 1. Design information.

| Importance class       | I (Warehouses with fully automated storage operations) |
|------------------------|------------------------------------------------------|
| Design life            | 50 years                                             |
| Warehouse type         | Automated warehouse                                   |
| Unit load weight ...   | Shown in Table 3                                      |
| Rf, rack filling ...   | Specified for each AMSW                               |
| Storage environment    | Standard                                             |
| Type of load ...       | Wooden Pallet                                        |
| Class of stored goods  | A (Compact Constrained)                              |
| Seismic sway of the building | Non-sway                                   |
| Structural damping     | 3%                                                   |
| Characterization ...   | C30/37 with a thickness of 80cm                       |
### 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 x Depth (mm) | Load dimensions Width x Depth x Height (mm) | Maximum Weight (kg) | Load levels |
|------|-------------------------------------|---------------------------------------------|---------------------|-------------|
| 1    | 1219x1050                           | 1300x1100x2100                              | 1000                | 1 to 2      |
| 2    |                                     |                                             | 800                 | 3 to 5      |
| 3    |                                     |                                             | 600                 | 6 to 9      |

### 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, Kd = 0.8)   | -20% (RF = 0.8)          | Yes (axial spring)        |
|      | DA        | -46% (q=1.5, Kd = 0.8)   | -20% (RF = 0.8)          | -                         |
| 2    | CA        | -46% (q=1.5, Kd = 0.8)   | -20% (RF = 0.8)          | Yes (reduced area of diagonal's cross-section) |
|      | DA        | -60% (q=2, Kd = 0.8)     | -20% (RF = 0.8)          | -                         |
| 3    | CA        | -46% (q=1.5, Kd = 0.8)   | NO (RF = 1)              | Yes (axial spring)        |
|      | DA        | -60% (q=2, Kd = 0.8)     | -20% (RF = 0.8)          | -                         |
| 4    | CA        | -46% (q=1.5, Kd = 0.8)   | -20% (RF = 0.8)          | No                        |
|      | DA        | -60% (q=2, Kd = 0.8)     | -20% (RF = 0.8)          | -                         |
| 5    | CA        | -34% (q=1.5, Kd = 1)     | -20% (RF = 0.8)          | No                        |
|      | DA        | -34% (q=1.5, Kd = 1)     | -20% (RF = 0.8)          | -                         |

Table 5. Material factors

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

Table 6. Structural geometries in the CA direction among all structures.
Table 7. Structural geometries in the DA direction among all structures.

| AMSW | A [m] | B [m] | C [m] | D [m] | E [m] | Total amount 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”              |

Table 8. Load level heights among all structures.

| AMSW | C [m] | D [m] | Total amount 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                            |
| 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  |

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 | Semi-rigid              | Semi-rigid              | Pinned                  | Semi-rigid              |
|                                             | DA        | Semi-rigid              | Semi-rigid              | Pinned                  | Pinned                  | Semi-rigid              |
| “Tower” upright columns’ base               | DA        | Semi-rigid              | Pinned                  | Pinned                  | Pinned                  | Semi-rigid              |
| Upright column                              | All       | Continuous from the base to the top, pinned at the roof beam | Semi-rigid              | Pinned                  | Semi-rigid              |
| Diagonal-to-upright column                  | CA        | Pinned with an axial spring | Pinned with an axial spring | Pinned | Pinned | Pinned |
| Horizontal-to-upright column                | CA        | Pinned in all structures | Semi-rigid              | Pinned                  | Semi-rigid              |
| Diagonal-to-upright column                  | DA        | Pinned in all structures | Semi-rigid              | Pinned                  | Semi-rigid              |
| 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. Key geometric parameters of “pallet: upright columns’ cross-section.
| Material  | AMSW-1 Upper | AMSW-1 Lower | AMSW-2 Upper | AMSW-2 Lower | AMSW-3 Upper | AMSW-3 Lower | AMSW-4 Upper | AMSW-4 Lower | AMSW-5 Upper | AMSW-5 Lower |
|-----------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|
| S350 GD   | 50.00        | 33.33        | S420 MC      | 30.00        | 30.00        | 29.20        | 33.20        | 50.50        | 40.80        | 25.20        |
| S420 MC   | 60.00        | 40.00        | S420 MC      | 60.00        | 60.00        | 40.00        | 40.00        | 34.50        | 49.28        | 23.00        |
| S355 MC   | 0.81         | 0.86         | 0.82         | 0.85         | 0.92         | 0.80         | 0.90         | 0.83         | 0.95         |
| S500 MC   | 2.27         | 2.19         | 5.18         | 4.97         | 2.15         | 1.76         | 0.39         | 1.35         | 1.12         |
| S500 MC   | 1.51         | 1.48         | 2.28         | 2.23         | 1.46         | 1.32         | 0.62         | 1.16         | 1.06         |
| Wz/Wy, left | 1.17        | 1.19         | 2.10         | 1.92         | 1.33         | 1.28         | 0.44         | 0.99         | 1.02         |
| Wz/Wy, right | 2.32        | 2.32         | 2.78         | 2.78         | 0.79         | 1.28         | 1.43         |
| y0/d      | 2.28         | 2.31         | 1.83         | 1.75         | 2.08         | 2.11         | 2.12         | 2.15         | 2.20         |

Table 12. Key geometric parameters of “tower” upright columns’ cross-section.
| Material      | AMSW-1 Upper | AMSW-1 Lower | AMSW-1 Lower reinforced | AMSW-2 Upper | AMSW-2 Lower | AMSW-3 Upper | AMSW-3 Lower | AMSW-4 Upper | AMSW-4 Lower | AMSW-5 Upper | AMSW-5 Lower |
|--------------|-------------|--------------|-------------------------|-------------|--------------|-------------|--------------|-------------|--------------|-------------|--------------|
| 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        |
| $\rho_z/\rho_y$ | 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 13. Key geometric parameters of diagonals in CA direction cross-section.
| Material      | AMSW-1 Upper | AMSW-1 Lower | AMSW-2 Upper | AMSW-2 Lower | AMSW-3 Upper | AMSW-3 Lower | AMSW-4 Upper | AMSW-4 Lower | AMSW-5 Upper | AMSW-5 Lower |
|---------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|
| S350 GD       | U-shape      | U-shape      | S280 GD      | U-shape      | S250 GD      | S250 GD      | S355 GD      | S355 GD      | S275 JR      | S275 JR      |
| Cross-section | CHS 30x1     | U-shape      |               | U-shape      | U-shape      | U-shape      |               |               | SHS 50x2     |               |
| A             | 42.67        | 32.00        | 26.67        | 46.67        | 16.00        |               |               |               |               |               |
| B             | 33.33        | 25.00        | 26.67        | 20.00        | 10.00        |               |               |               |               |               |
| I<sub>z</sub>/I<sub>y</sub> | 2.12         | 2.14         | 1.85         | 8.52         | 3.22         |               |               |               |               |               |
| ρ<sub>z</sub>/ρ<sub>y</sub> | 1.46         | 1.46         | 1.36         | 2.92         | 1.79         |               |               |               |               |               |
| W<sub>z</sub>/W<sub>y</sub>, up | 1.24         | 1.27         | 1.73         | 1.87         | 1.51         |               |               |               |               |               |
| W<sub>z</sub>/W<sub>y</sub>, bottom | 2.00         | 2.06         | 1.73         | 5.43         | 2.51         |               |               |               |               |               |
| y<sub>0</sub>/d | 2.28         | 2.25         |               |               |               | 1.49         | 1.79         | 2.11         |               |               |

Table 14. Key geometric parameters of diagonals in DA direction cross-section.
| Material  | AMSW-1 Upper | AMSW-2 Upper | AMSW-3 Upper | AMSW-4 Lower | AMSW-5 Lower |
|-----------|--------------|--------------|--------------|--------------|--------------|
| S355 JR   | S355 JR      | S280 GD      | S235         | S355         | S355 MC      |
| S355 JR   | L66x33x3     | L66x33x4     | U-shape      | L70x70x6     | L80x80x8     |
| L66x33x4  | U-shape      | L70x70x6     | Railing section 71x5 (x2) | RHS 140x80x10 |
| A         | 35.00        |              |              |              |
| B         | 17.50        |              |              |              |
| $l_z/l_y$ | 3.76         |              |              |              |
| $\rho_z/\rho_y$ | 1.94      |              |              |              |
| $W_z/W_y$, up | 1.57      |              |              |              |
| $W_z/W_y$, bottom | 2.20 |              |              |              |
| $y_0/d$  | 1.31         |              |              |              |

**Table 15. 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|
|                  |        |        |        |        |        |
| 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<sub>z</sub>/I<sub>y</sub> | 8.12   | 3.91   | 13.15  | 7.60   | 2.16   | 1.38   | 8.24   |
| ρ<sub>z</sub>/ρ<sub>y</sub> | 2.85   | 1.98   | 3.63   | 2.76   | 1.47   | 1.17   | 2.87   |
| W<sub>z</sub>/W<sub>y</sub>, up| 2.29   | 1.95   | 5.73   | 2.63   | 1.31   | 1.15   | 2.77   |
| W<sub>z</sub>/W<sub>y</sub>, bottom| 4.43 |

**Table 16. 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 90x3| U-shape| Boxed C-shape |
| A                | 50.00  | 22.22  | 75.00  |
| B                | 30.00  | 13.89  | 25.00  |
| I<sub>z</sub>/I<sub>y</sub> | 4.47   | 3.09   | 8.24   |
| ρ<sub>z</sub>/ρ<sub>y</sub> | 2.11   | 1.76   | 2.87   |
| W<sub>z</sub>/W<sub>y</sub>, up| 1.17   | 1.46   | 2.77   |
| W<sub>z</sub>/W<sub>y</sub>, bottom| 2.92   | 2.40   |

**Table 17. “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 = 150mm
Post-installed, 2M16 C8.8, L = 200mm
Post-installed, 2M16 C8.8, L = 120mm with gap filling
Post-installed, 2M20 C8.8, L = 92mm
Post-installed, 6M12 C8.8, L = 240mm

Table 18. “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 = 375mm
Post-installed, 4M16 C8.8, L = 300mm
Post-installed, 4M24 C5.8, L = 400mm with gap filling
Post-installed, 2M20 C8.8, L = 372mm
Post-installed, 6M12 C8.8, L = 240mm

Table 19. “Diagonal bracings’ base plates parameters.
| Anchors       | Post-installed 6M16 C8.8, L = 200mm | Post-installed 2M20 C8.8, L = 249mm | Post-installed 4M24 C8.8, L = 110mm |
|---------------|-----------------------------------|-----------------------------------|-----------------------------------|
| 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 |

**Table 20. 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 21. 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 |
Table 22. 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 23. 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 24. 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        |

**Table 25. Upright columns’ characteristics comparison.**
| AMSW | Upright column | Steel grade | Experimental tests | $A_{\text{eff}}$ | $\text{max}$ | $A_{\text{eff}}/A_{\text{tot}}$ | $W_{\text{el}}$ |
|------|----------------|-------------|---------------------|----------------|-------------|------------------|-------------|
| 1    | "Tower" reinforced | S350GD      | NO, data extrapolated from tests on similar sections | 4.67xA | 0.93 | 0.87 | 8.3x9W |
|      | "Tower" not reinforced | S350GD      | NO, data extrapolated from tests on similar sections | 2.28xA | 0.83 | 0.85 | 4.07xW |
|      | "Pallet" | S350GD      | NO, data extrapolated from tests on similar sections | 1.37xA | 0.85 | 0.86 | 2.14xW |
| 2    | "Tower" | S420MC      | NO                   | 2.55xA | 1.41 | 0.93 | 2.88xW |
|      | "Pallet" | S420MC      | NO                   | 1.28xA | 1.03 | 0.85 | 1.58xW |
| 3    | "Tower" | S355        | NO, data extrapolated from tests on similar sections | 2.71xA | 0.97 | 0.92 | 3.64xW |
|      | "Pallet" | S355        | NO, data extrapolated from tests on similar sections | 1.00xA | 1.40 | 0.8 | 1.00xW |
| 4    | "Tower" | S500MC      | NO                   | 1.76xA | 1.42 | 0.78 | 2.26xW |
|      | "Pallet" | S500MC      | NO                   | 1.35xA | 1.44 | 0.83 | 1.35xW |
| 5    | "Tower" | S500MC      | NO                   | 5.03xA | 0.93 | 1.00 | 10.19xW |
|      | "Pallet" | S500MC      | NO                   | 2.91xA | 1.04 | 0.95 | 4.65xW |

Table 26. 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 |
|------|-----------------|-------------------------------------|------------------------------------------------|-------------------------------------------------|
| 1    | CA              | Base connection                     | Brittle (combined pullout-concrete cone failure) | No                                             |
|      | DA              | Diagonal-to-upright connection      | Brittle (net section failure)                   | No                                             |
| 2    | CA              | Base connection                     | Brittle (combined pullout-concrete cone failure) | No                                             |
|      | DA              | Diagonal-to-upright connection      | Brittle (net section failure)                   | No                                             |
| 3    | CA              | Upright column                      | Brittle (buckling)                              | No                                             |
|      | DA              | Upright column                      | Brittle (buckling)                              | No                                             |
| 4    | CA              | Base connection                     | Brittle (combined pullout-concrete cone failure) | No                                             |
|      | DA              | Base connection                     | Brittle (combined pullout-concrete cone failure) | No                                             |
| 5    | CA              | Diagonal-to-upright connection      | Ductile (bearing failure)                       | Yes                                            |
|      | DA              | Base connection                     | Brittle (combined pullout-concrete cone failure) | No                                             |

Figures

Figure 1

ARSW during the construction [3].
a. Collapsed and undamaged parts of ARSW [8].

b. Longitudinal frame and its base connections failure in ARSW’s collapse in Sant’Agostino [9].

Figure 2

An ARSWs collapse in Sant’Agostino.

Figure 3

Automated Multi-Depth Shuttle Warehouse [10].

Figure 4
Cross-aisle and down-aisle directions [12].

| Montopoli-10% in 50 years | Montopoli-CS(AvgSA)-10% in 50 years |
|---------------------------|-------------------------------------|
| ![3D model of Montopoli](image1) | ![Spectral Acceleration](image2) |
| (a) Disaggregation analysis results | (b) Selected records and the 2.5<sup>th</sup>/50<sup>th</sup>/97.5<sup>th</sup> percentiles using $AvgSA$ as the conditioning IM corresponding to 10% in 50 years return period. |

**Figure 5**

Seismic hazard data used in the analysis
Figure 6

Analysed AMSWs' frames.
**Figure 7**

Plan view of the warehouse.
Figure 8

Non-dimensional geometry in the CA direction.

Figure 9

Non-dimensional geometry in the DA direction.
Figure 11

Elements’ connections in DA direction among all the structures.

- a. Ω-shape cross-section used for upright columns
- b. U-shape cross-section used for diagonals and beams
- c. Boxed C-shape cross-section used for beams

Figure 12

Cross-sections’ shapes characteristics of upright columns, beams and diagonals.
Figure 13

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

Figure 14

AMSW-1 CA Mode 1 deformed shape.
Figure 15

AMSW-1 CA Mode 3 deformed shape.

Figure 16

AMSW-1 DA Mode 1 deformed shape.
Figure 17

AMSW-1 DA Mode 2 deformed shape.
Fundamental structural periods and internal actions in the CA direction among all structures.

Figure 19

Fundamental structural periods and internal actions in the CA direction among all structures.
Figure 22

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

a. One bracing tower in each edge
b. Two bracing towers in each edge

Figure 23

Bracing towers distribution in plan.
Figure 25

Fundamental structural periods and internal actions in the DA direction among all structures.
Figure 27

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

Figure 28

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).
Figure 29

Assessment on the behavior of “pallet” upright columns in the CA direction among all structures.

Figure 32

AMSW-1 DA moment participation to upright check.
Figure 33

Assessment on the behavior of “pallet” upright columns’ base connections in CA direction among all structures.

Figure 34

Participation of forces in base plate checks in the DA.
Figure 35

Brittle failure of diagonals in bracing towers [41].

Figure 36

Example of upright reinforcement.
Concrete cone failure.