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SEISMIC PERFORMANCE OF DISSIPATIVE AUTOMATED RACK SUPPORTED WAREHOUSES

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Abstract. In this paper, performance of dissipative reduced-section diagonals implemented in Automated Rack Supported Warehouses (ARSWs) is analysed. ARSWs are storage systems where the steel racks traditionally used to store goods only, constitute also the primary system of the building. Given the lack of a reference design code to design these structures as seismic-resistant, and starting from a critical analysis of the current design approaches, a proper design strategy for these structures has been developed, which is based on the possibility to dissipate seismic energy in the bracings. However, the design of an over-resistant connection that would allow the yielding of the bracings is quite tricky due to the very low thickness of the elements usually adopted for these structures and is only possible by reducing the cross section of the profile. A wide experimental campaign is preformed to validate the behaviour of these diagonals, to be later implemented in the global numerical model of the structure to measure the effective performances of the case study ARSWs, both at local and global level.

1 BACKGROUND AND MOTIVATIONS

Automated Rack Supported Warehouses (ARSWs) are steel structures used for the automated storage of palletized goods. In these structures, steel racks, which are traditionally used for storage purposes, also constitute the primary structural system of the building. This upgrade of structural role of the racks has not been supported by the development of specific design rules for ARSWs, so to date the same regulations for traditional racks (EN15512 and EN16681 [1], [2]) are adopted by designers, together with Eurocodes, which unfortunately appear to be hardly applicable to ARSWs, being characterized by specific structural peculiarities which make them different for ordinary steel buildings [3]. Besides, ARSWs' racks keep the same assembly, structural characteristics and technological solutions of the traditional ones [4]. The lack of a dedicated design method and the consequent possible unsafe design can be noticed especially when ARSWs are installed in earthquake prone zones, due local damages or collapses that happened after seismic events. Speaking of ARSWs, some types of local damages can be considered as severe as global collapses, because they can temporarily prevent the usual operation of the warehouses, causing huge economic losses to its owner. This is a relevant issue when it comes to seismic-resistant structures [5].

Dealing specifically with seismic design, a critical analysis of the currently adopted design approach has been conducted by the authors through the structural assessment of 5 case study structures designed by 5 European companies that nowadays design and produce racks for traditional purposes and for ARSWs [6]. The aim of the research was to highlight the strong points and the critical aspects of the adopted design method, together with the resulting structural behaviour. To have comparable ARSW structures, some input design parameters have been fixed (structural types, seismic input, overall geometry, pallet load, etc.), and others have been set free to be chosen (structural schemes, profiles cross-section, material, definition of design seismic action and participant mass) to analyse the current trends in terms of structural choices, technological features and design strategies. The selected structural type is the "Double-Depth" warehouse, that together with the "Multi-Depth" one are the two big families for automated warehouses. The structural assessment of the designed structures has been performed through the execution of non-linear time history analyses and the execution of the safety checks of the main structural components [7]. Focusing on the transversal direction of the structure, which is also called Cross Aisle (CA) and where the resisting frame is made of the consequent upright trusses composing the racks (Figure 1a), the research highlighted that:

- Looking at the free-choice design parameters, the use of regulations for traditional racks for the definition of the design seismic force [2] implies adopting various parameters (besides behaviour factor) which strongly reduce the elastic seismic demand. The use of these parameters - which basically consider the contribution of pallets in dissipating seismic energy and the possibility of not having full load conditions in the warehouse - is scientifically justified for traditional racks [8] but not for ARSWs. Besides, all these reductions are not supported by proper safe design rules (as capacity design ones) that may prevent fragile failures, and this is evident from the outcomes of the structural assessment.
- About the structural assessment, the most critical components are base connections and diagonal-to-upright connections (Figure 1b). Most of the base connections are made of a steel plate, to which the column ("upright" in jargon) is welded, and the base plate is fixed to the concrete slab through post-installed bonded anchors. Failure is due to combined shear and tensile axial force with failure of anchors due to shear and concrete-cone mechanism for tensile force. Both the o mechanisms are brittle. Dealing with diagonal-to-upright connections, resistance is driven by bearing, due to the reduced thickness of the diagonal, which are thin-walled. Although this mechanism is not properly fragile, the ovalization of the hole may cause the connection to become loose. This kind of failure could be accepted only for limited expected dissipative capacity and preventing other fragile mechanisms to happen in all the other components.

To conclude, the currently adopted design strategies (in line with EN16681 [2]) could be applied only if the structure is expected to remain in the elastic field, but in any case, the adoption of the parameters for the definition of the seismic demand, which cause a strong reduction of the base shear, should be scientifically justified. On the other hand, if the building is expected to be dissipative, post-elastic sources seem not available or very limited, highlighting the necessity of the development of a proper new design approach for dissipative seismic-resistant ARSWs.

2 PHILOSOFY OF THE DESIGN METHOD

Starting from the previous considerations, a new design method has been developed which provides for the structure to be dissipative with concentration of dissipation in diagonals [15].

a)

b)

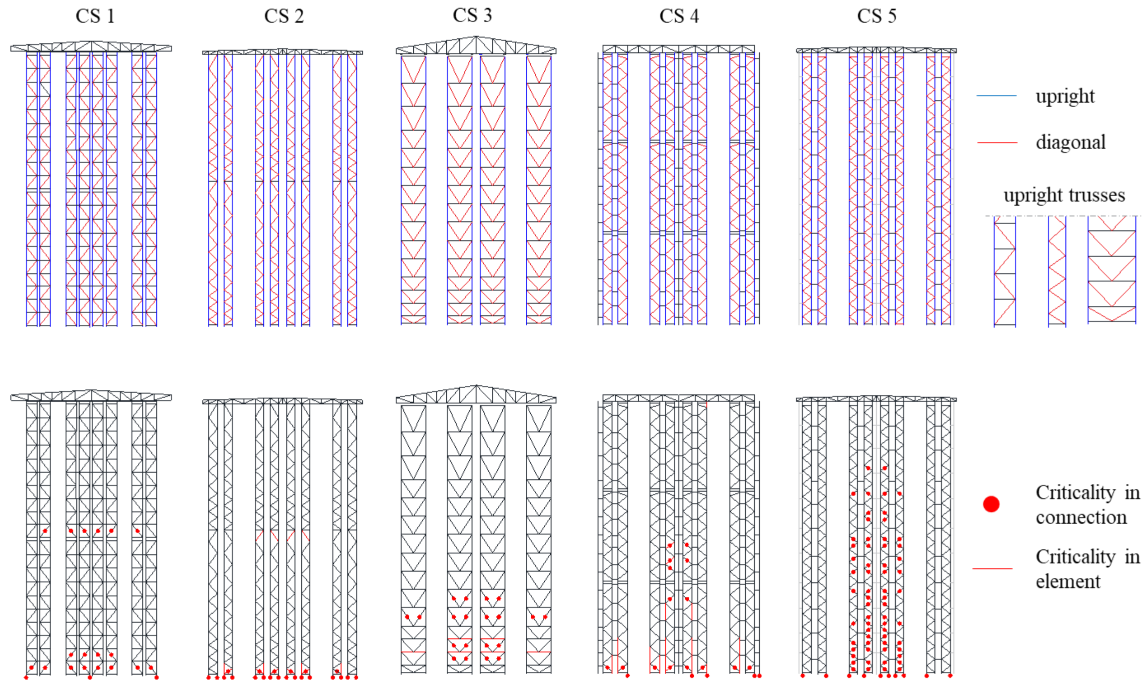


Figure 1: a) Geometrical configuration and structural schemes for the case studies along CA direction; b) localization of the criticalities characterized by D/Cs varying from the maximum one D/Cmax to the 0.9·D/Cmax.

This new approach is named "Over-resistant Connection Strategy" (OCS): it is applied to X tension-only structural schemes by adopting the Eurocode 8 prescriptions for medium-Ductility Class DC2 structures with the same structural scheme [9], [10], with slight modifications to meet ARSWs' structural peculiarities. In particular, since dissipation is expected in the diagonal element with all the other components designed to be over-resistant, the only way for diagonal-to-upright connection to be over-resistant is to locally reduce the cross-section of the bracing. Besides, only the lower part of the structure is involved in the collapse mechanism, while the higher remains in the elastic field (Figure 3a). This allows a better optimization and exploitation of the sources of the structure, being the higher forces concentrated in the lower part. This new design approach is studied for the CA frames. Indeed, in the longitudinal direction of the warehouse, stiffness and resistance toward horizontal actions is usually provided by standardly assembled X tension-only diagonals, similar to those observed for steel buildings with the same structural schemes.

The innovative part of the approach is the reduction of the cross-section of the diagonal, which in case of ARSWs is necessary to have an over-resistant diagonal-to-upright connection. According to capacity design rules for frames with concentric tension-only bracings, the connection of the dissipative element to the other components shall be designed by applying the (1):

$$R_d \geq \gamma_{rm} \cdot \gamma_{sh} \cdot R_{fy} \tag{1}$$

where R_d is the resistance of the connection; R_{fy} is the plastic resistance of the dissipative element, based on the nominal yield stress of the material; γ_{rm} is the material randomness factor in the dissipative zones; and γ_{sh} is the hardening factor in the dissipative zone.

Eq. (1) is very hard to be respected in the case of ARSWs' typical diagonal-to-upright connection: diagonals are connected directly to uprights through a single bolt. This is allowed by the open U shape of uprights cross-section, where the external lips as those to which diagonal is bolted without using additional sheets (Figure 2).

As previously highlighted, the capacity of these connections is led by bearing resistance $F_{b,Rd}$, which can be evaluated as in (2), according to [7]:

$$F_{b,Rd} = \frac{2.5 \cdot \alpha_b \cdot k_t \cdot f_u \cdot d \cdot t}{\gamma_{M2}} \tag{2}$$

where α_b depends on the geometry of the connection; k_t depends on the thickness of the element; f_u is the ultimate strength of the material of the element; d is the diameter of the hole, and t is the thickness of the element; γ_{M2} is the safety coefficient for checking steel connections. Disregarding the two coefficients α_b and k_t which can be easily maximised to 1.00 (their maximum allowed value), the only ways to increase the bearing resistance of the connection is to increase the thickness of the section, use a material with a higher strength, or increase the number of bolts. The first two possibilities also increase the cross-section's resistance, which implies that the connection's design force is increased according to (1), so the circle keeps turning. If allowed by the geometry of the profile and if the minimum distances for holes are respected, the increase of the number of bolts has to be balanced with the net section check, which gets difficult to be satisfied since the net section reduces. In any case, the only increase of the number of bolts is not sufficient to obtain the over-resistance of the connection. Indeed, the configuration of the connection is not helpful: the bolts can be placed only on the vertical sides of the diagonal, without having the possibility to use the upper side. If the area collaborating for bearing resistance is compared to the whole cross-section of the element (that is entirely mobilized for tension resistance), it can be noticed that the latter is far big than the former, and the use of ultimate stress f_u (that is used to calculate bearing resistance (2), while yield strength is used for tensile resistance) may be not sufficient to make up for this difference. For all these reasons, a local reduction of the cross-section of the bracing is necessary to limit the ultimate tensile resistance of the element and achieve the fulfilment of the (1).

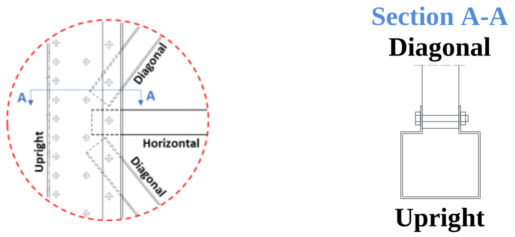


Figure 2: Typical configuration for an ARSWs' diagonal-to-upright connection.

The study of the applicability of the method is performed on an ARSW frame with the same global geometrical characteristics of the case studies previously discussed. Basically, the fixed input parameters are the same, but the free ones, with specific reference to the ones for the definition of the design seismic force, are defined following the "no-discount" policy, meaning that the parameters which are not justified for ARSWs but only for traditional racks are not considered. A **global study** is firstly conducted for the design of the CA frame, which is made by means of a linear response-spectrum analysis and the application of Eurocode 8 [9] capacity design rules for DC2 frames with concentric tension-only bracings, with slight modifications which regard: (i) the design of uprights and (ii) the design of the reduced section for the diagonals.

As regards (i), capacity rules for non-dissipative components are adjusted for the design of uprights only as showed in (3), (4) and (5):

$$N_{Ed} = N_{Ed,G} + \Omega \cdot N_{Ed,E} \tag{3}$$

$$M_{Ed} = M_{Ed,G} + \Omega \cdot M_{Ed,E} \tag{4}$$

$$V_{Ed} = V_{Ed,G} + V_{Ed,E} \tag{5}$$

where $N_{Ed,G}$, $M_{Ed,G}$ and $V_{Ed,G}$ are respectively the axial force, bending moment and shear force due to the non-seismic actions for the seismic load combination; $N_{Ed,E}$, $M_{Ed,E}$ and $V_{Ed,E}$ are those due to seismic action; and Ω is the seismic magnification factor, which in this case is equal to 1.5. In particular, eq. (4) is modified with respect to the original one, where $M_{Ed,E}$ is not amplified by Ω. In case of ARSWs this is necessary because the usually adopted U sections for the uprights are characterized by a low section modulus. Indeed, even if bending force due to seismic action is not high, bending resistance of these sections is limited. Figure 3 show the final configuration of the CA frame with the main characteristics of diagonal ad uprights.

As a preliminary activity to (ii), diagonals connections are designed, meaning that the cross section and thickness of the diagonals is determined in order to satisfy eq. (1), with R_{fy} equal to the design forces acting on the diagonals as resulting from the analysis.

Local optimization is necessary to calculate the amount of reduction of section, its length (to guarantee the desired level of ductility), and to define its layout (to obtain a good performance under cyclic action, considering that this is affected by behaviour in compression). In line with previous research findings [11], reductions are placed at the end sections of the element to affect the less possible the slenderness of the element.

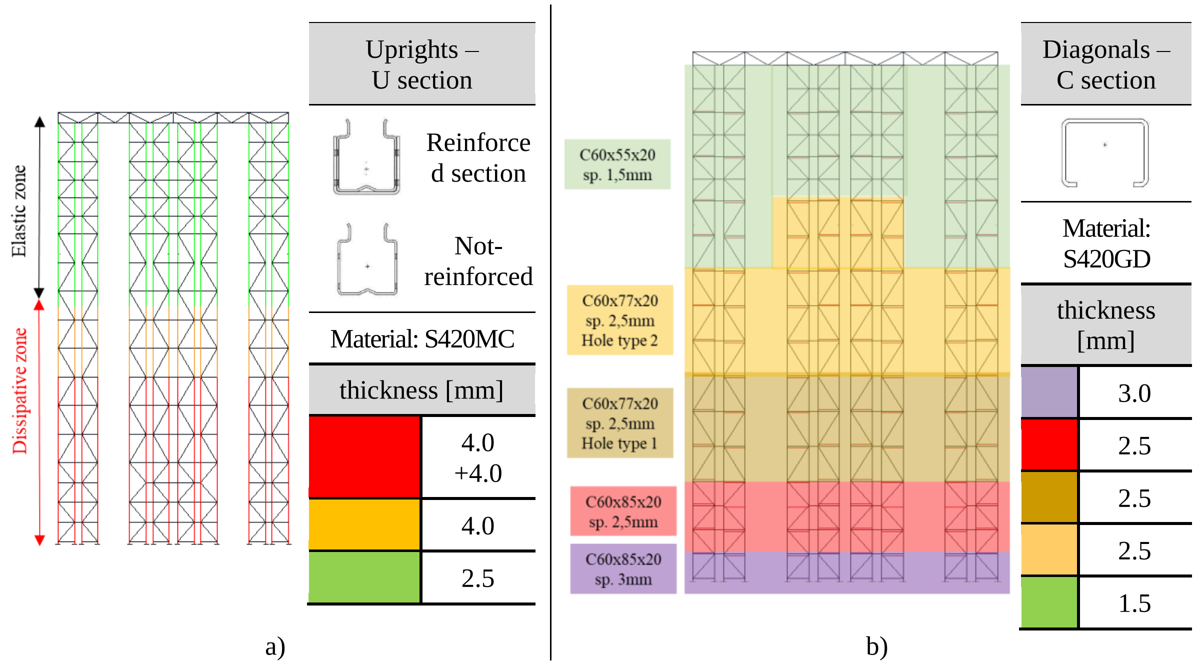


Figure 3: Application of the new strategy to the case study structure: a) frame view, where the dissipative zone and upright cross-sections are indicated; b) frame view, with cross-sections of diagonals indicated.

The optimization of the shape and number of the reduced sections is preformed looking at the behaviour in compression of the element. On one hand, the element with the reductions should have the more similar buckling resistance of the intact element to control and limit the increment of the element's slenderness. On the other hand, the buckling mode should be global, limiting or avoiding, if possible, the local buckling of the reduced parts. This study is performed comparing the behaviours in compression of the intact element and of possible solutions for the reduced section ones, which are obtained through numerical simulations with Abaqus® FEM software. These numerical simulations are performed for all the diagonals, and in the following, the main steps for diagonal D1 (highlighted in purple in Figure 3b), are showed. Figure 4 shows the buckling mode resulting from the numerical simulations on the intact D1 element.

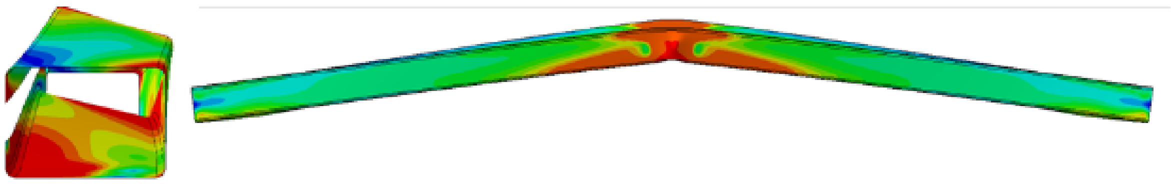
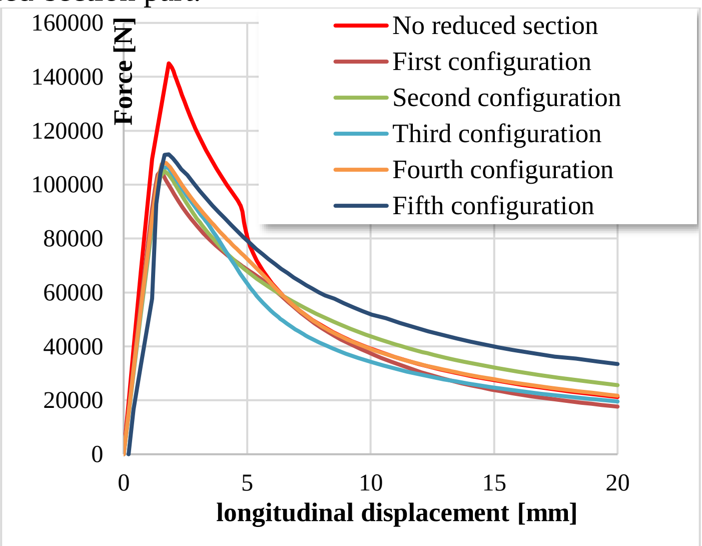
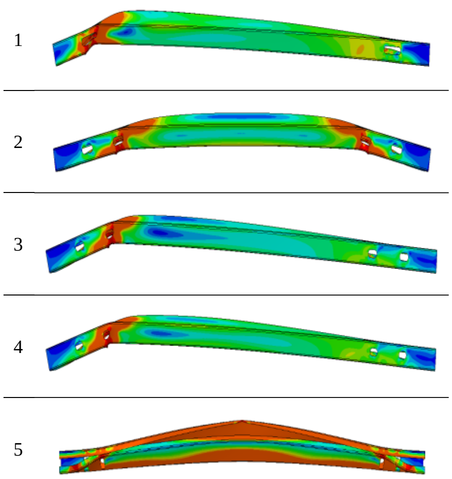


Figure 4: Diagonal D1: buckling mode resulting from the numerical simulations.

The same numerical simulations (with the same modelling strategies) are carried out for the D1 diagonal with the reduced sections. Different configurations for the reduced sections are studied, changing their number and shape. Figure 5a shows the force-displacement curves resulting from the numerical analyses performed for the different configurations. It can be noticed that, although the length, number and shape of the reduced sections change, the buckling resistances of the elements with the reduced sections are comparable with each other but always lower than the one of the intact element. Looking at the buckling modes of the diagonal with the reduced sections (Figure 5b), in the first four configurations deformations and buckling is local and concentrated in the reduced-section part of the elements, differently from the intact diagonal (Figure 4). In contrast, the fifth configuration's buckling mode is closer to the one of the complete diagonal, with a global involvement of the element and a little local buckling in the reduced-section part.



a)



b)

Figure 5: D1 diagonal with reduced sections: a) force vs displacements curves from numerical simulations; b) buckling shapes from numerical simulations.

Although the same resistance of the intact element cannot be reached, the fifth configuration seems to guarantee a better behaviour in compression. Hence, this last pattern is used and applied to all the other diagonals.

3 APPLICATION OF THE METHOD TO THE CASE STUDY STRUCTURES AND EXPERIMENTAL VALIDATION OF THE REDUCED-SECTION DIAGONALS

The new design procedure developed and showed in §2 is applied for the re-design of the case study structures presented in §1. To assess the effective performance of the reduced section diagonals, and so for the final evaluation of the global performance, a diagonal from the most stressed part of each re-designed structure is selected, and for each one 4 different possible layouts for the reduced sections are proposed. Each layout is tested in monotonic tension, monotonic compression, and cyclic load (x2), for a total of 80 tests. The purpose of these test is to compare the structural behaviour corresponding to the different layouts by mean of observing: tensile resistance and ductility; buckling mode, resistance and slenderness of the diagonal; possible resistance and stiffness degradation under cyclic load. Each diagonal is tested in its actual configuration, meaning that the diagonal has the right geometry as in the upright truss (Figure 6): the tested assembly is made of the diagonal connected to two upright pieces. In this way, the respect of the OCS design rules is also checked, also paying attention that the desired behaviour is obtained (no damage in upright-to-diagonal connection). This allows to check if the value of safety coefficient for the design of the over-resistant components has been correctly calibrated. The test set up is represented in Figure 7. The loading protocol is the standard as defined by the ECCS "Recommended testing procedure for assessing the behaviour of structural steel elements under cyclic loads" [13].

In this paper, tests results are showed for one of the selected diagonals and for one layout (L4), which characteristics are represented in Table 1.

Looking at the monotonic tests (Figure 8), the failure mode corresponding to the monotonic tension tests is characterised by yielding and fracture of the brace in the reduced cross-section which is closest to the connection (Figure 8b). No damage in correspondence with the connection is detected, meaning that the over-resistance of connection is guaranteed. Under monotonic compression loading, the failure mode is characterised by local buckling in correspondence of one reduced section (Figure 8c). This is not the general behaviour observed: in general, global buckling occurs first, and then local buckling occurs in the reduced sections mostly affected by the lateral deformations due to the global buckling mode. In this layout, local buckling leads due to the length of each reduced section, which is the highest among all the layouts.

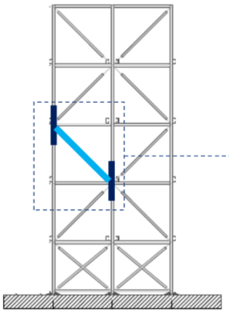


Figure 6: Configuration of the component for tests.

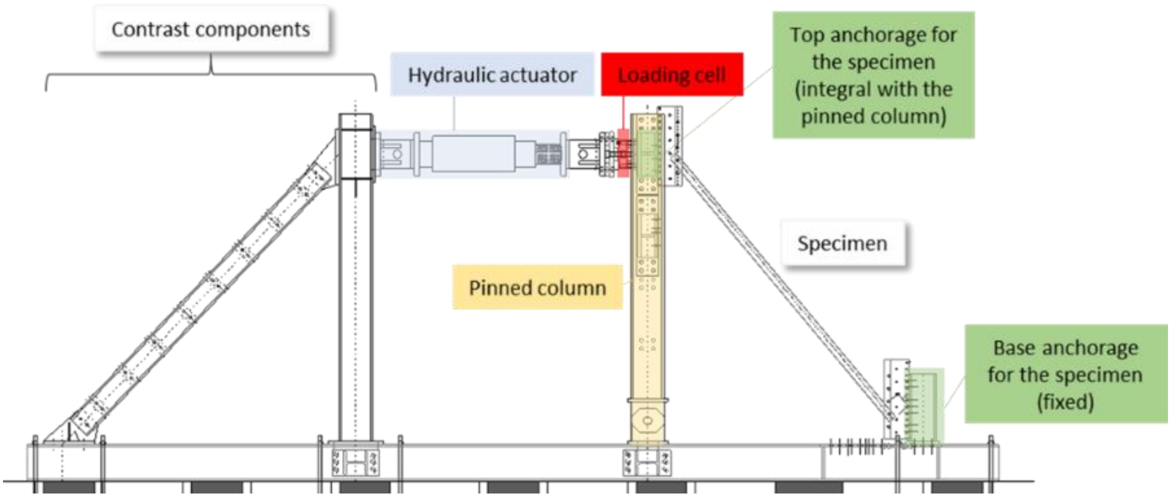


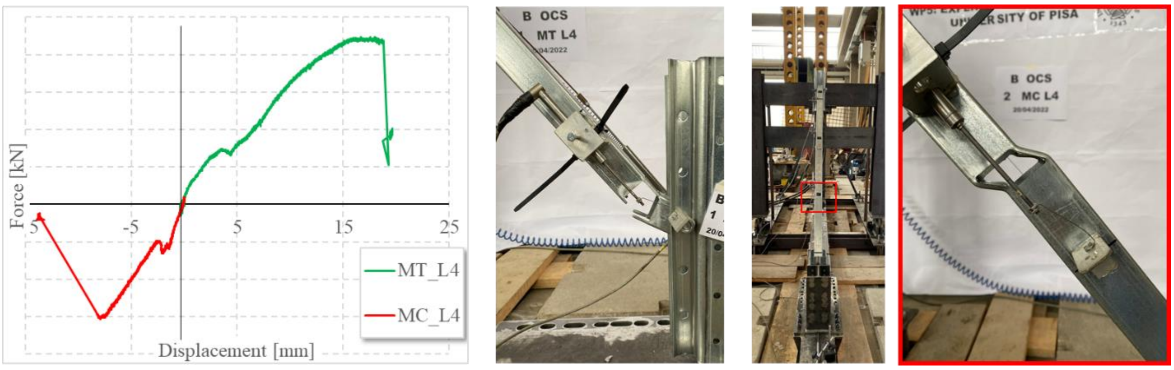
Figure 7: Test set up for OCS diagonals testing.

For cyclic tests (Figure 9) the behaviour is characterised by local buckling of the brace in the region containing the reduced sections, and fracture of the brace in the reduced section (which experienced local buckling, in the most cases, Figure 9b). In any case, no damage in connection is detected.

In general, from the point of view of the failure mode, the OCS design strategy proves to be effective in concentrating the damage only in the brace, and to avoid damages in the uprights and the connection zone. Although a strong reduction of the cross-section of the diagonal may be needed to have an over-resistant connection, the tested layouts allow in general to have global buckling before than local one (this is one of the attended goals), but if the length of the reduced section is too high, local buckling prevails. In any case, when local buckling of the reduced parts occurs, this affects the global behaviour of the diagonals, as proved from the cyclic tests, since failure mostly happens in the reduced section which experienced local buckling. Indeed, the load-unloading cycles may also induce little cracks formation in the locally buckled zone, which drives failure under tension in the so damaged reduced section. Globally speaking, no degradation of resistance or stiffness is observed.

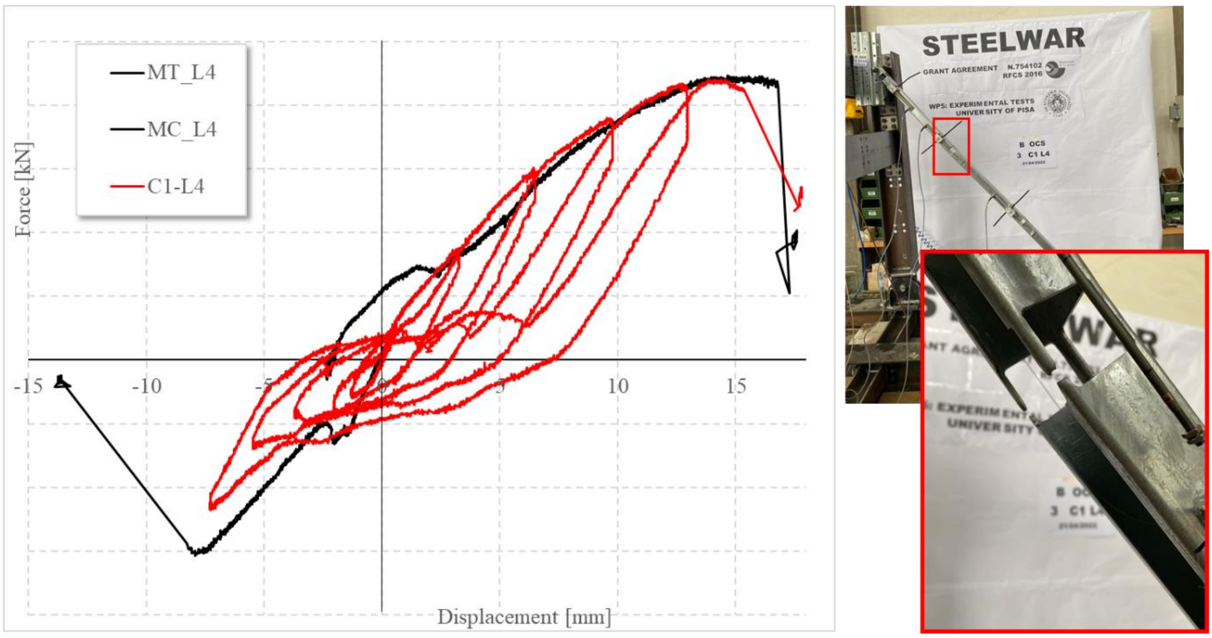
Characteristics of the reduced sections			
Total length [mm]	Distribution	Number	Length [mm]
124	diffused	4	31

Table 1: Characteristics of the reduced sections for the layout L4 of the selected diagonal.



(a) (b) (c)

Figure 8: Monotonic load tests for the L4: a) load vs displacement curves; b) tension load: failure in the reduced section; c) compression load: global buckling and then, local buckling of the reduced section.



(a) (b)

Figure 9: Cyclic load tests for L4: a) load vs displacement curves; b) failure in one of the reduced sections.

4 CONCLUSIONS AND FURTHER DEVELOPMENTS

In this paper, a new design approach for dissipative seismic resistant Automated Rack Supported Warehouses (ARSWs) is presented, named “Over-resistant Connection Strategy”. This method provides for the structure to be dissipative with concentration of dissipation in diagonals, and it is applied to X tension-only structural schemes by adopting the Eurocode 8 prescriptions for medium-Ductility Class DC2 structures [9], with slight modifications to meet ARSWs’ structural peculiarities. Since dissipation is expected in the diagonal element with all the other components designed to be over-resistant, the only way for the typical diagonal-to-upright connection to be over-resistant with respect to diagonal is to locally decrease the resistance of the diagonal by introducing some section reductions. The approach is firstly numerically studied and applied on a case study structure, with local studies around the better layouts for the reduced sections. Then, the actual behaviour of the reduced section diagonals is experimentally validated through an extensive experimental campaign on braces with different layout configurations. As resulting from this activity, the OCS design strategy proves to be effective in concentrating the damage only in the brace, and to avoid damages in the uprights and the connection zone. Although some diagonals have a strong reduction of the cross sections, the behaviour in compression is quite good, with global buckling anticipating local buckling of the reduced sections, in general. If the length of the reduced section is too high, local buckling prevails, ad happening for the showed layout L4. Under cyclic load, the local instability of the reduced section may induce, with the alternance of the cycles, the formation of cracks in correspondence of the reduced sections and the consequent failure under tension load in correspondence of the damaged section. A better compromise between connection resistance and section reduction, with a lower reduction of section, may allow a better cyclic behaviour, avoiding completely the local buckling of the reduced sections. The further step of this research is the assessment of the actual global performance of the structures, as showed in §2.2 but with the calibration of the behaviour of the diagonals on the one obtained in the experimental campaign.

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REFERENCES

[1] BS EN 15512:2020 Steel static storage systems. Adjustable pallet racking systems. Principles for structural design. (2020).

[2] BS EN 16681:2016 Steel static storage systems. Adjustable pallet racking systems. Principles for seismic design. (2016).

[3] Caprili, S., Morelli, F., Salvatore, W. and Natali, A. Design and Analysis of Automated Rack Supported Warehouses. *The Open Civil Engineering Journal* (2018), vol. 12, no. 1, pp. 150–166.

[4] Tsarpalis, D., Vamvatsikos, D., Delladonna, F., Fabini, M., Hermanek, J., Dot Morgotan, P., Sesana, S., Vantusso, E. and Vayas, I. Macro-characteristics and taxonomy of steel racking systems for seismic vulnerability assessment. *Bulletin of Earthquake Engineering* (2022).

[5] Di Ludovico, M., Digrisolo, A., Graziotti, F., Moroni, C., Belleri, A., Caprili, S., Carocci, C., Dall'Asta, A., De Martino, G., De Santis, S., Ferracuti, B., Ferretti, D., Fiorentino, G., Mannella, A., Marini, A., Mazzotti, C., Sandoli, A., Santoro, A., Silvestri, S., Sorrentino, L., Magenes, G., Masi, A., Prota, A., Dolce, M. and Manfredi, G. The contribution of ReLUIS to the usability assessment of school buildings following the 2016 central Italy earthquake. *Bollettino di Geofisica Teorica ed Applicata* (2017).

[6] Natali, A., Morelli, F. and Salvatore, W. On the Seismic Design and Behavior of Automated Rack Supported Warehouses. *Bulletin of Earthquake Engineering* (2022). doi: 10.21203/rs.3.rs-1577721/v1. Under review.

[7] EN 1993-1-3:2006 Eurocode 3: Design of steel structures - Part 1-3: General rules - Supplementary rules for cold-formed members and sheeting. (2006).

[8] Rosin, I., Calado, L., Proença, J., Carydis, P., Mouzakis, H., Castiglioni, C., Brescianini, J. C., Plumier, A., Degee, H., Negro, P. and Molina, F. Seismic behaviour of steel storage pallet racking systems (SEISRACKS2): final report. Luxembourg: Publications Office. (2014).

[9] EN 1998-1:2004: Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings. 2004.

[10] Braconi, A., Caprili, S., Degee, H., Guendel, M., Hjaij, M., Hoffmeister, B., Karamanos, S. A., Rinaldi, V. and Salvatore, W. Efficiency of Eurocode 8 design rules for steel and steel-concrete composite structures. *Journal of Constructional Steel Research* (2015), vol. 112, pp. 108–129.

[11] Moen, C. D. and Schafer, B. W. Elastic buckling of thin plates with holes in compression or bending. Thin-Walled Structures (2009).

[12] EN 10149-2 Hot rolled flat products made of high yield strength steels for cold forming – Part 2: Techical delivery conditions for thermomechanically rolled steels. (2013).

[13] ECCS – Technical Committee 1 – Structural Safety and Loadings Technical Working Group 1.3 – Seismic Design. Recommended testing procedure for assessing the behaviour of structural steel elements under cyclic loads. (1986).

[14] Mazzoni, S. The open system for earthquake engineering simulation (OpenSEES) user command-language manual. (2017).

[15] Caprili S., Mattei F., Salvatore W. Seismic performance of innovative dissipative replaceable components for steel braced frame (DRBRC). *Eccomas Congress*, 5-9 June 2022, Oslo, Norway.