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Advanced seismic design approach for Automated Rack Supported Warehouses

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Abstract

Automated Rack Supported Warehouses (ARSW) are huge steel buildings offering storage solutions. They are the latest in this field. When the number of pallets is relevant and the daily handling operations are numerous, they offer the best options for goods flow management and space optimization. They constitute the direct upgrade of traditional pallet racks, but, differently from these, the shelves do bear the weight of the pallets and are the warehouse's structure. To follow the fast-evolving market request, ARSWs acquired most of the racks' structural features without being supported by a specific regulatory framework. This lack brought with time to relevant catastrophes that highlight the lack of knowledge that concerns these structures. The absence of proper prescriptions to be followed to the design of ARSWs results, in most cases, in the adoption of the same guidelines defined for traditional steel racks (UNI EN15512 (2009) and UNI EN16681 (2016)) and to the adoption of the same structural choices and technical solutions. As an example, from the global point of view, the same structural schemes are adopted, and, from the local point of view, the same material and cross-sections of structural elements, as well as the same structural details as connections.

In this framework, this PhD thesis shows a possible new approach for the design of ARSWs. In particular, based on an accurate evaluation of safety levels and the design strategies now adopted in current practice, this new design approach focuses on seismic conditions, dealing specifically with Double-Depth structural typology. This approach is defined by assuming a dissipative behaviour and evaluating different and possible yielding patterns as an alternative to the global collapse mechanism, where the whole structure is involved. The optimization of the cost-benefit ratio is always considered as one of the design goals. The cost-benefit ratio consists of costs connected to a significant variety of construction details that may be implied by capacity design, while the benefits are related to dissipative behaviour that allows obtaining controlled yielding pattern and lighter structures. The study and the analysis of ARSWs are widely performed inside the European research project "STEELWAR: Advanced structural solutions for automated STEEL rack supported WARhouses" funded by the Research & Innovation, Research Fund Coal and Steel (RFCS) and coordinated by the University of Pisa. Thanks to this project, a series of experimental tests and research will be executed to support the results of the present PhD thesis. Besides university institutes, the research group participating in this project is formed by two engineering companies with specific competencies on design and inspection of rack systems and knowledge of the logistic industry, by five big rack producers (selling their rack systems solutions in Italy, Europe and Overseas), and by a supplier of storage technology systems.

Preliminary studies are executed, dealing with analysing the more suitable typologies of numerical analyses for the structural assessment of double-depth ARSWs structures. In particular, an ARSW case study is designed following Eurocode 8 directions, both adopting elastic and dissipative approaches. The final aims of this study are: (i) evaluate the applicability of Eurocodes prescriptions for steel buildings to ARSWs, focusing in particular to the capacity design rules; (ii) give a comparison of the two design approaches in terms of structural performances, also looking at the post-elastic behaviour of the two structures; (iii) evaluate the more suitable and efficient non-linear structural analysis for ARSWs. The methodology adopted to achieve these aims is organised in the following steps: (1) execution of non-linear numerical analyses of both elastic and dissipative structure, using lamped plasticity models; (2) implementation of the contribution of braces in compression within the non-linear numerical analyses

of the dissipative structure only. The findings of these preliminary analyses are used in the following steps, especially in the final part, when the numerical assessment of the structures designed with the new design approach is carried out.

The structural assessment of five case studies is performed to comprehend the existing structures' current design strategies. These case studies are designed in a high seismicity area by the Industrial Partners that participate in STEELWAR research project. The structural assessment of these structures is done through 3D or 2D models – based on the possible and doable simplification of the geometry of the system - and executing dynamic analysis, where the seismic input is defined according to natural accelerograms that are selected from the available database to obtain the worst damage scenario for the structural typologies considered, in relation to the seismic intensity level considered. Besides the structural assessment, the analysis of the design strategies currently adopted is executed to point out the positive or questionable aspects. The results obtained from the structural assessment and the analysis of the current design strategies are used as a starting point to define the possible strategies to optimize the design of ARSWs.

A new design approach is developed starting from the critical issues found in the analysed structures. The starting design rules are in line with those within Eurocode 8 (prEN 1998:2019). Firstly, an optimization at a global point of view is made, focusing on reducing eccentricities and all possible geometrical aspects that may negatively influence its structural behaviour. Then, the design inputs are discussed – as, for example, the definition of loads and participating masses - to point out the righter strategy. The more appropriate structural typologies to assure the desired dissipative structural behaviour are individuated. In particular, the possibility of the lower part of the structure only being involved in the plastic mechanism is studied. In this lower part, the more restrictive rules corresponding to the medium seismicity class – as defined by Eurocode 8 (prEN 1998:2019) – are applied. Finally, based on the previous design issues, optimization at local point of view is made, focusing on elements and structural details.

The structural assessment of the re-designed structure is carried out with non-linear dynamic analyses on 2D models, considering both geometrical non-linearities and dissipative elements' structural behaviour. The other elements are modelled as elastic, and they are checked in the post-process through the safety checks. A critical analysis of the new design approach is made through the results obtained from the non-linear analyses' execution.

Contents

1. Introduction

Automated Rack Supported Warehouses (ARSW) are huge steel buildings offering storage solutions. They are the latest in this field. When the number of pallets is relevant and the daily handling operations are numerous, they offer the best options for goods flow management and space optimization. They constitute the direct upgrade of traditional pallet racks, but, differently from these, the shelves do bear the weight of the pallets and are the warehouse's structure. To follow the fast-evolving market request, ARSWs acquired most of the racks' structural features without being supported by a specific regulatory framework. This lack brought with time to relevant catastrophes that highlight the lack of knowledge that concerns these structures. The absence of proper prescriptions to be followed to the design of ARSWs results, in most cases, in the adoption of the same guidelines defined for traditional steel racks (UNI EN15512 (2009) and UNI EN16681 (2016)) and to the adoption of the same structural choices and technical solutions. As an example, from the global point of view, the same structural schemes are adopted, and, from the local point of view, the same material and cross-sections of structural elements, as well as the same structural details as connections.

In this framework, this PhD thesis shows a possible new approach for the design of ARSWs. In particular, based on an accurate evaluation of safety levels and the design strategies now adopted in current practice, this new design approach focuses on seismic conditions, dealing specifically with Double-Depth structural typology. This approach is defined by assuming a dissipative behaviour and evaluating different and possible yielding patterns as an alternative to the global collapse mechanism, where the whole structure is involved. The optimization of the cost-benefit ratio is always considered as one of the design goals. The cost-benefit ratio consists of costs connected to a significant variety of construction details that may be implied by capacity design, while the benefits are related to dissipative behaviour that allows obtaining controlled yielding pattern and lighter structures. The study and the analysis of ARSWs are widely performed inside the European research project "STEELWAR: Advanced structural solutions for automated STEEL rack supported WARhouses" funded by the Research & Innovation, Research Fund Coal and Steel (RFCS) and coordinated by the University of Pisa. Thanks to this project, the present PhD thesis results will be supported by a broad experimental campaign. Besides university institutes, the research group participating in this project is formed by two engineering companies with specific competencies on design and inspection of rack systems and knowledge of the logistic industry, by five big rack producers (selling their rack systems solutions in Italy, Europe and Overseas), and by a supplier of storage technology systems.

This PhD thesis is organized in the following Chapters.
Chapter 2 shows and describes ARSWs main structural typologies and technical solutions, highlighting the common and the different aspects concerning traditional steel racks.
Chapter 3 gathers all the research, guidelines and regulations regarding the design of ARSW, the study of their structural behaviour from global and local point of view.
Chapter 4 shows the lacks and the issues that are now missing or hardly developed around ARSWs, focusing on the structural field.
Chapter 5 gathers the objectives of this PhD Thesis and the methodology adopted to reach these objectives.

Chapter 6 deals with the preliminary studies that have been carried out aiming to: (i) evaluate the applicability of Eurocodes prescriptions for steel buildings to ARSWs, focusing in particular to the capacity design rules; (ii) give a comparison of the two design approaches in terms of structural performances, also looking at the post-elastic behaviour of the two structures; (iii) evaluate the more suitable and efficient non-linear structural analysis for ARSWs. These findings are used in the following, especially in the final part of this study, when the numerical assessment of the structures designed with the new design approach is carried out.

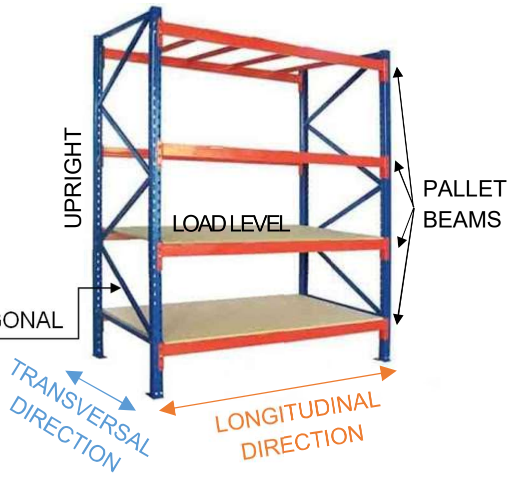
Chapter 7 deals with the analysis of the current design strategies and structural behaviour of ARSWs. This study is performed by analysing five case studies designed in a high seismicity area by the Industrial Partners that participate in the STEELWAR research project. The outcomes of this part highlight positive or questionable aspects concerning both the design phase and the resulting structural behaviour. These results are used as a starting point to define the possible strategies to optimize the design of ARSWs.

Chapter 8 deals with the development of a new design approach for ARSWs, starting from the critical issues that are illustrated in the previous steps, and going from a global to a local optimization: the more suitable structural typologies for the aimed dissipative structural behaviour are individuated, and then, local optimization is carried out to guarantee that the capacity design rules are observed, as well as the over-resistance of the non-dissipative elements towards the dissipative ones. Numerical assessment of the re-designed structure is performed with Non-Linear Time History analyses, whose results are presented and critically analysed.

Chapter 9 recalls all the work done, highlighting the outcomes and the possible future developments.

2. Automated Rack Supported Warehouses

Large scale economy and trade are widely diffused worldwide, involving both smaller and bigger realities. Large quantities of goods can be produced or bought, and some time can pass till they may be distributed or sold, resulting in the need for private stocking areas. Simultaneously, factories may need significant storing places where their stuff can be placed and easily collected or handled. As a traditional solution to these necessities, steel racks have been available from the last decades of the XX century: they are mostly made up of repeated modular shelves, very easy to be assembled, changed in geometry and removed, where goods can be placed after being organized and gathered in pallets (Figure 2-1). The shelves are entirely made up of steel elements that are arranged in a truss. Each truss is composed of vertical profiles, called "uprights", connected by diagonal elements. Two consecutive trusses are connected by couples of beams, where the pallets are placed. Indeed, they are called "pallet beams" in the sector jargon (Figure 2-1a). This is the main structure of a shelve that sustains the vertical loads, as initially designed. Then, after some collapses of the system due to seismic action (Figure 2-2) and several research studies about this issue (1) - (2), the structure of shelves has been re-arranged to give stability also towards horizontal loads. With this aim, plane bracing at each load level and vertical bracing along longitudinal direction have been added (the one where the pallet beams are placed).



a)

b)

Figure 2-1: Traditional steel racks: figure a) shows a shelf module, and figure b) shows a possible arrangement of the modules inside a stocking warehouse (italian.industrial-storagerack.com).

Figure 2-2: Collapse of some racks with parmesan wheels, placed inside a warehouse in Emilia Romagna, after the "Emilia earthquake" took place on the 20th of May in 2012.

Traditional shelves represent the integral part of industrial logistics. They are widely used, mainly because of their assembly's simplicity, which is allowed by specific technological components that have been studied with this aim. As an example, beam-to-uprights connections are, in most cases, not traditional (neither bolted nor welded) but made to allow an easy and fast-to-be-assembled connection. Besides, they are highly versatile, being suitable to multiple fields (industrial, commercial, etc.), and since the management of the handling of pallets inside the warehouse can be entrusted to specific mechanical devices as forklifts, to be driven by the employed responsible personnel (Figure 2-3). Anyway, this solution is indisputable optimal only when the number of pallets to be stored is limited because of the following reasons:

- The height of the traditional steel racks can be, in general, very high. Still, when it overcomes the limit of 20-25 m, the warehouse structure's realisation where the racks are placed could get very expensive and constitute the relevant main cost. Simultaneously, if the number of pallets is high, the limit of the racks' height could lead to the necessity of an increased number of shelves, so a larger area. As a consequence, also in this case, the structure of the warehouse could get very expensive to be realized.
- In connection to the previous point, traditional racks do not allow the full exploitation of the warehouse's available space. Indeed, the bigger is the number of pallets, the bigger is the number of load levels (to be limited) and the necessary plan area. Finally, the higher gets the price to realize the warehouse structure. The bigger the system gets, the more different the warehouse is from ordinary industrial buildings: structural requirements may be harder to be fulfilled, structural details get more sophisticated, and the amount of steel for primary elements grows.
- Traditional racks do not allow the automated handling of goods. Consequently, if the pallets' daily flow within the warehouse is high, the handling of pallets could get complicated to be managed, time-consuming, and it may slow down the planned activities.

Figure 2-3: A forklift used to handle pallets inside a warehouse (https://safestart.com).

The limits connected to the use of traditional racks when a high number of pallets have to be stored and the necessity of wider and optimized spaces led to the development of storage technology, consisting of Automated Rack Supported Warehouses (ARSWs) (Figure 2-4). This new structural solution consists of a building (that can also be huge, getting more than 100 m long and 40 m high if necessary) characterized by two main features: the first one is that the function of racks is not limited to store goods, but the racks themselves constitute the structure of the warehouse; the second one is about the handling of goods, that can be totally automated. Being the racks the primary structural system of ARSWs, they have to support the goods weight, all the environmental loads and the non-structural elements. In ARSWs, racks become the structure itself and cannot be considered independent pieces of equipment placed inside a building. Besides, the pallets' automated movement inside the warehouse is made through specific devices that take each pallet and put it on a particular available free spot based on the warehouse's logistic rules. These governing logistic rules are chosen from the very beginning of the structure's design and are defined after the analysis of the client's market. The logistic of the warehouse is one of the most defining parameters for choosing the warehouse's structural organisation.

The main advantages in adopting ARSWs are usually recognized to be the following:

(i) the automated handling of goods (from the collection at the entrance, the placement, and finally to the exit from the warehouse) allows for higher quality and quantity of service, making it easier to handle more elevated amounts of pallets. This results in an increment of performances of the warehouse and a decrease of mismanagement of the goods flow;

(ii) ARWSs allow high exploitation of the available space, increasing the storage density. Indeed, the full height of the warehouse can be used, and stocking-free areas are limited to those necessary for the movement of the devices for the goods handling;

(iii) since the handling of goods is automated, the personnel to be employed is limited and mainly finalised to the management of the pallets to be taken inside or outside the warehouse;

(iv) all the goods that enter the warehouse are controlled and uniquely identified, so they are entirely traceable;

(v) the construction periods are relatively short, especially considering that the structure can reach quite big dimensions, not comparable to the ordinary buildings. The assembly of elements is quite fast since all the modular pieces come to the construction site already welded, only to be fastened by bolts: each shelve is usually built on the ground and then lifted through cranes;

(vi) since the shelves also constitute the structure of the warehouse, there is no interference between the two, while this may happen in traditional racks, and so, in this case, this parameter needs to be considered in the design phase.

Anyway, there are also some disadvantages:

(i) the structure is hard to be modified after being built. As an example, it could be pretty tricky to place pallets with different dimensions or weight than those initially taken into consideration;

(ii) changing the intended use of the warehouse is not possible, precisely because the racks are the load-bearing structure of the warehouse. If they are removed, the cladding, the lateral panels, and all the facilities have to be removed.

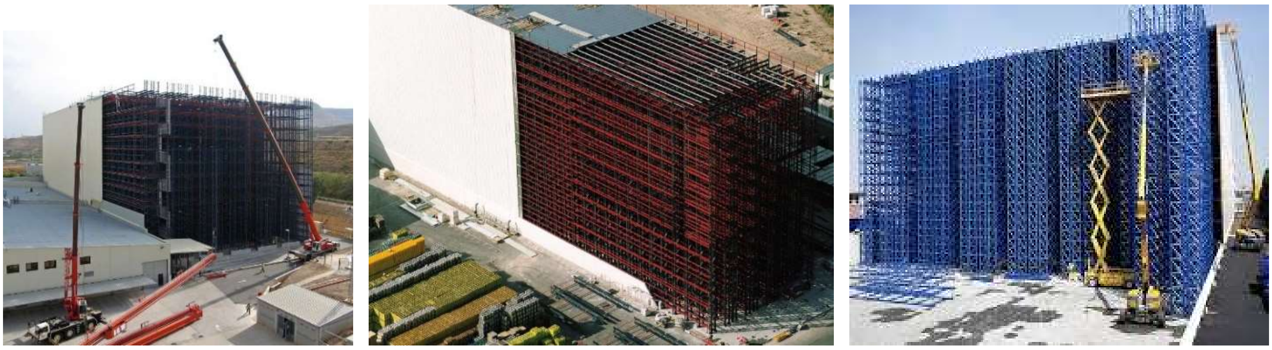


Figure 2-4: Some examples of existing ARSWs (https://mecalux.it/ for the two figures, and https://sacmaspa.it/ for the last one).

Looking at a plan and a transversal view of a generic ARSWs, it can be noticed that two main directions can be individuated: Cross Aisle one (CA) and Down Aisle one (DA) (Figure 2-5).

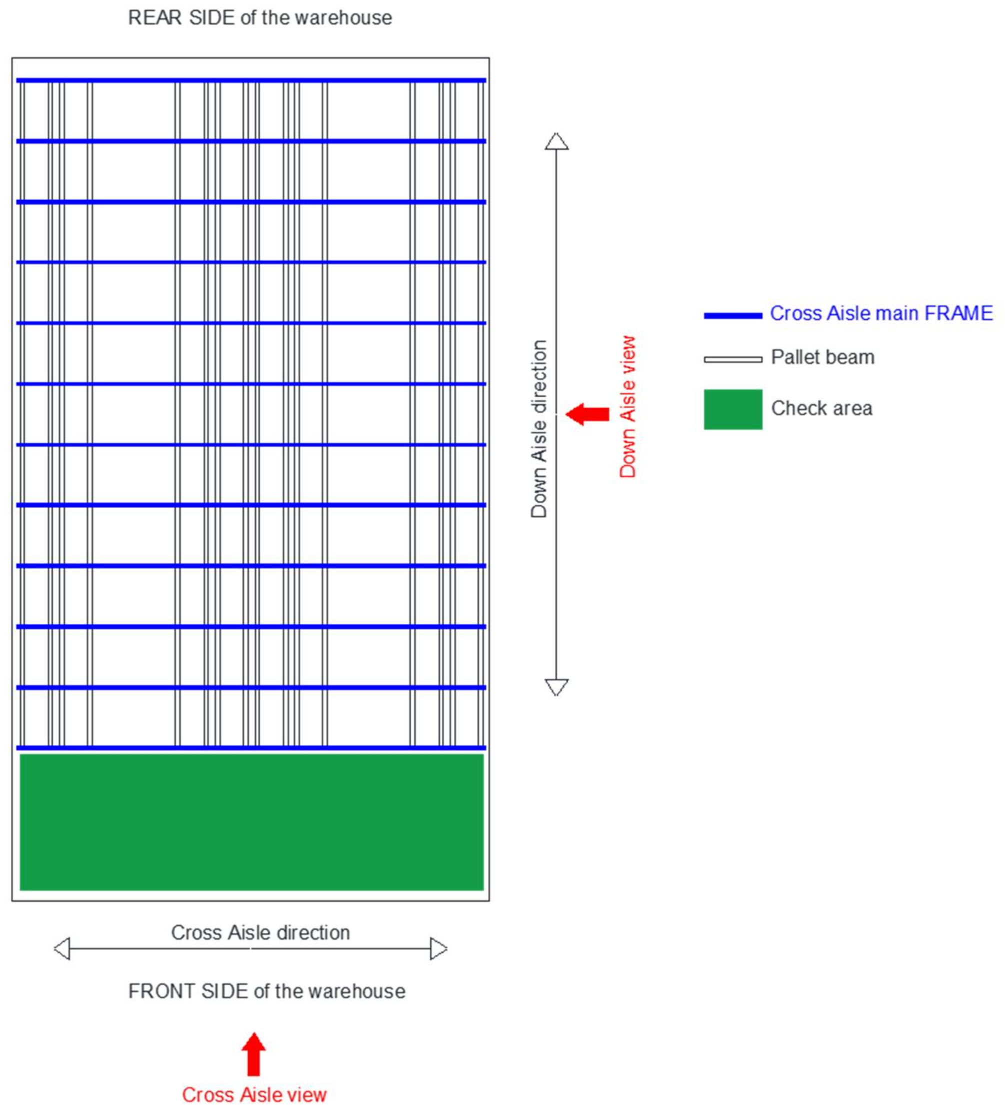


Figure 2-5: Plan view of a warehouse.

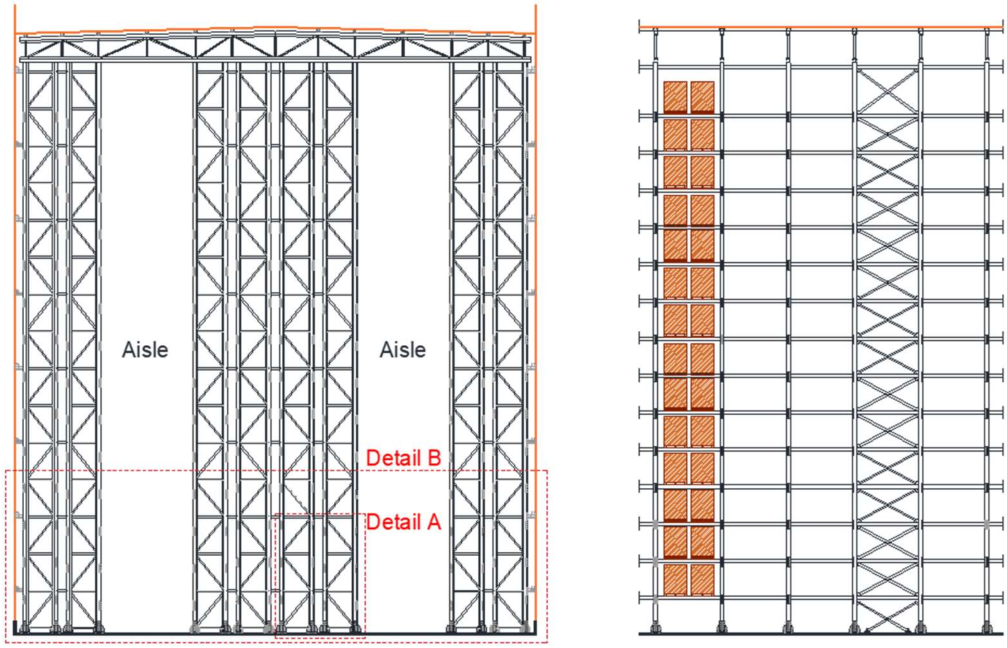
Along the CA direction, the shelves develop, constituting the transversal frame of the structure. This frame is then repeated (usually) equally along the DA direction, and the pallet beams assure the connection of each one to the following one. Different horizontal forces resisting solutions can be added to the main structure, especially along the DA direction. In this direction, uprights can be considered hinged to the foundation and the pallet beams are connected to them through semi-rigid connections, resulting in a very flexible structural scheme.

Among ARSWs category, two main different structural typologies can be individuated:

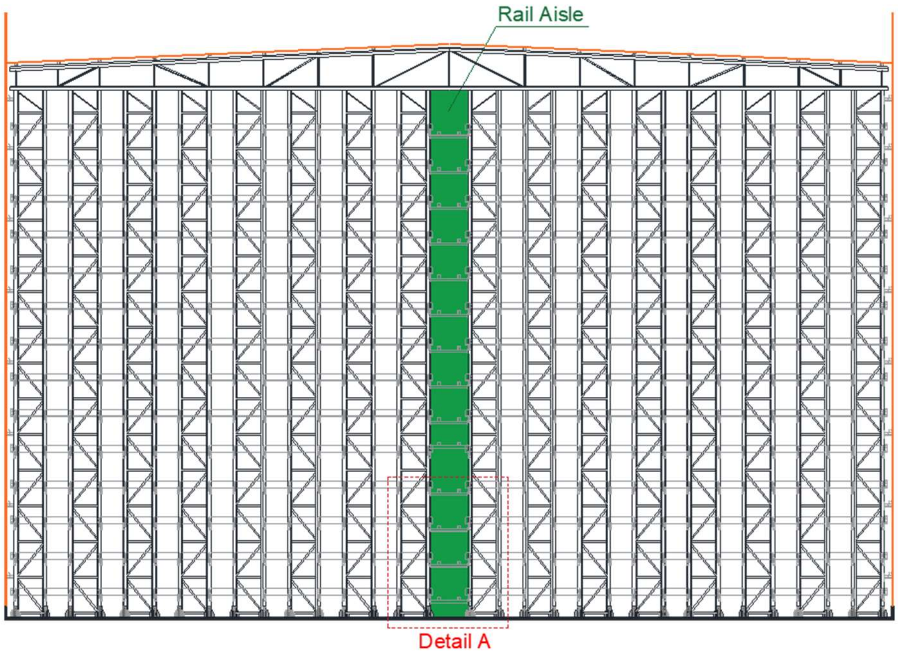
1. Single or Double Depth warehouses (Figure 2-6a): from the functional point of view, each shelve is accessible from one side only. The consecutive shelves can be up to four in case of double-depth and 2 in case of single-depth. The groups of successive shelves are divided by aisles, where the device to handle goods (called "stacker crane") goes along. From the structural point of view, this results in no connection of the shelves along CA direction, except for the base (the uprights are all fixed to the foundation) and the top, where the structure of the roof develops.

2. Multi-Depth warehouses (Figure 2-6b and c): in this case, the distribution of goods is made through another device called "shuttle" that moves on rails. There is an (or more) aisle, highlighted in green within the CA view illustrated in Figure 2-6b, where there are a couple of rails for each load level that allows the shuttle to move along DA direction. Then, to access a specific position within the racks, the rails are also placed on the pallet beams, directed perpendicular to them. This functional layout implies that, contrary to double-depth warehouses, all the shelves are connected punctually along their length at the load levels.

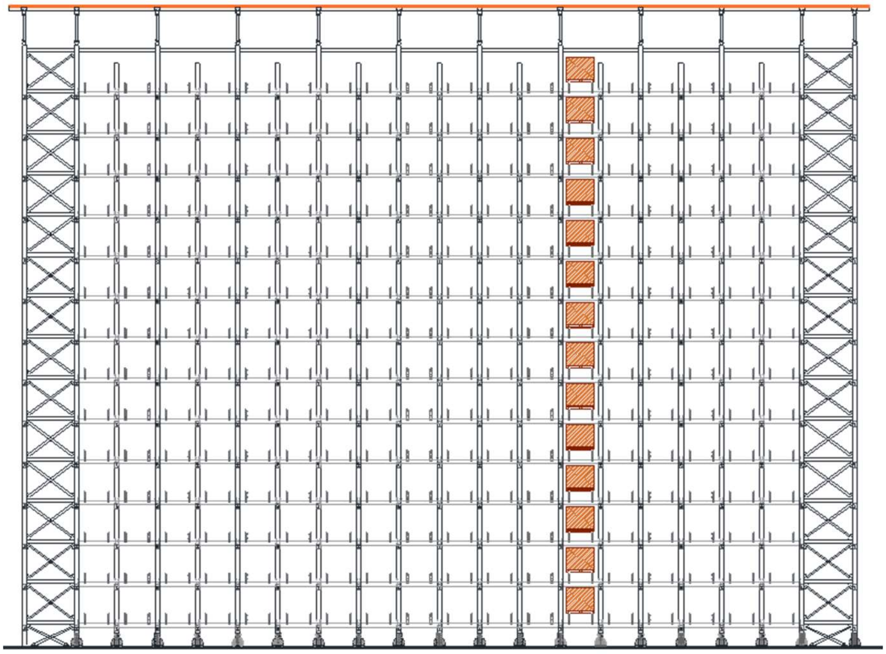
In the following paragraphs, firstly, a list of recurrent field terminology is given. The two structural typologies previously introduced are then analysed from the structural point of view, giving details of the current structural choices adopted for their realization, both from global and local perspective.



a) Typical CA and DA view of a double-depth warehouse.



b) Typical CA view of a multi-depth warehouse.



c) Typical DA view of a multi-depth warehouse.

Figure 2-6: Typical Cross Aisle (CA) and Down Aisle (DA) views of double-depth and multi-depth warehouses.

2.1. Recurrent nomenclature

ARSWs are generally composed of repeated modules made of different elements that a specific nomenclature can uniquely individuate. In the following, a list of useful terms that are typical of this field is given:

- *Pallet*: this is a way to collect all together many items giving them a regular shape that can be box-shaped, with a square base, or cylindrical (Figure 2-7).
- *Single*, *double* and *multi-depth* warehouses: they represent different solutions for storage warehouses. Single and double-depth warehouses are characterized by shelves accessible from one side only. Depending on the consecutive number of shelves between the aisles, from each accessible side, one or two couples of pallet beams can be available to place the pallets. In the case of one pair of beams available only, the warehouse is called single-depth, while if there are two couples of pallet beams, the warehouse is a double-depth one. In double-depth warehouses, the couple of beams nearest to the aisle constitute the 1st depth, while the other one is the 2nd depth. As an instance, Figure 2-8 depicts an example of a double-depth warehouse, where from *Aisle 1,* only *shelves 1, 2, 3* and *4 (S1, S2, S3,* and *S4)* are accessible, and S2 and S3 represent the 1st depths, being immediately overlooking the aisle, while S1 and S4 constitute the 2nd depths. Likewise, from *Aisle 2,* the accessible shelves are S5, S6, S7 and S8, where S6 and S7 represent the 1st depths and the other two the 2nd depths. The Single-depth warehouse is similar but without the 2nd depth (so no S1, S4, S5 and S8). Finally, the multi-depth warehouses have all the shelves mutually connected (Figure 2-6b), and so, many depths as the number of shelves are available and usable.
- *Cross* and *down aisle* (CA and DA) directions (Figure 2-5): they are the main direction of a warehouse, and they refer respectively to the transversal and the longitudinal ones. Along CA direction, the main frame can be individuated, constituted by repeated shelves. Along the DA direction, the main frames are connected through pallet beams, where pallets are placed.

- *Front* and *rear side* of a warehouse (Figure 2-5): these sides of the warehouse are placed along CA direction, and the former is where the pallets enter the structure, the latter is where they go out.
- *Logistic strategies:* they constitute the rules that are assumed inside a warehouse to choose the right spot to place a pallet that comes inside the structure. The variables that define these rules are: the measures of the pallet (height, weight), the content of the pallet, the handling class (that depends on how much time the pallet have to be kept inside the warehouse), the structural limits of the main structure (each upright is designed for a maximum vertical load that cannot be exceeded, and each load level can be designed assuming different maximum weight for the pallets and total height). In general, one of the most common and basic logistic rules that can characterize a warehouse is the following: each shelve has to be filled going from the front to the rear side, and then from the base to the top (this rule should help to distribute the pallets regularly, without creating too many differences along the height of the structure between the two opposite areas of the warehouse and limiting torsional effects due to load distribution).
- *Department*: this is the general name to individuate an area of the warehouse with specific functions. For example, there can be the check area (see below), the stocking area, where the pallet beams are placed, etc.
- *Check area* (Figure 2-5)*:* before being placed in a specific spot, each pallet is measured and weighted. This takes place in the check area, which is located at the entrance (front side) of the warehouse.
- *Shelve* (Figure 2-9): it is the single load-resisting module that composes the CA frame of the warehouse. Each shelve is repeated more times along that direction. The consecutive shelves can be connected punctually along their height one to another, but based on the structural typology of the warehouse, not all the shelves may be mutually connected (except for the base, where they are fixed to the foundation, and the top, where they are connected through the roof beam). For example, in double-depth warehouses, being necessary to leave the aisles clear to leave the stacker crane to move, only the consecutive shelves can be connected. Those separated by the aisle cannot. Differently from this case, in multi-depth warehouses, all the shelves can be connected along their height. Actually, this connection is necessary to allow the shuttle to move inside the racks.
- *Upright* (Figure 2-9)*:* it is the column of the shelve. It is usually made of a cold-formed-sectioned element. The cross-section is typically open, aiming for an easy-to-assembly connection to the diagonals and the horizontal beams composing the shelve.
- *Diagonal* (Figure 2-9)*:* The name indicates the diagonal element composing the shelf along the CA direction. It is usually made of a cold-formed-sectioned element. Diagonals can be arranged in different configurations, based on the assumed seismic-resistant structural scheme.
- *Horizontal beam* (Figure 2-9)*:* it is the name indicating the horizontal element composing the shelve along CA direction. It is usually made of a cold-formed-sectioned element.
- *Base connection* (Figure 2-9)*:* it is the name indicating the connection of the upright to the foundation.
- *Pallet Beam* (Figure 2-9)*:* it is the element upon which the pallets are placed. These elements develop along DA direction, connected to the columns of two consecutive CA frames. It is usually made of a cold-formed-sectioned element.

- *Beam-to-upright connection* (Figure 2-9): this is the name of the connection of pallet beams to uprights. This kind of connection can be traditional (bolted, welded) or, more frequently, atypical, meaning different from the traditional ones but explicitly created for racks. In fact, aiming to fast-to-be-assembled, adjustable and movable connection, specific connections have been made, as, for example, hooked ones. Hooked connections are characterized by an end-plate that is welded to the profile on the inner side, while in the outer one, there are some hooks (steel pieces) that go directly inside the holes of uprights.
- *Stacker crane and shuttle* (Figure 2-10): they are devices that allow the automatic handling of goods; the former (Figure 2-10a) is used in double-depth warehouses and moves along its assigned aisle. Each stacker crane can serve only the shelves that are placed along the aisle where they move. Shuttles (Figure 2-10b) are used within multi-depth warehouses and move on rails, that are made of steel profiles and placed on pallet beams (the direction of the rails is perpendicular to the pallet beams). Each shuttle can reach any part of the warehouse, depending on the path designed for the rails.

Figure 2-7: An example of a box-shaped palletized good (https://www.mecalux.it/).

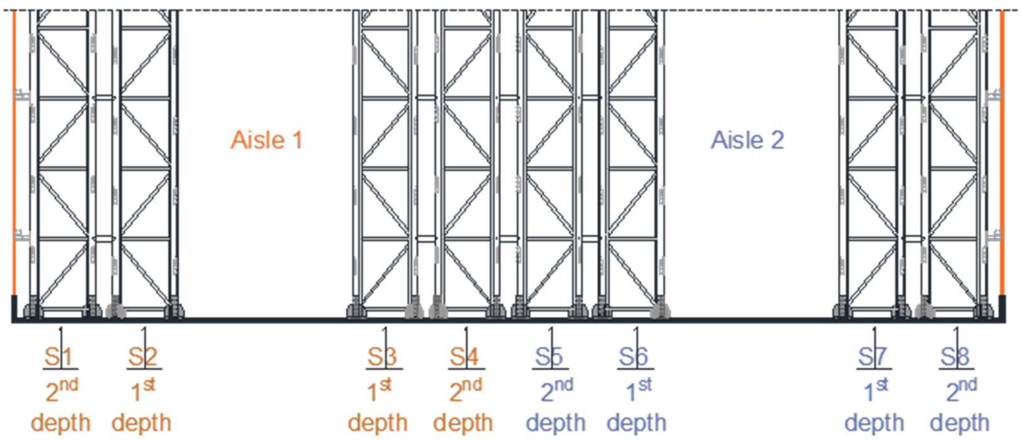
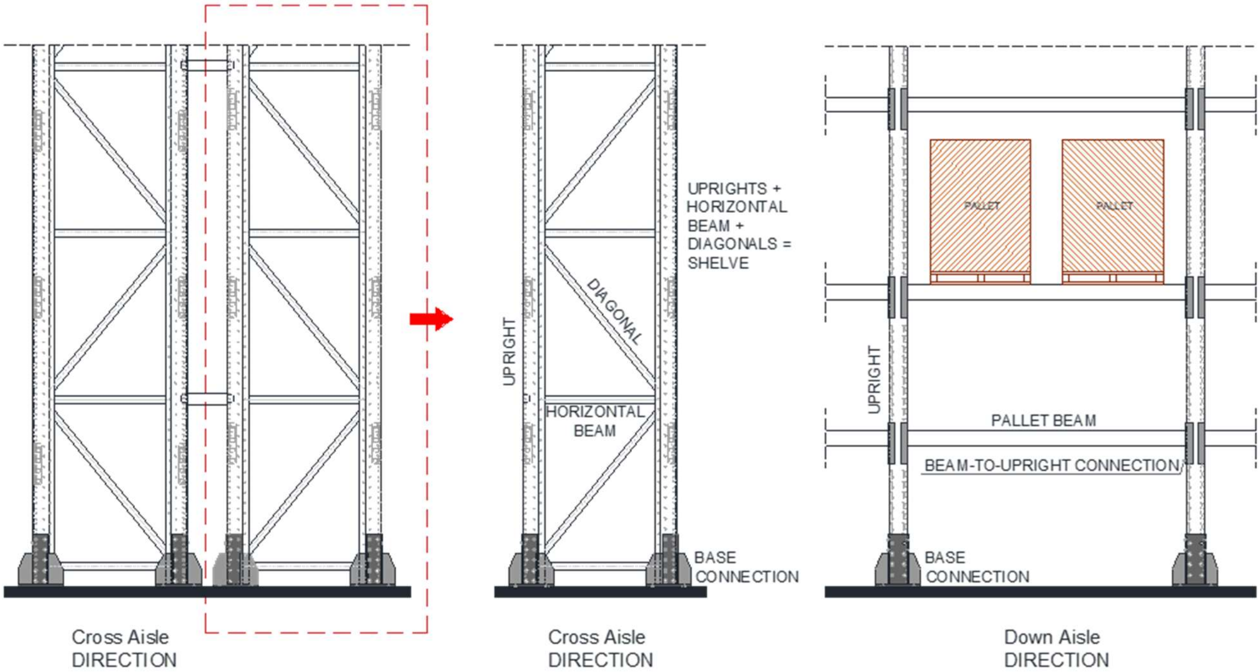
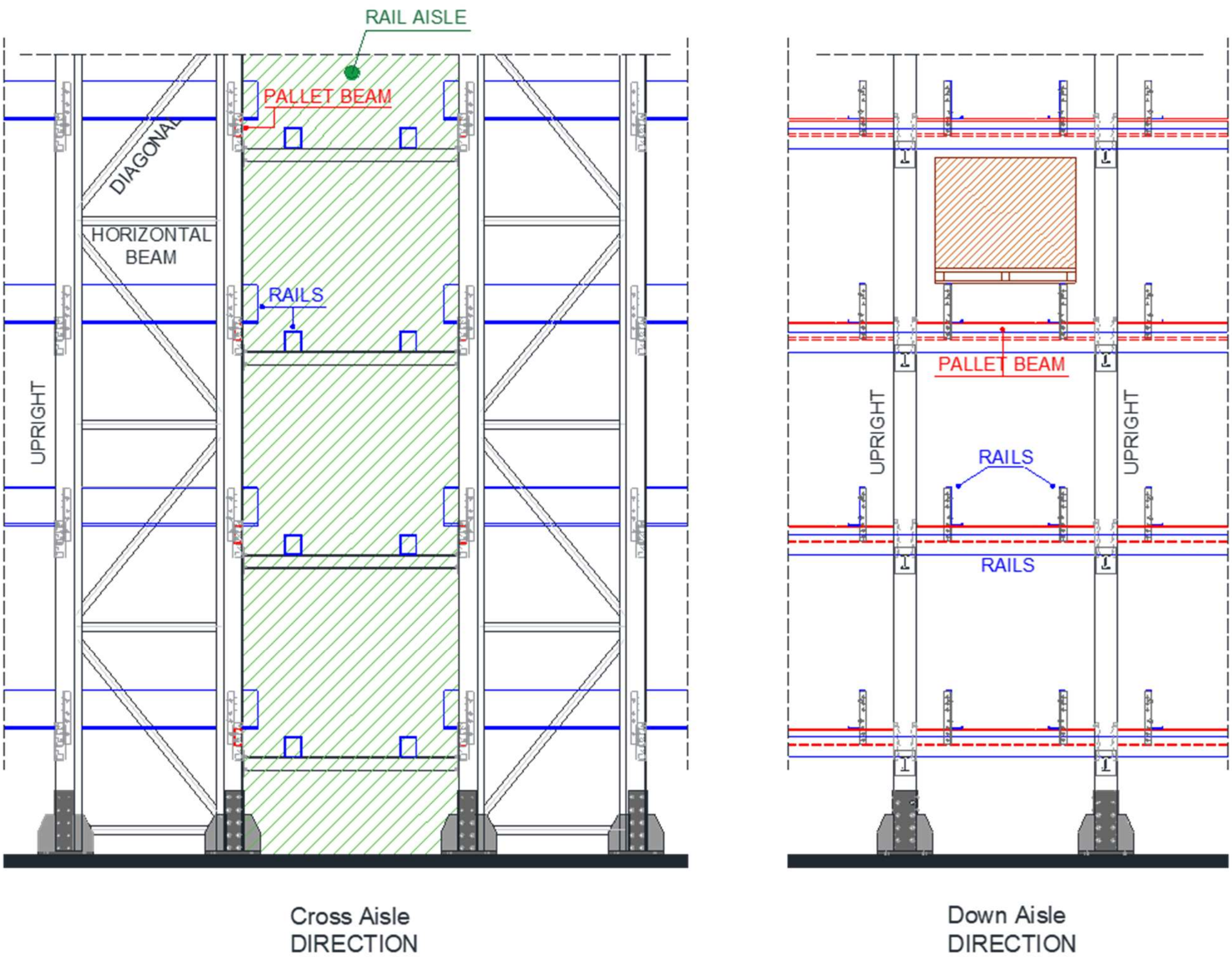


Figure 2-8: Detail B from Figure 2-6a: functional possibilities for a double-depth warehouse, with all available depths highlighted.



a) Detail A from Figure 2-6a: common shelve composition for double-depth warehouses.



b) Detail b from Figure 2-6b: common shelve composition for multi-depth warehouses.

Figure 2-9: Common shelve composition for double and multi-depth warehouses.

a) Stacker crane (respectively taken from http://ritmindustry.com/ and https://www.mecalux.it/).

b) Shuttle (https://www.mecalux.it/).

Figure 2-10: Devices for automated handling of goods inside a warehouse.

2.2. Typological and structural analysis of current structural solutions for ARSWs

ARSWs represent the last developed technology in the stocking field, joining the optimization of both spaces and management of goods flows. The spaces optimization is allowed by the extension of the racks from the base to the top of the structure, allowing the full exploitation of the building's height. The automated handling of pallets enables the optimization of the goods flow management, and logistic rules are set on the warehouse owner's necessities. Being the natural and rapid development of traditional steel racks, ARSWs have inherited several structural characteristics that are typical of racks, although the structural functioning and typologies are different.

The first useful classification of ARSWs can be given in based on the functional aspect. From this point of view, there are three main typologies of warehouse: the single, double and multi-depth (see §2 and §2.1). Synthetically, these three solutions are different because the shelves are arranged differently inside the warehouse (different number of depths). The device for the handling of goods is also not the same.

Consequently, also the structural typology may be different, as well as the structural behaviour. The choice among the three typologies is mainly based on achieving the maximum optimization of goods flow that is expected to occur, keeping under control all the costs related to the building, maintenance and technological devices. Although a universal strategy has not been defined, yet (P. Baker and Canessa 2009), as depicted by Gu, Goetschalckx, and McGinnis (2010), the design of a warehouse depends on five significant decisions: (i) determining the overall warehouse structure; (ii) sizing and dimensioning the warehouse and its departments; (iii) determining the detailed layout within each department; (iv) selecting warehouse equipment; (v) selecting operational strategies. All of these choices have to be made only after a deep analysis of goods flows that concerns the warehouse owner's activity. The overall structure - (i) - is a conceptual design of the warehouse and determines the pallet flow pattern within the structure, the specification of the necessary functional departments, and the flow relationships between departments. The sizing and dimensioning decisions - (ii) - determine the size and dimension of the warehouse and the space allocation among various warehouse departments. Department layout - (iii) - is the detailed configuration within a warehouse department, for example, aisle configuration in the retrieval area, choice of the optimal number of depths, and configuration of an automated storage/retrieval system. The equipment selection decisions - (iv) - determine an appropriate automation level for the warehouse, and identify equipment types for storage, transportation, order picking, and sorting. The selection of the operation strategy determines how the warehouse will be operated, for example, concerning storage and order picking, and basically defines all the logistic rules applied to the distribution of goods. Finally, operation strategies - (v) - refer to those decisions about operations that have global effects on other design decisions, and therefore need to be considered in the design phase. Examples of such operation strategies include the choice between randomized storage or dedicated storage, which means that, if in the warehouse there are pallets with different items inside, the storage area respectively cannot or can be organized in several specific sectors, each one for a particular kind of item. Within decision (iii), the choice between the more adequate functional solution among single, double and multi-depth is made. Only after these decisions, the structural design is completed.

From the functional point of view, as previously said, there are three main typologies of warehouse: the single, double and multi-depth (see §2 and §2.1). The main difference between these solutions is that, regarding single and double-depth, the shelves are accessible from the workers, since there are some clear areas between consecutive shelves (the aisles) that allow the movement of the stacker-cranes for the handling of pallets. On the contrary, within multi-depth warehouses, the shelves are not accessible as they are all connected. As previously mentioned, within ARSWs, two main directions can be individuated (Figure 2-11):

- CA direction, where the resisting frame is constituted by racks developing from the base to the top. The possible connections between consecutive racks depend on the number of the depths adopted (single-double-multi). In general, racks are connected at the base - they are fixed to the foundation - and at the top, through the roof truss. Racks are generally characterized by a truss structure, with the diagonals bolted directly to the uprights, and the uprights are hinged to the ground through a steel base connection. Along CA direction, racks constitute the resisting frame both to vertical and horizontal loads. In this direction, diagonals can be organized in different structural schemes depending mainly on the effects of horizontal actions on the structure and the structural behaviour adopted (dissipative or not).

- DA direction, along which the CA frame is repeated several times (as required by the design of the warehouse). Each frame is connected to the following one through pallet beams and horizontal bracing, which has to be placed at the same level as the pallet beams to not interfere with pallets' space. In this direction, the resisting frames are constituted both by the uprights and the pallet beams, which are connected through semi-rigid connections to the columns, and the longitudinal vertical bracing. The longitudinal bracing can be placed in the same plane of the frames or an eccentric position. Along DA direction, the resisting structure to vertical loads is constituted by the uprights and the beams, while the longitudinal bracing takes the horizontal actions.

Concerning the structural behaviour, these two directions can be considered almost independent regarding the response to the horizontal action, being the two resisting systems independent as well.

In the following, details about structural choices are given for each direction.

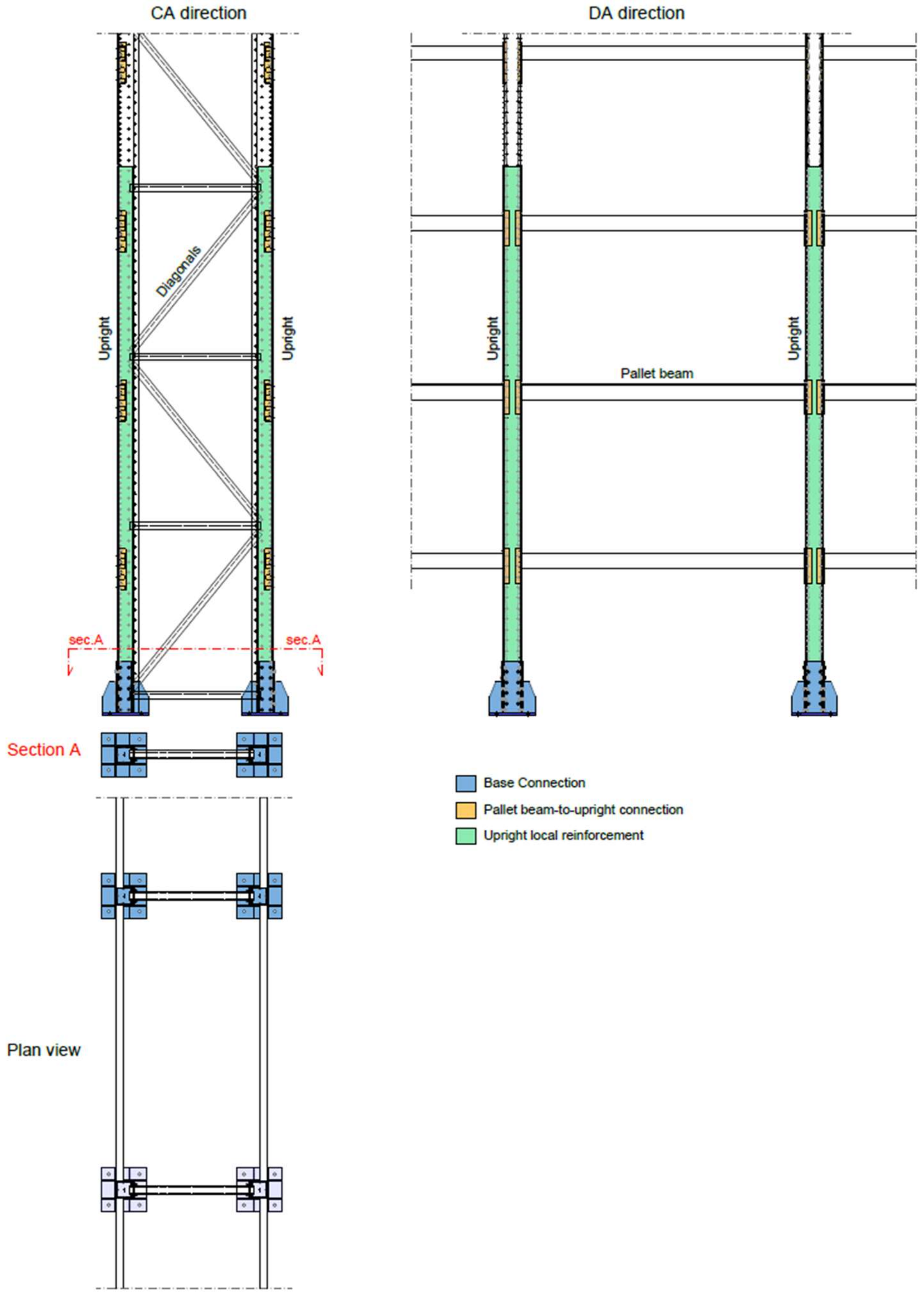


Figure 2-11: Representative drawing of a shelve highlighting the main elements.

2.2.1 CA direction: structural characteristics and details.

Along with CA directions, the main frame is constituted by steel racks. Each of them comprises two uprights connected through diagonals, and it constitutes the vertical load-resisting system. The diagonal layout depends on the structural scheme adopted for the horizontal resisting system (Figure 2-12).

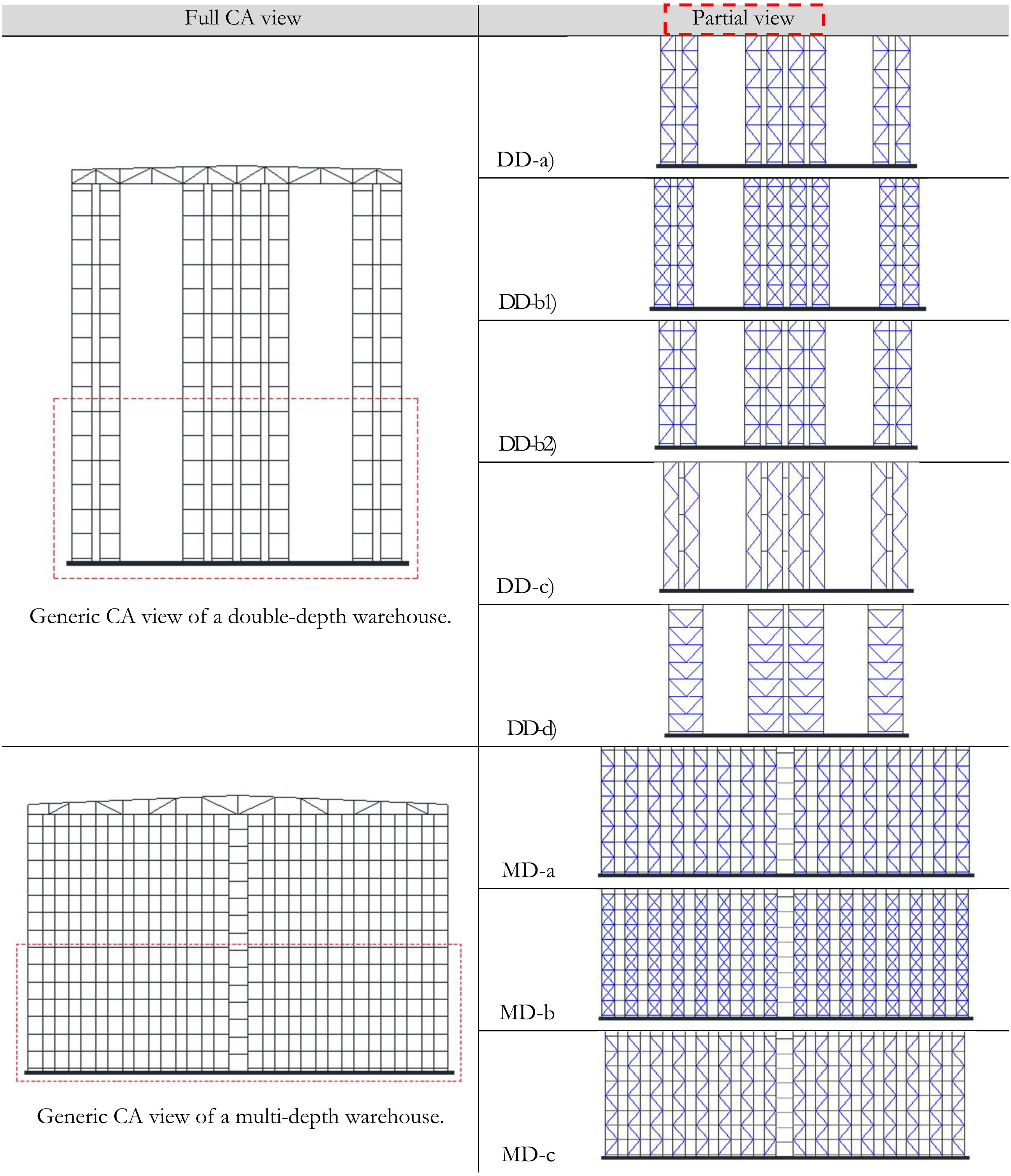


Figure 2-12: Possible diagonal layout for ARSWs to be adopted for CA direction seismic-resistant resisting frames. For the Double-Depth (DD) the possible solutions are: (DD-a) truss scheme, (DD-b1) not split X-shaped braces, (DD-b2) split X-shaped braces, (DD-c) K-shaped braces, (DD-v) V-shaped braces. For the Multi-Depth (MD) the possible solutions are: (MD-a) truss scheme, (MD-b) not split X-shaped brace, (MD-c) K-shaped brace.

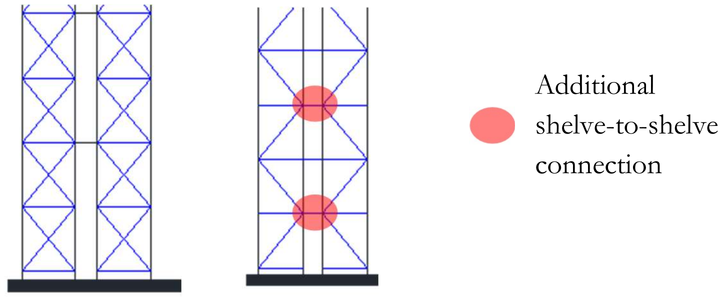


Figure 2-13: From DD-b1 and DD-b2 within Figure 2-12: NOT SPLIT X-shaped brace on the left and SPLIT X-shaped brace on the right.

As illustrated in Figure 2-12, for single and double-depth warehouses, the possible layout for diagonals to be adopted is the truss one (DD-a), the X-shaped one (DD-b1 and b2), the K-shaped one (DD-c) and the V-shaped one (DD-d). Regarding X-shaped braces, the not split solution consists of putting in each bracing span a couple of diagonals. In the split one, the two diagonals are separated and put in two consecutive spans (Figure 2-13). This last solution implies connecting the two consecutive shelves at each upright node. Dealing with Multi-Depth warehouses, the main diagonal layouts are truss scheme (MD-a), not split X-shaped (MD-b) and K-shaped (MD-c). The truss scheme and the K-shaped arrangement are typically adopted for traditional steel racks, and they directly inherited it. The other schemes have been introduced to improve the seismic-resistance structural performances of ARSWs.

Regardless of the diagonals' layout, all the profiles constituting CA frames are usually made of cold-formed sections. This is another important feature that has been inherited directly from traditional racks, as well as the shape of cross-sections and the configuration of the structural details:

- for uprights, the most adopted cross-section is the open U one with lips. The open section allows an easy connection of diagonals, that can be inserted into uprights from the open side and bolted straight to the upright without using any additional components (as gusset plates). In fact, these additional components would affect the production phase with further processing and increase the total weight of the structure. These two aspects could finally determine an increment of the structure's cost, which is a relevant parameter that has to be taken under control by the designer. Uprights may also have diffused holes along their length to allow an adjustable connection with diagonals (along CA direction) and pallet beams (along DA direction). The interspace between consequent holes can also be minimal (also 5 cm). Anyway, although it is generally possible to move an element (diagonal or pallet beam) and link it in another position, this is not typical of ARSWs, where geometry is fixed from the very beginning of the design process and connected to the logistics rules of the warehouse. From the structural point of view, the holes pattern of uprights both affect the weight of these elements, which get lighter, and may affect their structural performances, especially in compression and bending. These aspects have to be taken into consideration during the design phase.
- for diagonals, several sections may be used, also depending on the structural scheme adopted. One of the most used shapes is the C one, which is suitable for all the structural schemes, except the not split X-shape layout, because of the difficulties connected to the realization of the connections. In the other configurations, the C-sections are inserted into the upright's open side and directly bolted on it. In alternative, a closed section (as a square of a rectangular hollowed

one) can be used, but the connection with the open-sectioned upright – without using any additional plate) is only feasible squeezing it at the end sections, and then bolting it to the column.

The upright base connection is made of a steel base plate that is usually connected to the foundation - that is mostly a concrete slab - through post-installed anchors, that guarantee a fast installation. When the horizontal actions become relevant, the post-installed anchors are replaced by tethered anchor bolts, that assure higher tensile and shear resistance. Still, their installation may slow down the construction phase. The connection of the upright to its base plate is usually made by bolting (or welding) the upright to an external and bigger C section that is welded to the base plate.

Along CA direction, the structural behaviour of the connections described above (diagonal-to-upright and base ones) can be assumed as "hinged" (Figure 2-14a).

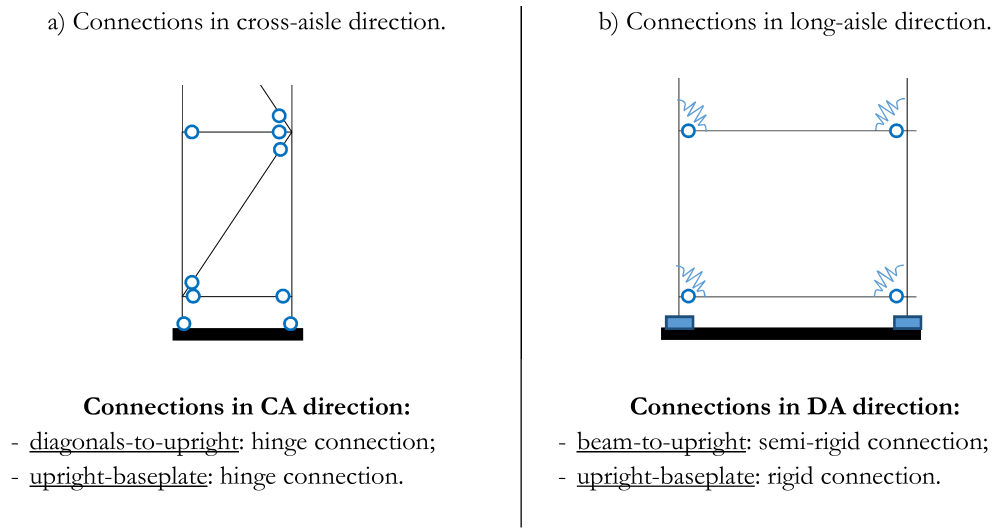


Figure 2-14: Structural schemes for CA and DA direction.

2.2.1 DA direction: structural characteristics and details.

Along DA direction, the two resisting frames are constituted by uprights connected to pallet beams (from here on "pallet frames") and by the longitudinal bracing system. The first resisting frame is mainly for vertical loads (being very flexible, especially compared to the frame with bracings). In contrast, the frames with bracings are the primary system for horizontal actions. Dealing with the braces' layout, they are in most cases arranged in the X-shaped one. Figure 2-16 shows the possible solutions for the distribution of bracings along DA direction. Dealing with double-depth warehouses, the bracing system can be inserted in the same plane of the uprights (*in-plane bracing system*) or in an eccentric position with respect to them (*eccentric bracing system*). The first solution (DD-a) implies that the braces need to be placed in specific spots, named "bracing towers", that won't be available to place the pallets anymore (this has to be considered during the design phase). In case of choosing an eccentric bracing system, both bracing towers (DD-b) and diffused braces (DD-c) can be adopted, since there is no interference with the activity of movement of the pallets. Obviously, the bracing frames have to be put in the outer or external zones with respect to the shelves, not to interfere with the shelves served by the stacker cranes. Concerning multi-depth warehouses, the only available option is using bracing towers and placing them at the extremities of the shuttles' servable areas (MD-a), not to interfere with the shuttles' activity. If this is not sufficient, bracing towers can also be placed among the racks, but that space won't be usable anymore for pallets.

Also, for this direction, all the elements are made by cold-formed sections, as usual for traditional racks, except the possibility of adopting hot-rolled profiles for braces, if high horizontal loads are involved. In particular, in the following, the most adopted structural choices are given for the elements and connections along DA direction:

- concerning pallet beams, both open sections, as Z ones, or close sections, as rectangular hollowed ones, can be used. As inherited from traditional racks, pallet beams are connected to the uprights through particular semi-rigid connections constituted by a coupling device. It is called "hooked connection" because of the particular configuration of the end-plate, which is welded to the end sections of the profile, while its outer part has several hooks (or tabs) that can be directly inserted within the holes of the uprights. This technique allows a high-speed connection during the assembly procedures (Figure 2-15). Besides, this connection allows the beam to be moved in a different position in a reduced time without removing other elements to carry out this operation (except for other elements that can be directly connected to the pallet beams, possibly horizontal bracings). As already said, this feature is more typical of traditional racks than ARSWs because the position of load levels is one of the input design parameters of the structure and strictly connected to the logistic strategies.
- dealing with diagonal braces, any cross-section can be selected to optimise the structural behaviour of the warehouse along DA direction. The connection of these elements to the uprights is usually bolted, using additional elements like gusset plates. Obviously, if the bracing system is not in line with the uprights, it is necessary to connect them. Usually, this connection is made at each node of the bracing system.

Along DA direction, the pallet frames' structural scheme corresponds to uprights hinged or fixed to the ground and pallet beams connected to uprights through semi-rigid connection. Concerning the bracing frames, uprights can be considered hinged to the ground, and the braces hinged to the uprights (Figure 2-14b).

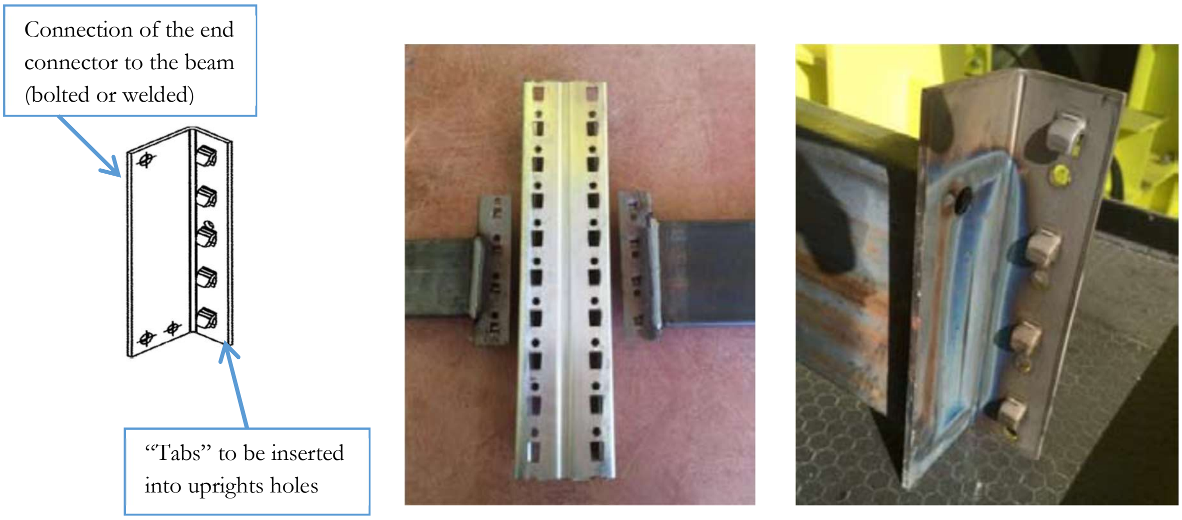


Figure 2-15: An example of a "hooked connector" (images from El Kadi et al (2017)).

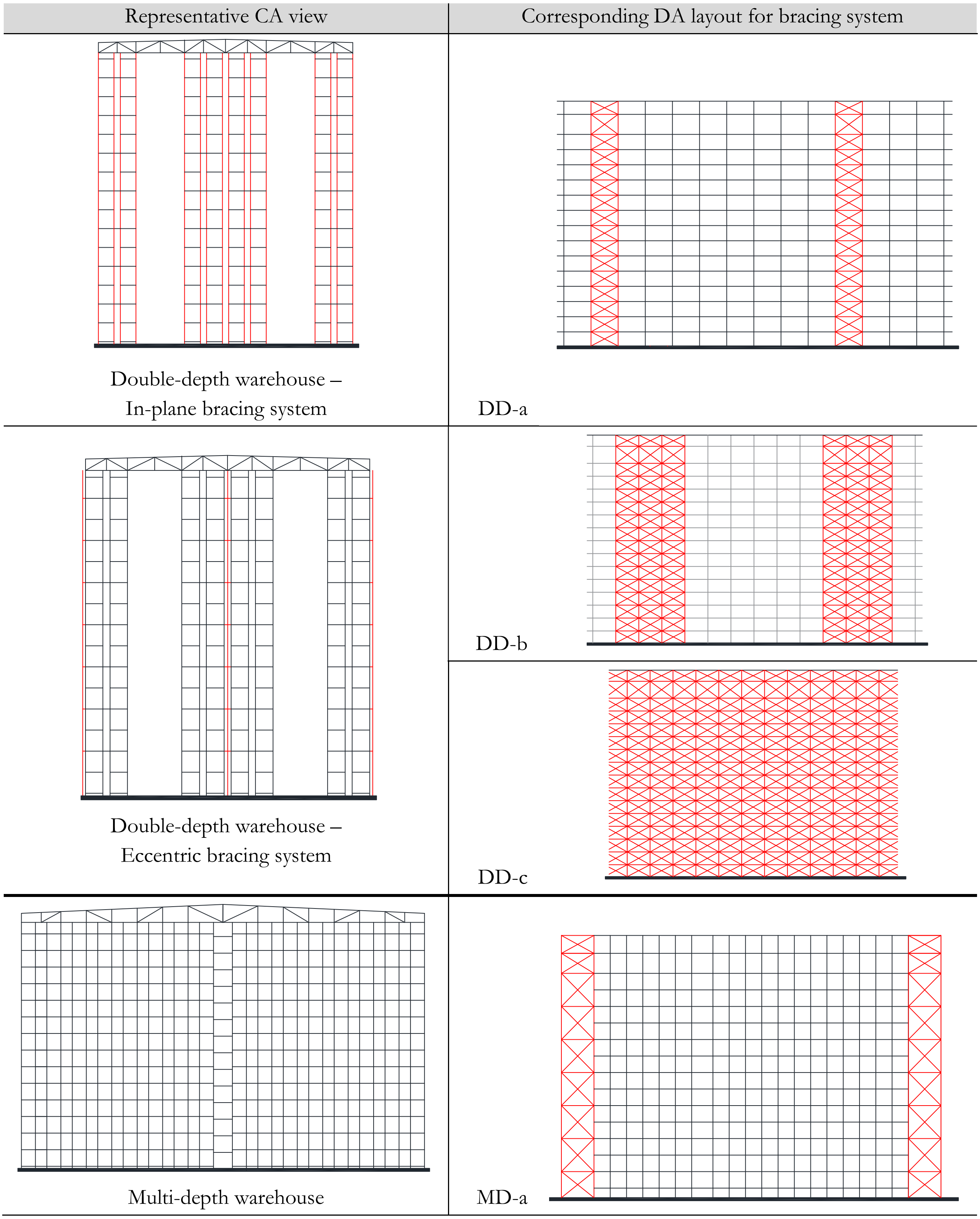


Figure 2-16: Possible bracing configuration for DA direction: for the double depth warehouse with in-plane bracing frames, the bracing tower solution is used (DD-a); for double-depth warehouse with eccentric bracing frames, both bracing towers (DD-b) or diffused braces (DD-c) can be used. For multi-depth, bracing towers (MD-a) are the only possible choice.

3. State of the Art

Automated Rack Supported Warehouses constitute one of the latest updates of steel racks. They developed very fast, following the needs of the market about, as an example, bigger places to stock the goods and automated handling of them. They inherited many structural characteristics from traditional racks since, being new fastly-developed solutions, their design has been not supported by a specific regulatory framework, that is indeed available only for traditional steel racks. In the following, the state of the art about ARSWs is organized in two different parts: the first one is the research field, where information is given about the studies around ARSWs local and global structural behaviour and related studies, that can be extended and applied also to ARSWs; the second one is about guidelines and regulations that are at now used to design ARSWs.

3.1. Research

The design of ARSWs is a very complicated process and it is firstly based on meeting the operational need using the most economic technology, and most economic structure. So, two of the fields of interest about the study of ARSWs can be recognized as the following:

- Logistic design.
- Structural design.

Logistics plays a fundamental role and is the first aspect that is analysed when designing ARSWs. In fact, the logistic choices are those that drive the whole organization and layout of the warehouse, and so also the structural choices (starting from the choice of the structural typology). In the framework of this thesis, logistics is not taken into consideration, because this research is focused on proposing structural solutions for one of the two structural typologies of ARSWs (double depth), and so, logistic strategies are supposed to be already determined. In any case, before focusing on the research about the structural components and behaviour of ARSWs, to catch a global view of the whole design process, in the following is firstly described the whole design flow of ARSWs, highlighting the part where structural choices are made. Then, a deep analysis of structural filed is given.

Although an universal design method has not been determined, yet, various strategies for the logistic design of a storage warehouse are proposed in literature, that in any case share the same leading path. This common thread corresponds to the one illustrated within §2.2, as introduced by Gu et al. (2010). As an example, Backer and Canessa (2009), based on previous numerous studies and researches, propose a framework for the logistical design of a warehouse, that is based on the following steps:

1. Definition of system requirements, meaning to gather all the necessities connected to all the fields within which the warehouse is involved and operates, including also, for example, business strategies requirements, warehouse role within the supply chain… .
2. Define and obtain data, meaning to understand the market share of the company and includes product details, order profiles, goods arrival and despatch patterns, cost data. Here is important to analyse the historic data of the company and discuss also future business interest (if any).
3. Analyse data, using the database defined at pervious step and spreadsheet models and involving an analyst computing a number of routine statistics from the order database and then the designer uses his experience to interpret these statistics. Frazelle (2002) presents a set of such routine

statistics in a section on warehouse activity profiling. These include: (i) Customer order profiling (e.g. pallet/carton/item mix profiles and lines per order distribution); (ii) Item activity profiling (e.g. item popularity distribution and demand variability distribution); (iii) Inventory profiling; (iv) Calendar-clock profiling (e.g. seasonality and daily activity distributions); (v) Activity relationship profiling (i.e. importance of certain functions being located nears other functions). This step gives the basis for the definition of the goods flow inside a warehouse and all the necessities connected (e.g. whole capacity of the warehouse, typology of goods to be stored, pallet dimensions…).

4. Establish unit loads to be used. In this part, the results of the analysis of the previous step is fundamental, as well as the designer experience.
5. Determine operating procedures and methods. This part regards the definition of the functionality of the warehouse: each necessary process is pointed-out with corresponding characteristics and needs. The interaction among the individuated processes is analysed, too. Besides, another important part of this step is to decide the number of zones into which the warehouse should be divided (e.g. zones for different product groups, temperature regimes…). Finally, the possibility to incorporate flexibility into warehouse design is evaluated by taking into consideration the resources that can be more adapt to be re-arranged and how to accommodate potential change (e.g. by extra capacity).
6. Consider equipment types and characteristics. Within this step, the devices for the handling of pallets is selected. There are a wide range of techniques used by warehouse design companies. As an example, Naish and Baker (2004) describe a step-by-step approach to equipment evaluation, comprising: (i) High-level technology assessment, based on such general factors as the scale of the operation and the flexibility required; (ii) Equipment attributes, to identify whether each equipment type is suitable for the application; (iii) Decision trees, which act as representations of what happens if a certain handling system is adopted; (iv) Full costing comparison, to calculate all the costs associated to the adoption of each equipment type and relative attributes; (v) Sensitivity analyses, to identify whether the preferred systems still perform well under alternative business scenarios; (vi) Computer simulation, to test the effectiveness of the preferred system under different conditions (e.g. crane breakdown). This is an heuristic method - which is based on a close examination of different design alternatives through intuitive rules and based on experience - but also other alternatives are possible, as depicted in Ashayeri and Gelders (1985): analytic methods, which are used to calculate an optimum solution, or simulation methods, which conduct to a "what if" analysis.
7. Calculate equipment capacities and quantities. This is generally a matter of calculation, and formal spreadsheet models tend to be used, based on warehouse flows and performance standards (e.g. from historic activity sampling). Many of the analytic and simulation methods, mentioned in the previous step, in fact address equipment capacities and quantities.
8. Define services and ancillary operations.
9. Prepare possible layouts. This is a very important phase, where given the results from steps 4, 5, 6, 7 and 8, possible organizations of the warehouse are given. Frazelle (2002) presents a five-step methodology for warehouse layout, which combines some of the above techniques: (i) Space requirements planning. This involves determining the space required for each zone (as in the block layout technique described earlier); (ii) Material flow planning: The determination of the overall flow pattern (e.g. U-shape or flow-through); (iii) Adjacency planning. This phase takes

into consideration the related activities, starting from the warehouse activity relationship chart (driven by the results from step 5); (iv) Process location: The split of areas by low-bay and high-bay usage[1]; (v) Expansion/contraction planning: Consideration of how the facility may be changed in the future. There are thus a number of techniques available to assist warehouse designers in formulating the layout of a warehouse, but these are generally designed to assist an experienced warehouse designer. As noted by Canen and Williamson (1996) there are many qualitative factors, such as safety and aesthetics, to consider as well as the purely quantitative factors, such as the flows of goods.

10. Evaluate and assess. That this step is largely concerned with validating the operational and technical feasibility of the proposed solutions, checking that they meet the requirements of step 1 (the initial requirements), and undertaking capital and operational cost evaluations. Simulation tools are used to build different scenarios and to consider a series of different situations in which flexibility of the design can be tested, and finally, to give an evaluation of the solutions.
11. Identify the preferred design. This step is basically the drawing together of all of the above elements into a coherent design, identifying, for example, the unit loads to be used, the operations and flows, the information systems, the equipment types and quantities, the internal and external layouts, the staffing requirements and the costs.

This design procedure is very complex, not only because many scientific fields are involved (mathematic, statistic, logistic, economical, management), but also given that most of the decisions have to be driven by the experience of the designers, and the use of computer-aided tools and software is only a support.

The logistical design gets the basis of the layout of the warehouse, and then each area needs to be implemented and designed properly to communicate with the other areas that are connected to it (based of the goods flow) and to be adapted to the equipment that has been chosen to optimize the functional operations of the warehouse. In particular, the storage area is the main and the biggest one inside an ARSW. Since the equipment (pallets handling devices, possible aisles) for this area is already chosen in the logistic design, as well as characteristic of the pallets (dimensions, weight), the structure of the racks needs to be arranged on these previously taken decisions, choosing the number of depths (that name the warehouse as single, double or multi-depth), the steel profiles and structural details. As this structural system is relatively new compared to the traditional steel structures, till now, very little effort has been put into the development of a design guidelines for them. Besides, ARSWs are generally treated as an evolution of ordinary racks (that is actually the truth, but cannot be considered the same), about which plenty of research can be found, being such particular structures. In the framework of this PhD thesis, the field of interest of literature about racks regards the analysis of their structural behaviour with the final aim of finding their proper design strategies. In particular, in the following, major details are given about the status of the research in these areas:

- Structural behaviour of racks components also under cyclic (seismic) action;
- Global structural behaviour of racks under cyclic (seismic) action;
- Interaction of pallets with steel structure under cyclic (seismic) action.

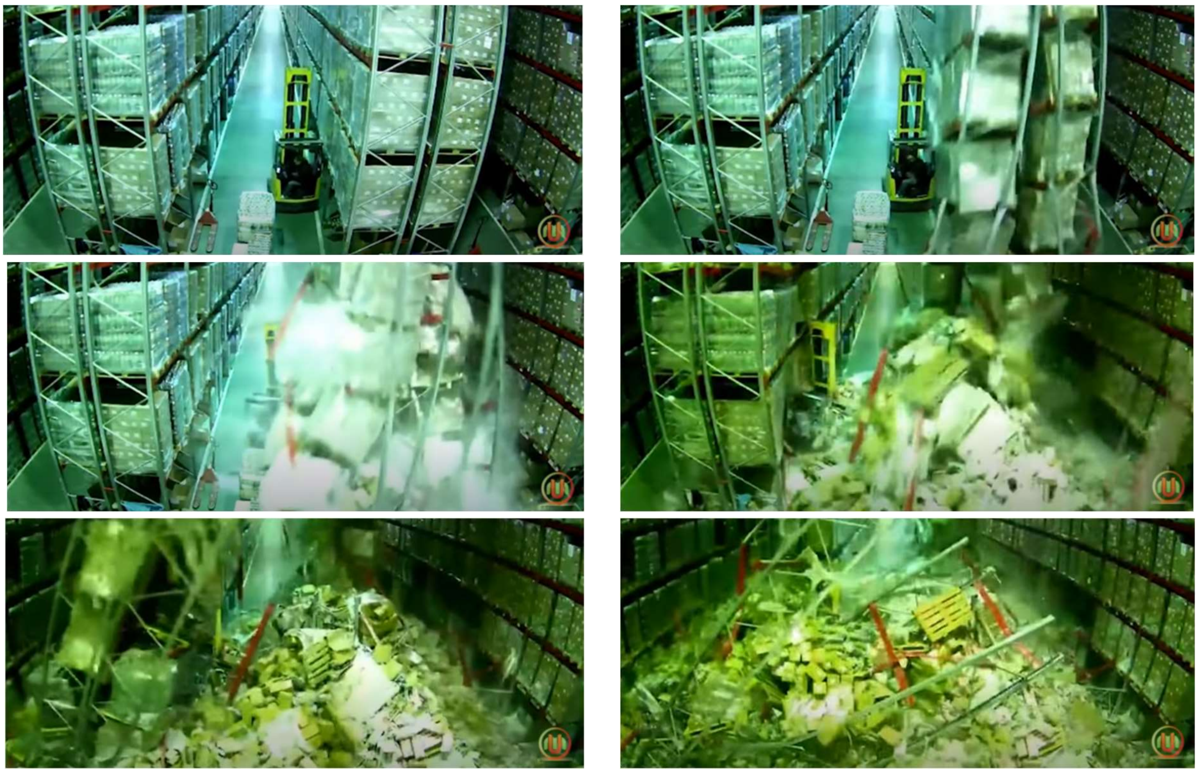
[1] Low-bay and high-bay refer to different heights of the warehouse: high-bay is for heights major than 20 m, while low-bay is for heights between 12 and 20 m.

3.1.1 Structural behaviour of racks components also under static loads and cyclic (seismic) action

Steel racks were originally designed to withstand vertical actions, and, being considered as secondary structures, with the aim of decrease their self-weight and make their assembly easier and faster, they were completely made by cold-formed sections. This second characteristics still remains, while the first one has been reconsidered, also based on collapse episodes that happened after seismic events (Figure 2-2).

In general, the collapse of a rack could lead to relevant consequences, from the death of the possible workers inside the warehouse to economic losses for the owner of the activity. The economic losses are connected:

- with a minor impact, to the damaged racks. When the damage is relevant and diffused, the structure needs to be replaced. Besides, it is very easy to have diffused damage in these structures, even if the trigger of the disaster is confined to a little part. As an example, a catastrophic event happened in a warehouse storing vodka in Russia, when a worker, that was driving a forklift, slightly impacted on a rack causing the collapse of almost all the racks inside the warehouse. The dynamic of the collapse suggests that in these structures, being very slender, characterized by low robustness, and considering that the weight of the pallets far higher than the one of the structures itself, it is very easy to initiate a domino effect (Figure 3-1).
- with a relevant impact, to the stuff within the pallets, or placed on the shelves (Figure 2-2), that may be not usable or saleable anymore after falling on the ground.
- with a relevant impact, to the time that the activity inside the warehouse needs to be stopped, till all the damaged structure is removed, the area is clear and the new structure is placed inside with new goods.



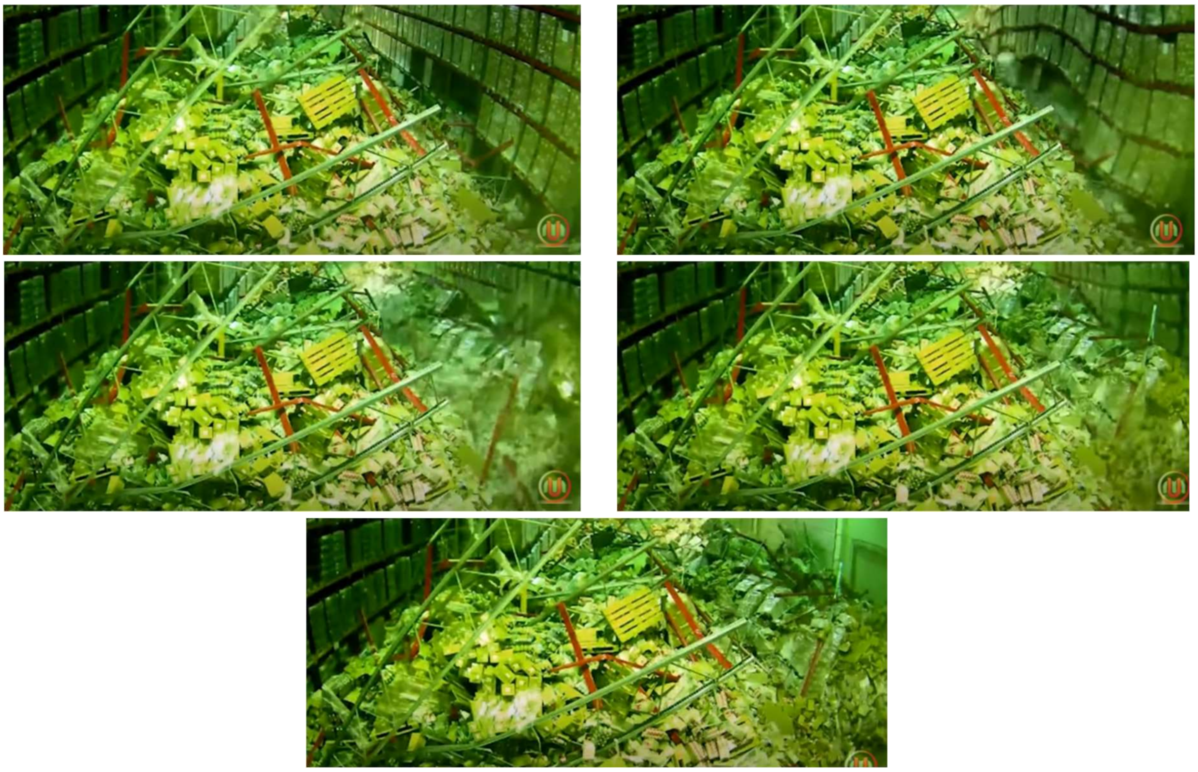


Figure 3-1: Domino effect collapse of racks in a Russian warehouse due to a forklift impact on the rack structure (images extracted by the video at the web page https://www.youtube.com/watch?v=gJc4akBOWKw).

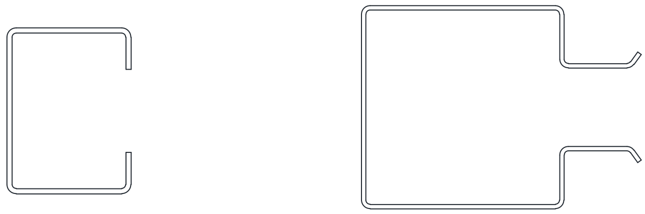
With this aim, several studies have been carried out, and some are still ongoing, to understand the structural behaviour of such particular structures also under horizontal load. In particular, these studies have been focused both at local and global behaviour, since these structures are characterized by unique structural non-standard solutions that are uncommon for steel structures and basically made *ad-hoc* for steel racks. Within steel racks, the main structural elements or connections that have been studied are:

- Uprights;
- Pallet beam-to-upright connection (DA direction);
- Diagonal-to-upright connection (CA direction);
- Upright base connections.

UPRIGHTS

Uprights are characterized by particular features: (i) they are made of cold-formed steel, the thickness of their cross-section is reduced, (ii) their shape is mostly an open, mono-symmetric lipped one; (iii) they may have holes along their height. These characteristics make the behaviour of this element, especially in compression and bending, uncommon and different from hot-rolled sections.
Indeed, as a consequence of (i), local buckling may affect the behaviour of the element both in compression and in bending. This aspect has to be considered in the design phase; indeed, as an example, Eurocode 3 part 3 (UNI EN 1993-1-3) suggests the "effective width method", to be used in the verification phase, as a way to calculate the reduced resistant area, taking away those parts that buckles and only considering those that can reach the yield stress of the material. This allows to determine the buckling resistance in compression of class 4 sections, where local buckling is relevant.
Regarding point (ii), open, mono-symmetric, thin walled sections (Figure 3-2) are characterised by three basic modes of buckling in compression, as showed in Figure 3-3. Local buckling is a mode involving

plate flexure alone without transverse deformation of the line or lines of intersection of adjoining plates (no out-of-plane movement of lips take places); distortional buckling is a mode of buckling involving change in cross-sectional shape excluding local buckling, where flanges rotate with respect to the vertical plate, going out-of-plane; flexural-torsional is a mode in which compression members can bend and twist simultaneously without change of cross-sectional shape (Hancock 1998) (Figure 3-4). Local buckling is more relevant for short buckle half-way lengths, distortional buckling mainly happen for intermediate half-way lengths and global buckling is typical of higher ones. In any case, to determine the buckling resistance of such elements is necessary to consider the effects of local and distortional mode.
Finally, the holes along the length of the elements - point (iii) - affect the buckling resistance and the stiffness of uprights, and this has to be taken into account both in modelling and design/verification phase.



a) b)

Figure 3-2: Typical shapes for thin walled sections that may develop a distortional buckling mode: a) C-section (or lipped channel section); b) U section with lips.

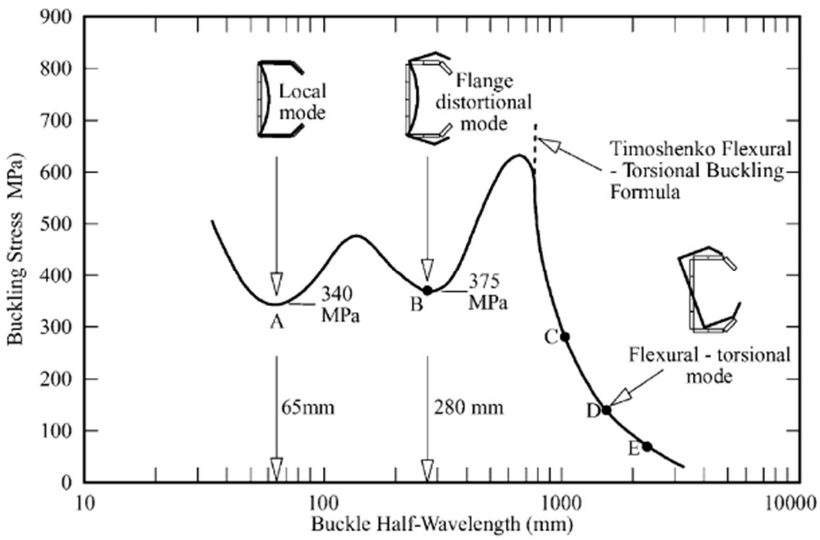
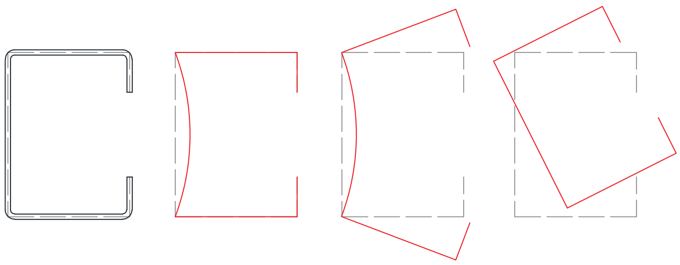


Figure 3-3: Buckling modes of lipped channel in compression (Hancock 1998).



a)

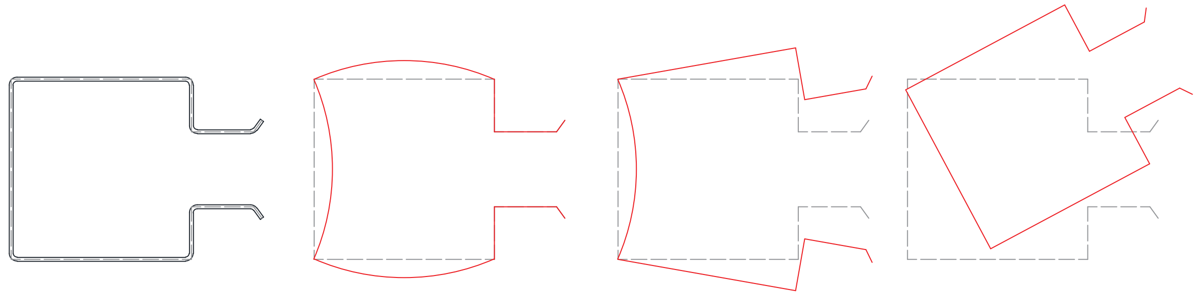
b) 

Figure 3-4: Deformed shapes (red lines) corresponding to C and U-lipped sections possible buckling modes: local, distortional and flexural-torsional respectively.

In general, the behaviour of cold formed members in compression has been studied by several authors, and in particular, Young and Rasmussen (1998a; 1998b; 2000) focused on the behaviour of cold-formed plain and lipped channel columns (C-shaped sections) - that are very common to be used for steel racks columns and diagonal element - studying in particular the effects of local buckling and its relation with global buckling. Within (1998a), an experimental investigation has been made into the behaviour of these sections when locally buckled in compression, highlighting the effects of local buckling on the behaviour of fixed-ended and pin-ended channels by comparing strengths, load-shortening and load-deflection curves, as well as longitudinal profiles of buckling deformations. Besides, the effects of the shift of the centroid when the section locally buckles were more deeply investigated within (1998b), highlighting that this shift causes a re-distribution of stresses in the section (that before local buckling is uniformly compressed), inducing a variable distribution, with higher stresses near the angles. This may affect the determination of the effective widths. Besides this "local" aspect, in singly symmetric cross-sections the redistribution of longitudinal stress caused by local buckling also produces a shift of the line of action of the internal force. When the section is compressed between pinned ends, this shift introduces an eccentricity and hence overall bending. Both of these aspects have been taken into consideration by the authors to propose a change in the calculation of the "effective widths" as proposed by the North American Specification for the Design of Cold-Formed Steel Structural Members (American Iron and Steel Institute 2007). Finally, within Young and Rasmussen (2000), a technique for determining overall flexural and flexural–torsional bifurcation loads of locally buckled cold-formed singly symmetric columns has been summarised and applied to fixed-ended plain channel sections, using an inelastic nonlinear finite strip buckling analysis. The interaction between local and overall buckling has also been studied also by Becque and Rasmussen (2009b; 2009a) through numerical simulations and experimental analyses executed on stainless steel columns. These columns are characterized by C thin-walled sections, that have been numerically and experimentally analysed both as an individual element and coupled. In this work, the importance of accurate representation of the stress–strain properties of the stainless steel material in the numerical modelling procedures is noted to be significant, while in the experimental campaign, it was noted that the effect of geometrical imperfections of the specimens are relevant in the determination of the interaction between local and global buckling. This experimental campaign is quite vast, and can be used as a benchmark to calibrate numerical models in absence of a proper experimental campaign.

Distortional buckling mode and its interaction with the other buckling modes has also been studied by several authors. Bambach, Merrick and Hancock (1998) and Kwon and Hancock (1993) have studied the distortional buckling behaviour of thin-walled channel section columns. Explicit analytical expressions were derived from Bambach, Merrick and Hancock (1998) to predict the distortional buckling stress of thin-walled channel section columns, and consideration has been given by Kwon and Hancock (1993) to the post-buckling response of such sections when undergoing local and distortional deformations. The distortional buckling of cold-formed steel columns is described in comprehensive detail in the AISI research report by Schafer (2000). The report gathers design methods which account for local and distortional interaction and which include distortional and Euler interaction. Schafer et al (2002) considered the local, distortional, and Euler buckling of thin-walled columns with regard to their treatment in design. At the time of this work the North American design specifications for cold-formed steel columns (American Iron and Steel Institute 2007) ignored local buckling interaction and did not provide an explicit check for distortional buckling. As a result of this, Schafer (2000) proposed a new method for design that incorporated, explicitly, the local, distortional and Euler buckling aspects of thin-walled columns, that is named "Direct Strength Method" (DSM) and it is now inserted within the American Specification for the Design of Cold-Formed Steel Structural Members (American Iron and Steel Institute 2007) as the analytical procedure for the design of cold-formed elements in compression. The Direct Strength Method employs gross cross-section properties, but requires an accurate calculation of member elastic buckling behaviour. Numerical methods, such as the Finite Strip Method (FSM) or generalized beam theory, are the best choice for the required stability calculations (Á. Ádány S. and Schafer 2006b; S. Ádány and Schafer 2006a). In particular, the open source software CUFSM[2], were the FSM has been implemented, has also been developed to easy execute an elastic buckling analysis of a user-defined thin-walled section, which results can be easily use to be applied to DSM and predict the ultimate strength of the element (B W Schafer and Ádány 2006). Further developments of DSM are given within (B.W. Schafer 2008), where is highlighted also its reliability, proving that it offers predictions that equal or better the traditional Effective Width Method for a large database of tested beams and columns. This method is now being implemented to cover shear, inelastic reserve, and members with holes. Especially this last field is very interesting, since most of the uprights used for steel racks and ARSWs have holes all along their height.

The interaction between local, distortional and overall buckling has also been studied by Kwon, Kim and Hancock (2009), through an execution of a series of compression tests conducted on cold-formed U channels with and without intermediate stiffeners in the flanges and web fabricated from high strength steel plates. In this work, the aims are: (i) to evaluate possible solutions to improve the compression strength of the elements without increasing their weight, so using high strength steel with reduces thicknesses; (ii) to limit the local buckling phenomena through the introduction of intermediate stiffeners. Various lengths, stiffeners positions and thicknesses were tested, noticing that for intermediate lengths a significant interaction between local and distortional buckling occurs; as well as a noticeable interaction between local and overall buckling was also observed for the long columns. In any case, a significant post-buckling strength reserve. Some little modifications of the DSM formulas failing in the mixed mode of local and distortional buckling have been studied, and the strengths predicted by the strength formulas proposed are compared with the test results for verification. Finally, Dinis et al (2012) also contributed to understand the structural response and predicting the ultimate strength of cold-formed steel lipped channel columns affected by local, distortional and global interaction and to develop an efficient DSM approach to predict their ultimate strength. An experimental campaign ad numerical studied calibrated on it and the extended have been used as a base to compare the ultimate strengths obtained by this database and the ones calculated through DSM. It is noticed that the DSM fairly accurately predict the

[2] This software is open source and available on the website https://www.ce.jhu.edu/cufsm/downloads.

results obtained by the numerical simulations, while over-estimate the ultimate strengths obtained from the experimental campaign. The disparity between the DSM strength curves required to predict adequately the experimental and numerical column ultimate strengths is mainly due to the differences between the measured initial geometrical imperfections (shapes and amplitudes) and those adopted in the numerical parametric study.

Since most of the uprights have holes along their height, the behaviour in compression of this configuration has been studied by several authors. Pu et al (1999) studied the local buckling behaviour of symmetrically perforated lipped channels both experimentally and analytically. In particular, a total of 63 stub column tests[3] have been carried out to investigate the effects of the size, position, and aspect ratio of rectangular holes on the ultimate compressive strength of perforated columns. The main results of this study are that: (i) the size and positions of perforations are important factors determining the strength reduction, and if the holes are in the effective area, the strength reduction can be severe, whereas if the holes are located in an ineffective area, the strength is hardly reduced; (ii) a formulation has been proposed to predict the failure load of perforated columns, and the unstiffened strip approach is again shown to provide a good prediction for a single central perforation. Moen and Schafer (2008; 2009a; 2009b; 2011) focused their research in this area with the aim of providing an easy method to evaluate the ultimate strength of continuously holed, cold-formed uprights and beams, relatively in compression and bending. In particular, the final aim was to re-adapt the DSM also to those elements. Within (2008), they evaluated the relationship between elastic buckling and the tested response of cold-formed, short and intermediate-length steel columns with holes. It was found that slotted web holes may modify the local and distortional elastic buckling half-wavelengths, and may also change the critical elastic buckling loads. Experimentally, slotted web holes are shown to have a minimal influence on the tested ultimate strength in the specimens considered, although post-peak ductility is decreased in some cases. Within (2009a), simplified methods for approximating the global, distortional, and local critical elastic buckling loads of cold-formed steel columns and beams with holes are developed and summarized, as an alternative to shell finite element eigen-analysis. These methods provide engineering approximations appropriate for design, but are intended to be general enough to accommodate the range of hole shapes, locations, and spacings common in industry. Within (2009b), the influence of number an position of multiple holes in plates has been studied, and closed-form expressions for approximating their influence on the critical elastic buckling stress in bending or compression are developed, validated and summarized. The finite element parametric studies demonstrated that holes may create unique buckling modes, and can either decrease or increase a plate's critical elastic buckling stress depending on the hole geometry and spacing. Finally, within (2011), design expressions are derived that extend the American Iron and Steel Institute DSM to cold-formed steel columns with holes. The proposed design expressions are validated with a database of existing experiments on cold-formed steel columns with holes, and more than 200 nonlinear finite-element simulations that evaluate the strength prediction equations across a wide range of hole sizes, hole spacings, hole shapes, and column dimensions. Casafont et al (2012) investigated on the use of the Finite Strip Method to calculate elastic buckling loads of perforated cold formed storage rack columns, as an alternative of using the Finite Element Method (FEM), where the holes can be directly modelled, but it is more time-consuming. The problem with FSM is that holes cannot be modelled, but, within this study,

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[3] Stub column test is a compression test that is made on short-length elements to investigate the effect of local buckling on the ultimate compression tests and to calculate the effective area. The procedure for the execution of this test is available also within UNI EN15512 (2009).

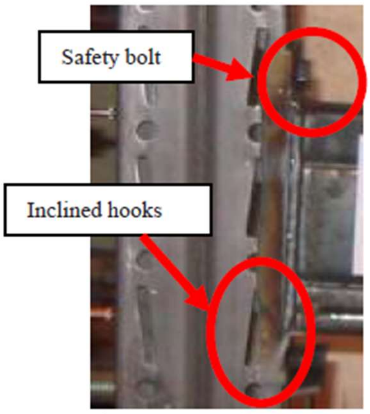
the concept of the reduced thickness of the strips where the holes are placed is applied to take into account their effect. A formulation is presented for the reduced thickness that has been calibrated with loads obtained in eigen-buckling FEM analyses. Bernuzzi e Maxenti (2015) discussed the European available design strategies to design racks with perforated members. In particular, they referred to the pallet rack code UNI EN15512 (2009), to Eurocode 3 part 3 (EN 1993-1-3) (that is specific for cold-formed elements) and to the general approach for steel structures Eurocode 3 part 1 (EN 1993-1-1). These directions were applied for the design of uprights and beam members. Substantial differences were found between the three approaches, and, if reference is made to values of the effective length of interest for routine design and typical of racks with inter-story level and/or upright frame panel of great height, the overestimation of the load carrying capacity with respect to the elastic buckling load becomes non-negligible (up to 20%, approximately). A revision of these approaches is suggested to properly consider the buckling mode interaction. Baldassino et al (2019) investigated on the behaviour in compression and bending of perforated uprights to evaluate the influence of holes on the ultimate strengths. In particular, 48 compression and 24 bending tests on perforated and unperforated profiles have been carried out. Key design geometric parameters have been individuated and three different proposals to evaluate the effective second moments of area have been developed, discussed and applied.

PALLET BEAM-TO-UPRIGHT CONNECTIONS (DA DIRECTION)

Pallet beam-to-upright connections are a fundamental component for rack systems, especially considering their contribution in strength and stiffness with respect to horizontal actions along DA directions. In fact, along this direction, traditional steel racks may not have additional vertical bracing system, and so uprights and semi-rigid connected pallet beams represent the only resisting system to horizontal forces. As a consequence, the structural behaviour (monotonic and cyclic) of both pallet beam-to-upright connection and upright base connection is relevant to assess the cyclic structural behaviour of rack systems along DA direction. In addition, these connections are mostly non-standard (§2.2.1), and, due to the great number of types of beam-end-connectors, as well as the different geometry of the connected members (i.e., beams and columns), theoretical approaches to evaluate the performance of such joints are not currently available as their structural behaviour is not easy to be predict. Experimental tests are indeed necessary to be executed to assess the main parameters characterising the response. The study of semi-rigid connections started earlier, interesting bolted standard beam-to-column connections with angles (Wu and Chen 1990; Kishi and Chen 1990) and finalized to proposed an analytical model to represent their behaviour based on three parameters: the initial stiffness, the ultimate bending capacity and the shape parameter. This model is then used to represent the entire moment-rotation behaviour of these connections. In rack field, Tan et al (1996) developed an analytical model to represent the non-linear behaviour of rack connections, based on experimental test executed on specimens where the two changing parameters were the thickness of the upright and of the connector plate. With regard of these previous studies, the load was only a monotonic gravitational one, and so only sagging bending was investigated. As a further develop, Bernuzzi and Castiglioni (2001) carried out an experimental campaign to study the behaviour of beam-to-column joints, as a part of the wider project "Seismic Design of Steel Storage Pallet Racks", sponsored by the Rack Manufacturing Companies Group of the Italian Association of Steel Constructors (ACAI), and in particular by the Companies which form the "Seismic Working Group". In total, 11 tests have been executed on two different types of commercial products (hooked connection type). Both monotonic and cyclic tests have been carried out, and, in particular, the cyclic ones highlighted that the geometry of the connection plays a big role in the structural behaviour of the whole system, since plastic deformation of the hooks cause large sway of the columns, with a consequent increasing of second order effects. This confirms that the study of the behaviour of these connections is fundamental to assess the structural behaviour of steel racks along DA direction, especially when vertical bracing is absent. Prabha et al (2010) conducted 18 experiments on a commercially available pallet rack connection (a hooked one) by varying the most influencing parameters (named "size parameters") such as thickness of the column, depth of the connector and the depth of the beam, aiming to quantify their flexibility and develop a three parameter based model that may represent moment versus relative rotation. Further great contribution in the study of pallet beam-to-upright connections was given by the research project "SEISRACKS: Storage Racks in Seismic Areas" (European Commission. Directorate-General for Research 2009). This research project (2004-2007) focuses on the preliminary study of different structural solutions for traditional racks, analyzing their structural behaviors resulting from the application of current design. Within this project, besides numerous academic realities, different Industrial Partners nowadays producing steel racks were involved to appreciate the current design strategies and structural choices. In general, the structural behavior of racks was analyzed both from the local and global point of view, from the study of the connections (base ones, diagonal to upright ones, pallet beams to upright ones) to the interaction of the pallets with the structure and the global behavior of the system. The main outcomes of this research project were included in FEM 10.2.08 (*FEM 10.2.08: Recommendations for the Design of Static Steel Pallet Racking in Seismic Conditions* 2011), which led to a quality standard for the design of racks in seismic areas. On the base of FEM 10.2.08 (2011), the UNI EN15512 (2009) was produced as a benchmark for the static design of steel racks, pointing out structural typologies, actions to be taken into considerations, possible numerical analyses, design and verification of structural elements. SEISRACKS found its continuaton in "SEISRACKS2: Seismic Behavior of Steel Storage Pallet Racking Systems" (C. A. Castiglioni, Kanyilmaz, Bernuzzi, et al. 2014) (2011-2014), aiming to solve the remaining lack of knowledge leading to unconservative design rules and to focus more in the characterizaton of seismic behaviour of steel racks. Through extensive full-scale testing and numerical simulations, a significant difference was found in terms of ductility between the experimental and design response, due to the adoption of unsuitable values of behavior factors. The results pointed out the need to revise the guidelines for the seismic design of pallet racks as regards the exploitation of their ductile resources. The project results were the baselines for the recent UNI EN16681 (2016), that are the current European Standards for the seismic design of steel racks, assuming also the use of dissipative solutions, based on the adopted structural scheme. Within the framework of these projects, the behaviour of pallet beam-to-uprights connections was studied, and, in particular:

- Within SEISRACKS (2009), commercial products provided by the industrial partners participating to the project were experimentally investigated to assess the moment-rotation curves and the collapse modes under monotonic and cyclic load. All the end-connector were hooked. A strong asymmetry was noted both in plane and in out-of-plane direction because of the structure of the connection: as an example, for the in plane behaviour, one type of connection had a security bolt only in the upper side of the connection, over the flange of the beam, and the beam is welded to the plate of the connection along three sides only, leaving the lower flange un-welded; for the out-of-plane behaviour, the hooks were placed in one side only of the end-plate (Figure 3-5). This strengthen the need of testing both sagging and hogging moment to characterize the structural behaviour of these components. For each connection type, the varying parameter was the geometry of the beam, while the upright was the same. Besides the characterization of the behaviour of these connection type, an important outcome is the innovative procedure that was used for the execution of cyclic tests on beam end connectors of rack structures, that was then introduced within UNI EN15512 (2009) as the standard one.

- Within SEISRACKS2 (C. A. Castiglioni, Kanyilmaz, Bernuzzi, et al. 2014), starting from case studies designed by industrial partners, pallet beam-to-upright connectors were experimentally tested both with monotonic and cyclic load, with the final aim to calibrate a numerical model of the whole rack taking into account also each component behaviour and calculate the q-factors of the structures.

a) 

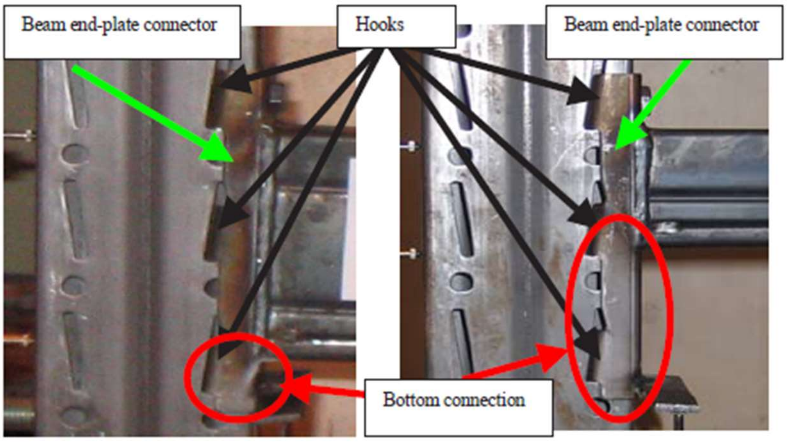
b) 

Figure 3-5: Pallet beam-to-upright connector considered in SEISRACKS experimental campaign: figure a) represents the geometry of the connection, while figure b) highlights the element composing the end-plate (These photos are taken from SEISRACKS final report (2009), and correspond to Figure 3a and Figure 4 from Chapter 2).

Shah et al (2016) used some previous experimental investigations as the calibration bases for their numerical study. A three dimensioned non-linear finite element model was developed and calibrated against the experimental results, taking into account material non-linearities, geometrical properties and large displacements. This validated model was further extended to perform parametric analysis to identify the effects of various parameters which may affect the overall performance of the connection. Yin et al (2016) explored bearing capacity and energy dissipation of hooked connections in various configurations: no bolts added, one or more bolts added, addition of both bolts and welding all around the beam. Experimental monotonic and cyclic tests were conducted to assess their behaviour and possible pros and cons related to each connection addition. Dai et al (2018) also conducted an experimental investigation into the flexural behaviour of beam-to-upright hooked connections with bolts addition of steel storage pallet racks with the aim of examine deformation patterns, failure modes, rotational stiffness, moment resistance and corresponding connection rotations. A total of 21 specimens were tested under monotonic loading, including three different size pallet beams, three different upright thicknesses, and beam-end-connectors with two or three tabs, in order to catch the influence of these parameters on the structural behaviour. Finally, a preliminary theoretical model based on the component method is proposed to predict the initial rotational stiffness of such particular connections. Finally, Gusella et al (2019) worked on a simplified model to represent the cyclic behaviour of pallet beam-to-upright connections. The model is calibrated on some data from literature and from laboratory experimental tests, and can be easily implemented in commercial software commonly used for non-linear seismic vulnerability analyses.

DIAGONAL-TO-UPRIGHT CONNECTIONS (CA DIRECTION)

Along CA direction, upright and diagonals constitute a truss that constitute the vertical and horizontal resisting system, and behaves very similarly as a built-up column. Diagonals are usually directly connected to uprights with a bolt, without using additional plates. Since the stability of steel storage racks in the CA direction is typically ensured by these upright frames. Sensitive to second-order effects, accurately determining the shear stiffness of these frames is essential for seismic design and for ensuring the stability of the rack, especially for high-bay racks and racks supporting the building enclosure, as ARSWs, where the outer rack frames must withstand cross-aisle horizontal actions due to wind loading.

Rao et al (2004) conducted experimental and numerical studies to evaluate the shear stiffness of the frames as a fundamental parameter to determine its buckling load. Different thickness combinations for upright and diagonal elements were involved in the experimental campaign. The effects of various parameters such as connection eccentricities, bolt bending and rotational release about bolt axis was identified, but further study needed to be carried out to find the significance of joint flexibility and to propose a better procedure for the evaluation of the shear stiffness of upright frames. In this framework, Godley and Beale (2008) conducted a series of experimental tests involving full cyclic loading through the origin, not only to determine the stiffness of the frame but also the looseness inherent in the connection. Design recommendations were indeed provided about including the effects of looseness in cross-aisle sway analyses and to use cyclic testing to determine the shear stiffness of racking frames. A parametric study was conducted on a commercial frame which showed that if the design recommendations are adhered to that, the influence of shear stiffness in CA direction does not normally have a significant effect on frame capacity. The possible methods to assess the shear stiffness of upright frames were reviewed by Gilbert et al (2012), by analysing the main factors that influence the shear deformation through the execution of a vast experimental campaign. In particular, the following approaches were taken into account: (i) the Rack Manufacturers Institute (RMI) specification (2008) conservatively uses Timoshenko and Gere's theory (1961); (ii) the European Specification UNI EN 15512 (2009) recommends testing, however it is not clear whether the shear stiffness obtained using the recommended test procedure is correct; (iii) the newly revised Australian Standard AS 4084 (1993), that adopted the European approach but also introduced an alternative test method for determining the combined bending and shear stiffness of upright frames in the transverse direction. The upright frames have been tested using the two test methods, and the experimental results are presented, discussed and compared with finite element analysis results. Recommendations on how to use the test outcomes in design are also provided, showing that the two test methods are not equivalent and yield different results for the transverse shear stiffness of upright frames. Far et al (2017) investigated two sets of experimental laboratory tests on braced cross-aisle frames in order to determine their shear stiffness values. The experimental results were used to compare the accuracy of different methods of analysis for establishing the shear stiffness of those frames. These methods included a detailed 3D Finite Element model, a 2D frame analysis with beam elements and a simple hand calculation. Significant variation of results compared with experimental values was found. A simplified modelling approach for 2D elastic analysis of braced frames was suggested, aiming to use it for practical applications to account for the flexibility in bolted connections and give an instrument that leads to better approximation of the shear stiffness. Talebian et al (2019), starting from the same discrepancies found also by Gilbert et al (2012), developed a model to capture the transverse shear stiffness of upright frames using shell elements and advanced

FEA. Its accuracy is verified against published experimental test results performed on three commercially used upright frame configurations, and factors contributing to the transverse shear deformation of the frames are quantified and discussed.

<u>UPRIGHT BASE CONNECTIONS</u>

Upright base connections play a fundamental role in the cyclic behaviour of steel racks along DA direction. In fact, in lack of a vertical bracing system, the seismic resistant elements are uprights and pallet beams, constituting a moment resisting frame. Within SEISRACKS research project (2009), the behaviour of some base connections provided by the industrial partners participating to the project was characterized through both monotonic and cyclic tests for different levels of axial force acting on the upright. The final aim of this experimental campaign was to assess the moment-rotation curves of these components as well as the collapse modes. These tests were executed both in CA and DA direction. Besides, in order to simulate different floor conditions, the specimens were connected either to a steel member or to a concrete block, where the steel member wants to simulate the steel deck. This kind of influence cannot be unconsidered *a priori* and, if not, in some cases could lead to unsafe design, over-estimating the load-carrying capacity of the structure and so not correctly defining the effective in-service capacity (Bernuzzi, Persico, and Simoncelli 2016). In the framework of SEISRACKS studies, the results proved that connecting the column base to a concrete slab or to a steel deck does not change the mechanical properties or the failure mode of the connection. Within SEISRACKS2 (2014), further investigations were carried out to determine the plastic behaviour of the column bases of rack systems under seismic loading, and to assess the moment rotation characteristics of the connection between the upright and the column base. The components were extracted from the case studies that were designed by the industrial partners participating to this project. Also in this case, both monotonic and cyclic tests were executed in DA and CA directions for various values of axial force. Significant correspondence between axial load and moment resisting capacity of the connection (or buckling effects on the upright) were found. Petrone et al (2016) focused on the behaviour of base connections along CA direction in seismic conditions, highlighting that, although their down-aisle response is relatively well understood (since is relevant for the DA design when the vertical bracing is absent in that direction), there is little understanding of their cross-aisle contribution. Six full scale tests on braced frames representing storage racks in the cross-aisle direction have been carried out, investigating the influence of base plate thickness and dimensions, and the upright cross-section. These tests are complemented by Finite Element (FE) simulations of the base connections, and analytical equations were proposed for characterizing the backbone curve of the hysteretic response, for use in displacement based design methods.

3.1.2 Global structural behaviour of racks under cyclic (seismic) action

The structural behaviour of steel racks has been studied during the last decades. Both CA and DA directions were investigated, although main attention has been paid to DA direction (Petrone et al. 2016), where the lateral stiffness is lower and, in absence of vertical bracing system, depending only: on upright base connections, on the continuity of uprights at their intersection with pallet beam and on the semi-rigid pallet beam-to-upright connections. As a consequence of this, it is fundamental to know to the behaviour of these components to fully understand the behaviour of the whole, and since there are various solutions available for these, experimental testing is necessary, justified and supported form the “design by testing” strategy found in all of the guidelines for the design of steel racks. In this framework, Baldassino and Bernuzzi (2000) before analysed, tested (Baldassino, Bernuzzi, and Zandonini 1999; 2000), and then carried out a numerical study on the response of pallet racks commonly used in Europe. Firstly, typical rack components from 17 commercial Italian manufactures have been experimentally investigated, identifying the performance of short columns (stub column tests), different pallet beam-to-column joints, upright base-plates joints and perforated columns. From the numerical study on the overall structure, the influence of pallet beam-to-column joint is singled out with reference to both service condition and ultimate limit states, but also base-plate joints play a role on the overall stability of the rack systems since they provide stiffness and strength of the column bases. Within SEISRACKS research project (European Commission. Directorate-General for Research 2009), after the execution of experimental tests on components as previously described, a rich experimental campaign was carried out in both CA and DA direction starting from pushover tests, then pseudodynamic tests and finally dynamic tests. The pushover tests were executed in order to evaluate the possibility to propose in a Standard Design Code for Racks in Seismic Areas, static pushover analyses (currently available in many commercially available software packages for structural analysis) as alternative to dynamic (linear or non-linear) analyses, as it seems to be the current trend in many codes for seismic design of building structures. Failures modes were individuated and q-factors were deducted. Then, the pseudodynamic tests were executed only along DA direction in order to assess the seismic resistance and the damage accumulation of the pallet racking system, even if due to the intrinsic quasi-static nature of the pseudodynamic testing procedure, no information was derived about the effects caused by the sliding of the pallets on the beams during a seismic event. So, as an addition to the previous results obtained, dynamic full-scale tests were executed both along DA and CA direction with real palletized goods containing different stuff characterized by different values of density. Failure modes and effects of the sliding of pallets were investigated, as well as the assessment of q-factors. Degee et al (2011) conducted a parameter study comparing the various methods commonly used in practice for analysing the seismic structural behaviour of racks (i.e. modal response spectrum analysis and lateral force method analysis) as well as the different ways to account for geometrically nonlinear effects in these conventional methods of analysis in the case of structures designed for low ductility. Asawasongkram et al (2014) numerically modelled a steel rack structure located in Thailand to assess its structural behaviour under different levels of seismic actions. The model was calibrated on experimental tests executed on the single relevant components. Within SEISRACKS2 research project (C. A. Castiglioni, Kanyilmaz, Bernuzzi, et al. 2014), after an experimental campaign on components, the global behaviour of the case studies structure was assessed through full-scale racks in DA direction. Then, preliminary linear and nonlinear static analyses have been performed according to the FEM 10.2.08 recommendations (*FEM 10.2.08: Recommendations for the Design of Static Steel Pallet Racking in Seismic Conditions* 2011), and finally these models have been calibrated with the results of experimental studies on component and full-scale tests. With the calibrated models, q-factors have been calculated and parameter studies have been carried out to investigate the influence of occupancy rate and merchandise properties. One of the important outcomes of this research is the determination of the procedure for the execution of full-scale pushover tests that previously were not considered as fundamental to be executed, but actually it was pointed out that for the down aisle direction it could be really meaningful (C. A. Castiglioni, Kanyilmaz, Angeretti, et al. 2014; Kanyilmaz, Brambilla, et al. 2016; Kanyilmaz, Castiglioni, et al. 2016). In fact, to catch the stiffness of the structure and possible relevance of second order effects, the shear test on upright frame can be sufficient for the CA direction, at least when the height to depth ratio does not exceed 10 (for higher values the experimental investigation of the large displacement effects can be meaningful). On the contrary in the down aisle direction the full-scale test allows assessing the combined effects of the different properties of the components and connections. This test can be used to calibrate and to validate the numerical pushover analyses performed to assess the behaviour factor.

3.1.3 Interaction of pallets with steel structure under cyclic (seismic) action.

Racks are usually loaded with tons of (more or less) valuable goods. Loss of these goods (due to fall during an earthquake) may represent for the owner a very large economy loss, much larger than the cost of the whole rack where the goods are stored. Racks may be also placed in areas open to the public, or were there are people working. Falling of the pallets, in this case, may endanger the life of those people. In fact, the sliding of the pallets on the racks and their consequent fall represents a serviceability limit state for such structures. This phenomenon depends only on the dynamic friction coefficient between the pallet and the steel beam of the rack and the support conditions of the pallet: if sliding occurs and a pair of beams only directly supports the pallet, the chance of pallets falling down during an earthquake is very large. Only in case additional depth beams (or cross bars) are used, the time of the earthquake of approximately 5 to 15 seconds might be sufficiently small that sliding will not lead to falling down, but only to an overloading of one of the beams (C. Castiglioni 2003). Many times, after an earthquake event, loss of goods was reported, with or without contemporary failure of the steel rack structural system. Most probably, these structural failures might be a consequence of the fall of the pallets and of the impact of the goods on the beams at the lower levels, creating a progressive dynamic collapse (Figure 2-2).
The uncertainties associated with a clear assessment of the causes of such failures (if they were due to structural design faults or if they were caused by fall of the pallets) and with the effects of the sliding of the pallets (without falling) on the structure motivated the research about the study of the interaction of the pallets with the rack structure.
Dynamic tests on four full-scale steel storage pallet racks at the Laboratory of Earthquake Engineering of the National Technical University of Athens were carried out (C. Castiglioni 2003) as a part of a research sponsored by EU within the ECOLEADER Research Programme for Free Access to Large Scale Testing Facilities by the FEM (European Federation of Maintenance) and MIUR (Italian Ministry for Education, University and Research). The main aim of this research, besides comprehending the vibration frequencies and the failure mode of such structures under seismic load, was to study the effects of the sliding of the pallets on the structure (whose governing parameter is the friction factor between pallets and rack beams), that was actually detected to happen at ground accelerations lower than the design rate, and so need to be taken into consideration in the design phase. In particular, dynamic tests were executed on steel racks that were designed by different manufacturers, and the load of the goods was represented by concrete blocks laying on euro-pallets[4] (Figure 3-6). To catch the different aimed

[4] The EUR-pallet, also Euro-pallet or EPAL-pallet, is the standard European pallet as specified by the European Pallet Association (EPAL). The EUR/EPAL-pallet is 1,200 mm × 800 mm × 144 mm (47.2 in × 31.5 in × 5.7 in); it is a four-way pallet made of wood that is nailed with 78 special nails in a prescribed pattern. Around 450-500 million EUR-pallets are in circulation (https://www.epal-pallets.org/eu-en/load-carriers/epal-euro-pallet).

pieces of information, three different dynamic input were given through the shaking table test located in the laboratory of the National Technical University of Athens.

Figure 3-6: An EURO-pallet (https://www.epal-pallets.org/eu-en/load-carriers/epal-euro-pallet/).

Degee and Denoël (2006) investigated the effects of earthquakes on storage racking systems, focusing in particular on the possible sliding and friction of the stored good subjected to a horizontal inertial force. Firstly, a theoretical SDOF system was studied to derive general indications on the sliding behaviour and to serve as a reference for the validation of a more advanced numerical model, that was then applied to a simple rack structure.

SESRACKS research project (European Commission. Directorate-General for Research 2009) also dedicated to the the assessment of the friction factor between pallets and rack beams, which governs the "pallet sliding" phenomenon. Actually, it turned out to be the most important effect governing the dynamic behaviour of racking systems. In particular, both static and dynamic tests were executed, not on full-scale structures but on substructures arranged based on the purposes of the test. Dealing with the static tests, these were executed both for CA an DA direction using an "inclined plane" device by slowly increasing the inclination of the plane, and measuring the sliding of the pallet on the rack steel beams. The influence of different parameters was investigated: type of beam (namely type of surface finish of the beam), type of pallet, geometry and weight of mass resting on the pallet. Dealing with the dynamic tests, these confirmed that "sliding" is, under severe dynamic conditions, the main factor influencing the rack response. Besides, hysteresis loops were obtained, showing the presence of an energy dissipation through sliding.

Gilbert et al (2013) - further implementation (Gilbert et al. 2014) - investigated the influence of horizontal bracing restraints provided by the friction between pallet and rail beams on the static behaviour and design of steel drive-in storage racks. The pallet bracing restraints were shown to significantly influence the structural behaviour of the rack, and their effect on the bending moment distribution of the uprights was studied, with the aim of clarifying the loading patterns governing the structural design and determining the friction coefficient, or strength of a restraining device, required to prevent the pallets from sliding.

Within SEISRACKS2 research project (C. A. Castiglioni, Kanyilmaz, Bernuzzi, et al. 2014), as previously anticipated, full-scale pushover tests were executed, that was then used to calibrate numerical models of the structures previously tested. These numerical models were then used to study also the local behaviour of the racking system regarding the effects of the sliding phenomenon on pallet beams. In particular, to take into account these effects, the following methodologies were suggested to design a sliding-resistant rack structure: (i) additional forces to apply to the pallet beam and geometrical limitations; (ii) correction coefficients for the horizontal bending; (iii) buckling length modification for pallet beams. From the global point of view, the sliding of pallets was proved to provide a damping effect with respect to the

bare structure, that was suggested to be considered through a modification factor of the response spectrum in the design phase.
Adakamos et al (2018) extended the SEISRACK2 research (C. A. Castiglioni, Kanyilmaz, Bernuzzi, et al. 2014) concentrating on three issues in order to verify or improve rules of European Codes on racks: (i) the horizontal seismic forces on the pallet beams; (ii) the developing horizontal bending moments; (iii) whether the buckling length of the pallet beams on the horizontal plane may be reduced due to a potential diaphragmatic action offered by the pallets. Investigations are based on theoretical and numerical analyses, and the main results are: (i) opposite to the Code provisions, the lateral seismic forces are not equal distributed between the pallet beams; (ii) concerning the horizontal bending moments appropriate correction coefficients were proposed that deviate from the codified values; (iii) pallet-to-beam friction does not affect so much the buckling length.

3.2. Guidelines and Regulations

ARSWs constitute the immediate upgrade of standard steel racks. This upgrade came quite fast with the changing of the market and the related necessities, not giving the chance to the regulatory framework to be renewed as fast, too. This resulted in the adoption of the same or very similar design directions, structural solutions, and details used for traditional racks also for ARSWs, although the structural functioning and behaviour of ARSWs and traditional steel racks cannot be overlapped. In the following, the main rules of “good practice”, national technical rules and regulations adopted for the design of these structures are given, starting from those specifically defined for traditional racks, and applied also to ARSWs, depending on the designers' approach and experience.
Among the rules of “good practice”, we can find:

- FEM 10.2.06-1 ‘The design of hand loaded low rise steel static shelving - SHELVING DESIGN CODE’ (2012).
- FEM 10.2.06-2 ‘The design of hand loaded steel static shelving by analytical methods’ (2014).
- FEM 10.2.07 ‘The design of drive-in and drive-through racking - DRIVE-IN DESIGN CODE (2012).
- FEM 10.2.09 ‘The design of cantilever racking – CANTILEVER DESIGN CODE’ (2015).
- FEM 10.2.10 / FEM 9.841 ‘Storage systems with rail dependent storage and retrieval equipment – Interfaces’.
- FEM 10.2.12 ‘Guidelines for revisions to the testing section of EN15512:2009’ (2013).
- FEM 10.02.08: ‘Recommendations for the Design of Static Steel Pallet Racking in Seismic Conditions’ (2011).

Within the rules of “good practice”, FEM 10.2.06 (2012), FEM 10.2.06-2 (2014), FEM 10.2.07 (2012), FEM 10.2.09 (2015) are for a specific typology of steel racks that are different from pallet racks in terms of structural elements, structural behaviour and operating methods[5]. So, they are not applicable to

[5] Drive-in and drive-through pallet racking systems are comprised of uprights and rail beams. The uprights provide vertical support, and the rail beams are used for pallet storage. Uprights are aligned and connected by the rail beams. All the upright lines are put in parallel, independently of one another (no mutual connection), and pallets are placed on two consecutive rail beams. Cantilever racks are comprised of uprights, beams and cantilevers. Along longitudinal direction, uprights are aligned and connected by the beams. Along transversal direction, steel elements are placed along the height of each upright, fixed at one side, ad free at the other one (as a cantilever). This typology of racks is not suitable for palletised goods, but for long, stocky objects.

ARSWs. On the other hand, FEM 10.2.10 / FEM 9.841, FEM 10.2.12 (2013), and FEM 10.02.08 (2011) in the daily practice are also applied to ARSWs. In particular, the research project SEISRACKS (European Commission. Directorate-General for Research 2009) gave the basis for FEM 10.02.08 (2011), where recommendations for the static design of steel racks is given.
Among the national Italian technical rules, we can find:

- UNI 11262-1:2008 'Scaffalature metalliche – Scaffalature commerciali di acciaio – Parte 1: requisiti, metodi di calcolo e prove'.
- UNI/TS 11379:2010 'Scaffalature metalliche - Progettazione sotto carichi sismici delle scaffalature per lo stoccaggio statico di pallet'.
- UNI 11575:2015 'Scaffalature metalliche - Progettazione delle scaffalature drive-in e drive-through per lo stoccaggio statico di pallet'.
- UNI 11598:2015 'Sistemi di stoccaggio statici di acciaio – Scaffalature cantilever –Principi per la progettazione strutturale'.
- UNI 11636:2016 'Scaffalature industriali metalliche - Validazione delle attrezzature di immagazzinamento'.

Among these national technical rules, the first two have been withdrawn from the regulatory framework, while the others, that are valid for steel racks, are not applicable to ARSWs, because they refer to particular typologies of racks (drive-in, drive-through and cantilever[5]) that are different from pallet racks in terms of structural elements, structural behaviour and operating methods.
Under existing community legislation, we can find:

- UNI EN 15512:2009 'Sistemi di stoccaggio statici di acciaio - Scaffalature porta-pallet - Principi per la progettazione strutturale' *(Steel static storage systems - Adjustable pallet racking systems - Principles for structural design)*.
- UNI EN 15620:2009 'Sistemi di stoccaggio statici di acciaio - Scaffalature porta-pallet – Tolleranze deformazioni e interspazi' *(Steel static storage systems - Adjustable Pallet racking – Tolerances, deformations and clearances)*.
- UNI EN 15629:2009 'Sistemi di stoccaggio statici di acciaio – Specifiche dell'attrezzatura di immagazzinaggio' *(Steel static storage systems – Specification of storage equipment)*.
- UNI EN 15635:2009 'Sistemi di stoccaggio statici di acciaio – Utilizzo e manutenzione dell'attrezzatura di immagazzinaggio' *(Steel static storage systems - Application and maintenance of storage equipments)*.
- UNI EN 15878:2010 'Sistemi di stoccaggio statici di acciaio – Termini e definizioni' *(Steel static storage systems – Terms and definitions)*.
- UNI EN 16681:2016 'Steel static storage systems - Adjustable pallet racking systems - Principles for seismic design'.

All of these community legislations are at now, in the daily practice, applied also to ARSWs, even if they have been made for steel pallet racks. In particular, the first two are for the static design of steel racks, and UNI EN 15512:2009 standard specifies the structural design requirements applicable to all types of adjustable beam pallet rack systems fabricated from steel members intended for the storage of unit loads and subject to predominantly static loads. For the seismic design, we can refer to UNI EN 16681:2016, that specifies the structural design requirements applicable to all types of adjustable pallet racking systems fabricated from steel members, intended for storage of unit loads and subject to seismic actions. The basis for UNI EN 16681:2016 were given by the results obtained from SEISRACK2 research project.

For both standards, it is specified that these European Standards give guidelines for the design of clad rack buildings when requirements are not covered in Eurocode 3 (EN 1993) or Eurocode 8 (EN 1998) series. Clad rack building is the general name for warehouses where the racks also constitute their structures, so ARSWs are clad rack buildings with the peculiarity of having the total automatic management of the goods handling. In any case, UNI EN 15512:2009 and UNI EN 16681:2016 are only guidelines and are not written specifically for ARSWs.
With regard with national and community (European) regulations, we can find:

- NTC2018 'Norme tecniche per le costruzioni' *(Technical rules for buildings)* (DM 17.01.2018).
- EN 1993-1-3 'Eurocode 3: Design of steel structures Part 1-3: General rules and rules for buildings.
- EN 1993-1-3 'Eurocode 3: Design of steel structures Part 1-3: General rules - Supplementary rules for cold-formed members and sheeting'.
- EN 1998 'Eurocode 8: Design of Structures for Earthquake Resistance'.

These documents are part of the regulatory framework that is valid in Italy (NTC2018) and in the whole Europe (Eurocodes), respectively for the design of buildings in general (in static and seismic conditions), of steel buildings and thin walled steel members made of cold-formed steel (in static conditions) and of seismic-resistant buildings.
Basically, in today current design, the rules to apply for the design of ARSWs are those for steel buildings contained in the national and community legislation, and for particular aspects it is possible to refer to UNI EN 15512 or UNI EN 16681 no specific guidelines, rules of "good practice", technical rules or regulations have been produced specifically for ARSWs. UNI EN15512 (2009) and UNI EN16681 (2016) are technical standards and are not mandatory to be adopted. Besides, as previously said, they are based on studies and investigations on traditional steel racks, that cannot be confused with ARSWs. An ARSW can be considered a specific and proper structural typology, a building, where the racks constitute its bearing structure but are characterized by specific typological features. As a building, an ARSW should be designed following Eurocodes directions for ordinary buildings, or, in any case, applying compulsorily the National Standards (NTC2018 (DM 17.01.18) for Italy, as an example). In any case, the application of these standards is not easy, considering the typological features and specific needs of ARSWs, that are different from those that can be usually found in ordinary buildings.
As a result, the current design of ARSWs is based on a mixture of the directions defined for traditional racks (EN15512 (2009) and EN16681 (2016)), of the directions for ordinary buildings (Eurocodes and National Standards), and finally, the experience of rack designers, that rapidly adapted these structures to the needs of the evolving market.
Finally, in Italy, the "Servizio Tecnico Centrale" (Central Technical Service) of the "Consiglio Superiore dei Lacovi Pubblici" (Supreme Council responsible for overseeing public works) is currently drawing up a document titled "Linee guida per la progettazione, esecuzione, verifica e messa in sicurezza delle scaffalature metalliche" (Guidelines for the design, execution, verification, and safety of steel racks). These Guidelines aim to give a regulatory and procedural framework for the design, the upgrading and the adjustment of steel rack in seismic conditions. These Guidelines highlight the relevant regulatory aspects to design new steel racks and give the first directions for the seismic vulnerability assessment of the existing ones. Besides, these Guidelines are developed in accordance with the current NTC2018 (DM 17.01.18) and consider the specific existing regulations available for steel racks. In these guidelines, a specific reference to ARSWs as a proper structural typology is given, pointing out the main

directions given within EN16681 and how to apply them to ARSWs, suggesting how possibly reconsider them.

4. Open Problems

ARSWs are the latest in the storage solutions field. When the number of pallets is relevant and the daily handling operations are numerous, they offer the best options in terms of goods flow management and space optimization. They constitute the direct upgrade of traditional pallet racks, and, to follow the fast-evolving market request, they acquired most of the structural features of racks without being supported by a specific regulatory framework. In this respect, Haque and Alam (2015) focus on the seismic design aspect, highlighting that as this structural system is relatively new compared to the traditional steel structures, very little effort has been put into the development of a seismic design guideline for them. In particular, in their study, pushover and incremental dynamic time history analyses have been carried out on a case study structure to calculate the overstrength and ductility related force reduction factor. The case study structure was designed adopting the force-based design. The same authors previously evaluated the structural performances of another case study that was designed by the adoption of the direct displacement-based design approach (2013).
The lack of a specific regulatory framework brought with the time to relevant catastrophes that highlight the lack of knowledge that concerns these structures (Figure 4-1 represents collapse of the warehouse "Ceramiche Sant'Agostino" after the "*Emilia Romagna Earthquake*" that happened in Italy in 2012; the warehouse was built in 2001). The absence of specific prescriptions to be followed to the design of ARSWs results, in most cases, in the adoption of the same guidelines defined for traditional steel racks (UNI EN15512 (2009) and UNI EN16681 (2016)) and to the adoption of the same structural choices (starting, from the global point of view, with structural schemes, and, from the local point of view, with the material, cross-sections for the structural elements and structural details as connections).

Figure 4-1: Collapse of Ceramiche Sant'Agostino warehouse after the "Emilia Romagna Earthquake" that happened in Italy in 2012.

In any case, this kind of approach it as actually allowed: as previously said (§3.2), it is specified by both UNI EN15512 (2009) and UNI EN16681 (2016) that these community legislations are dedicated to traditional steel pallet racks but they can be also taken as guidelines for the design of clad rack buildings when requirements are not covered in Eurocode 3 (EN 1993) or Eurocode 8 (EN 1998) series. This statement suggests also that ARSWs cannot be considered as standard racks, but as buildings, and so they have to be designed following the directions contained within those European Standards that are written for buildings. Ultimately, being huge and complex buildings, ARSWs have to be characterized by the same safety levels of the other constructions and have to be designed following technical standards.

Anyway, the application of Eurocodes regulations for buildings is not easy and may be not suitable, because ARSWs are characterized by particular features that are not common for ordinary buildings:

1. From the global geometrical point of view, they are very tall buildings and have a short inter-storey height: the distance from one load level to the other one is usually from 1 m to 1.5 m. In general, the common inter-storey height for buildings is around 3 m. In this regard, Caprili et al (2018), in the first part of their study, tried to design an ARSW case study following European standards (EN 1993 and EN 1998). The geometry of the structure was adopted as common to existing ARSWs, but it was chosen to use hot-rolled elements instead of the typical cold formed sections, with the aim of evaluating possible alternative solutions for these structures. Two approaches (elastic and dissipative) were adopted for the design, highlighting differences in terms of steel weight, seismic performance at Life Safety (LS) limit state, collapse mechanisms, construction details and other relevant aspects. In particular, regarding the dissipative approach, it was not possible to apply all the restrictions in terms of maximum over-strength factor variation[6] and non-dimensional slenderness ratio λ, that are mandatory to be respected, neither applying a great variation in section, thickness and steel grade of bracing profiles. This impossibility is due to the reduced inter-storey height and the high number of storeys, that make ARSW structural typology very different from the common steel buildings.
2. ARSWs are characterized by a high live-to-dead load ratio (as an example, for racks the medium ratio between the vertical design load and the weight of the structure is 5% (Orsatti 2013)). Besides, since the distribution of loads is driven by the logistic rules that govern the goods flow, it may happen that there is not a uniform distribution of the pallets along the height and the length of the structure. This means that, referring to the design of these structures, considering the full load condition, as happens for ordinary building, may be the worse one for the static design, but considering the seismic conditions, this may lead to neglect alternative pallet distributions that could generate additional relevant eigenmodes and corresponding dangerous forces distributions.
3. From the global point of view, the structural typology is very atypical and characterized, along transversal CA direction, by steel frames, where the columns are constituted of truss. The presence of the mutual connection between all the truss is possible only in case of multi-depth warehouse typology, while for the others is not possible due to the operational and logistic layout (§2.2). In any case, all the trusses are surely connected at the top through the roof structure. In the longitudinal DA direction, the steel frames are connected through the pallet beams with semi-rigid connections, and the stability toward horizontal actions along this direction can be given through additional vertical bracing systems that have to be placed without compromising the functionality of the warehouse (aligned and localized bracing towers or eccentric and diffused or concentrated bracing system). The presence of rigid floors is in general not possible, with the only exception of multi-depth warehouse, where the trusses are all connected, and so a diffused horizontal bracing can be placed, but only at the load levels, without interfere with the goods handling operations.

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[6] The over-strength factor Ω_i for each dissipating element is defined as the ratio between the tensile force resistance ($N_{pl,Rd,i}$) and the design force (N_{Ed}). The maximum variation allowed among all the dissipating elements, corresponding to the difference of the maximum and the minimum values of Ω_i, is equal to 25%.

4. Dealing with structural choices, cold formed elements are used, and this means having thin walled class 3 or 4 sections, with also particular features as, for examples, continuous holes along the length of uprights. In static conditions, the use of these elements is in general allowed also for primary structures, with the support of FEM or adequate advanced analysis and/or experimental testing to assess the structural performances, if they are not easily predictable. In particular, prEN1993-1-3:2019 (§4.(5)) individuates the "Structural Class I" for applications in which cold-formed steel members are designed to contribute to the overall strength and stability. In seismic conditions, prEN 1998-1-1:2019 provides a new categorization of seismic resistant buildings regarding their deformation capacity and cumulative energy dissipation capacity (§4.4.2.(3)): In Ductility Class 1 (DC1) only the overstrength capacity is take into account, while deformation capacity and energy dissipation are disregarded (this means using a maximum behaviour factor equal to 1.5); in DC2 the local overstrength capacity, the local deformation capacity and the local energy dissipation capacity are taken into account; in DC3, the ability of the structure to form a global mechanism at Significant Damage limit state and its local overstrength capacity, local deformation capacity and local energy dissipation capacity are taken into account. For DC1 class, since the structure substantially remains in the elastic field, no specific requirements are requested, according to prEN 1998-1-2:2019, §11.7.2., and so, also the use of cold formed elements is allowed as long as Eurocode 3 (EN 1993-1-1 and EN 1993-1-3) provisions are respected. For DC2 class, some restrictions are given for each structural typology. In particular, the basic ones refer to the limitation of the behaviour factor and the class of section allowed. Referring to the latter, class 3 and 4 sections can be used only for portal frames, single-storey moment frames and lightweight systems. These structural categories are not applicable to ARSWs, where usually concentric bracings are used, and for them, class 1 or 2 can be only used. Another important aspect to be considered is the design of the connection of the dissipative elements: for dissipative structures within DC2 and DC3 classes, the connections of the dissipative elements has to be over-resistant with respect to the element strength. This condition is not easy to be fulfilled with the sections commonly adopted for diagonals in ARSWs: Figure 4-2 shows an example of a typical upright-to-diagonal connection along CA direction, where the diagonal is the dissipative elements. In this example, the diagonal has a C-shaped cross-section and it is bolted directly on the upright. Besides, as all the other cold-formed elements used for racks, it is characterized by a very low thickness. Because of this reason, the mechanism that is characterized by the lower resistance for the connection is very probably the bearing one (plastic ovalization). The bearing resistance of a bolted connection with thin-walled cold formed elements involved, can be calculated as follows (prEN1993-1-3:2019 table 10.5):

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

where: α_b depends on the geometry of the connection (in any case $\alpha_b \leq 1.00$); k_t depends on the thickness of the element, and if the thickness is minor than 1.25 mm, k_t is minor than 1.00, but in any case $k_t \leq 1.00$; 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 to be used for checks on steel connections. In case of having both α_b and k_t equal to 1.00, the only way to increase the bearing resistance of the connection is to increase the thickness of the section, or using a material with a higher strength, or increasing the number of bolts. The first two possibilities also increase the resistance of the cross-section, that implies that also the design force of the connection is increased, so the circle keeps turning. The increase of the number of bolts, if allowed by the geometry of the profile and if the minimum distances for holes are respected, has to be balanced with the net section check, that gets difficult to be satisfied since the net section reduces.

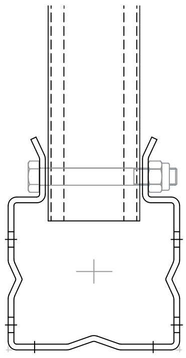


Figure 4-2: Common solution for an upright-to-diagonal connection.

These factors highlight that ARSWs are particular structures with features that are not common for ordinary buildings, and so Eurocodes may be not adequate and may not cover all their particular necessities. At the same times, the technical rules for racks (UNI EN15512 (2009) and UNI EN16618 (2016)) may appear more suitable, but only from the "local" point of view. In fact, ARSWs and traditional pallet steel racks have the following characteristics in common: (i) structural choices in terms of material (cold formed steel), profiles (cross-section shape), additional distinguishing features (holes along the height of the uprights); (ii) structural details, as for example non-traditional pallet-to-upright connections, as well as single-bolted no-plate-added diagonal-to-upright connections. But, even if the structural behaviour of the single, isolated members and components is comparable, the global structural functioning is not the same given that ARSWs are buildings, and they should be treated as such and be characterized by the appropriate safety level. The lack of a specific regulatory framework for ARSWs is indeed relevant and this aspect has to be implemented.
From this perspective, the Limit States (LS) as defined by Eurocodes, should be re-arranged for this particular structural typology:

- Construction Phase. This is not an actual LS, but during this phase in necessary to consider wind effects on the bare or partially cladded structure and the necessities connected to the automated handling of goods.
- Serviceability Limit State (SLS). Referring to the static conditions, deformations caused by wind action has to be limited in order to not compromise the functionality of the warehouse. Considering the seismic conditions, the possible sliding of pallets has to be considered and it is necessary to prevent the pallet to fall because of safety reasons, the possible damage of the pallet or of its content, and because a fallen pallet may impede the automated devices to move (as an example, if a pallet falls down in an aisle, it may impede the stacker crane to move, slowing down the regular activity of the warehouse).
- Ultimate Limit State (ULS). As a general consideration, the most relevant aspect is that the design methodology needs to be supported by experimental activity to predict the ultimate resistance of components or elements that are atypical and not standardisable. Besides, the design is at now

made considering full load conditions, without considering that the distribution of unit load is made through logistic criteria, that could determine alternative load distributions that may be dangerous both for static and seismic conditions. Considering seismic conditions only, there are several issues that have to be taken into account. From the global point of view, the main aspects deal with: (i) the contribution of pallet units to the energy dissipation and quantification of participating mass; (ii) based that pallets may be distributed in a non-uniform manner, alternative mass (and load) distribution inside the warehouse may be relevant to be considered; (iv) evaluation of a possible capacity design for ARSWs and definition of the proper behaviour factors.

The erection procedure to follow during the Construction phase of an ARSWs is not determined *a priori*, but it is decided by the designer of the structure and is organized so that stability is guaranteed at each step (Figure 4-3). Anyway, especially when the structure is partially cladded, unexpected wind gusts may cause premature collapses (Figure 4-4). This effect has obviously to be prevented and avoided. With this aim, the effects of the wind during the assembly phase should be studied, in particular focusing on two cases: structure partially cladded and bare structure, so the study of the effects of wind on the elements (also holed ones), too. These aspects have at now not been taken into consideration, since traditional pallet racks are placed inside the warehouse, so the effect of wind is not considered. Another important aspect deals with the necessity related to the automated management of goods flow. In fact, being the handling of goods totally automated, there are tight tolerances to be respected to allow the correct functioning of the machines, that impose, for example, the absolute verticality of the uprights and the precise respect of the geometrical dimensions required. As a reference point for the tight tolerances to be respected, FEM 10.2.10 / FEM 9.841 and UNI EN 15620:2009 are available, where limits are specified depending on the retrieval machines used within the warehouse.

Figure 4-3: Assembly phases of a single-depth ARSW.

Figure 4-4: Partial collapse of an ARSW under construction.

At SLS, referring both to static conditions, the tight tolerances that involve automatic storage (FEM 10.2.10 / FEM 9.841 and UNI EN 15620:2009) impose also limited elastic deformation, e.g. due to wind action: if excessive, they can result in an interruption of activities. This aspect has to be taken into consideration also referring to seismic action. Moreover, seismic accelerations may cause the sliding of pallets, when the dynamic horizontal force of the pallets exceeds the friction between the pallets and the beams where they lie. In fact, unit loads are in general simply supported vertically by the racking structure and kept in their position when loaded by inertial actions only by friction. On one hand, this phenomenon contributes to the dissipation of seismic energy, reducing the seismic action on the racks. On the other hand, the excessive sliding of pallets may cause them to fall, and the fallen pallets may compromise the operational activities of the warehouse, i.e. hinder the roads of the stacker cranes, making not possible to use them. At SLS, the sliding of pallets should be limited with the aim to avoid the pallets to fall or be misplaced on the beams, not allowing the devices to retrieve them anymore; anyway, at the same time, the sliding of pallets should not be totally avoided, considering their contribution to the dissipation of energy. From EN 16618 (2016) §9.2.2.2, it is suggested that in seismic zones, when displacements of the unit loads are likely to occur, additional components shall be installed or appropriate countermeasures shall be taken in order to prevent pallet falling inside the warehouse and cause local or global collapse of the structure, injury to persons and damage to stored goods, especially in case of high racks and narrow aisles. This suggestion is justified by the fact that the real displacement due to sliding under a severe earthquake is quite unpredictable because of the random nature of earthquakes and also for the number of parameters which can affect the behaviour of pallets (i.e. friction coefficient, etc…).

At ULS, the main general fact is that ARSWs are composed by non-standard elements that really affect the structural behaviour of the whole structure both at static and seismic conditions. Being non-standard, experimental tests may be used to assess of their ultimate resistance force and their structural behaviour under cyclic action for seismic conditions. This kind of approach, that is called "design by testing", it is at now proposed and supported by EN 15512 (2009) and EN 16681 (2016), but it surely makes the design phase very laborious. The components may need this kind of approach are the following:

- <u>Uprights</u>. According to the typical structural choices for traditional racks, ARSWs' uprights are always made by cold-formed profiles with holes along all their height to assure fast and easy assembling connections to beams (along DA direction), and diagonals (along CA direction). The typical cross-section adapted for these elements is the U-shaped ones with lips, that allows a very easy connection for the diagonals, that can be directly inserted inside the upright profile and hinged through a single bolt. Due to vertical loads, the relevant force acting is the axial compression one. Besides, biaxial bending can be found due to the continuity of the upright also in the intersection with the other elements (beams and diagonals). Since they are cold-formed thin-walled sections, they can be classified 3 or 4 according to Eurocode3 (EN 1993-1-1) classification for steel sections and they may be affected by local buckling phenomena that reduce their resistant section. In addition to this, the effect of the holes has to be taken into consideration, studying their effect and influence on the buckling resistance of the profile. In any case, the littler is the pitch of the holes, the higher is the impact on the reduction of the effective area of the cross-section in compression, and so on the performance of uprights. Besides, the particular shape of the section, that is opened and mono-symmetric with a consequent non-coincidence of the shear center and the centroid can induce a particular buckling mode based on the free length of the elements. In fact, for short elements, local buckling is usual, for longer elements, usually, global buckling occurs, and finally, for intermediate lengths, distortional buckling mode happens (Figure 3-4). This is a particular mode that is caused by the out-of-plane buckling of the external lips of the section. Whatever is the buckling mode, it is necessary to assess the effective area of the element that has to be taken into consideration in the verification checks, in order to obtain a trustable value of the resistance of the upright (prEN 15512:2018) §Annex. The value of the effective area can be assessed via experimental tests (stub column test for the effects of local buckling and compression tests for the effects of global buckling), by performing linear or non-linear buckling analysis using numerical methods, or by using a simplified procedure that requests the determination of the elastic buckling stresses (prEN 1993-1-3:2019 §7.6). Referring to this last method, the determination of the elastic buckling stresses can be done through numerical methods as Finite Element Methods (FEM) or Finite Strip Methods (FSM) (Á. Ádány S. and Schafer 2006b; B W Schafer and Ádány 2006). These directions are applicable to not holed uprights, but, if upright has holes along its length, the simplified procedure is not applicable anymore, and so, only the experimental tests or the numerical methods remain to assess their ultimate strength, making very hard the design phase.

- <u>Pallet beam-to-upright connections</u> are surely non-standard. In fact, in order to speed-up the assembly procedures, beam-upright connections are not bolted nor welded. One of the most used connection types is the hooked one: at the ends of each beam there are special pieces also called "beam-end-connectors" that are on one side bolted or welded to the beam, while on the other they are shaped in order to be directly inserted into uprights' holes (Figure 2-15). Since there are not specific prescriptions to design these connections, there is a huge variety of commercially available beam-end-connectors. As a consequence, it is not possible to individuate an identical structural behaviour, and this is the reason why codes (e.g. EN 15512 (2009)) suggest to execute experimental tests to predict their stiffness, ultimate resistance and structural performance both for static and cyclic loads, and obtain moment-rotation curves that are necessary for numerical analyses and in general for the design of the connection. As an instance, Figure 4-5 shows a moment-rotation curve obtained from experimental monotonic and cyclic tests executed on a specimen beam-to-column connection inside the SEISRACKS2 Research Project (C. A. Castiglioni, Kanyilmaz, Bernuzzi, et al. 2014) and highlighting the non-linear behaviour that characterizes them. In summary, the main issue can be identified in the fact that, if referring to typical connections between steel elements in civil/industrial constructions, beam to upright connections for steel racks are non-standard because of the way they are realized (not bolted, not welded) and also because each ARSWs producer independently designs them. This is why EN15512 (3) suggests the execution of experimental bending tests on these connectors to determine their stiffness and bending strength. These two factors depend on lots of variables: (i) the type of the upright; (ii) the thickness of the upright; (iii) the type of the beam; (iv) the position of the beam on the connector; (v) the method of connection of the beam to the connector; (vi) the bracket type; (vii) the properties of the material used. These aspects and the influence of these parameters in the structural behaviour of these connections were studied within SEISRACKS and SEISRACKS2 (European Commission. Directorate-General for Research 2009; C. A. Castiglioni, Kanyilmaz, Bernuzzi, et al. 2014), but, in any case, the number and the variability of these parameters highlight the necessity of executing these tests on each produced specimen and the difficulty to extend the result obtained from one specimen to another one, even if they are similar.

- <u>Uprights base connections.</u> They are characterized by a non-linear behaviour that can be noticed for example, from moment-rotation curves obtained by experimental monotonic and cyclic load tests on specimens produced by industrial partners participating to SEISRACKS2 research project (2) (Figure 4-6). The methodology to be followed to execute these tests is described within EN 15512 (2009). Given the fact that each ARSWs producers independently designs them and all the variables that influence the structural behaviour of connections, it's necessary to execute experimental tests on each produced specimen, also because it may be difficult to extend the result obtained from one specimen to another one, even if they are similar.

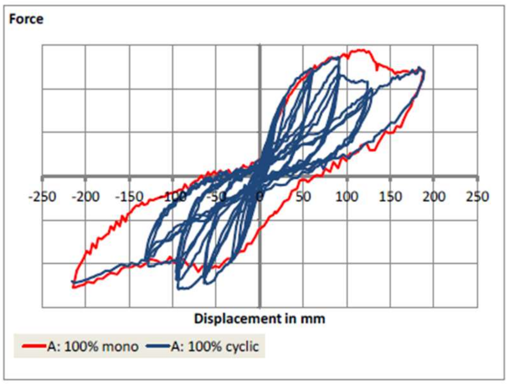


Figure 4-5: On the left a typical example of beam-to-upright hooked connections; on the right an example of moment-rotation curve obtained from experimental monotonic and cyclic test executed on a specimen of the same typology inside SEISRACKS2 research project (C. A. Castiglioni, Kanyilmaz, Angeretti, et al. 2014).

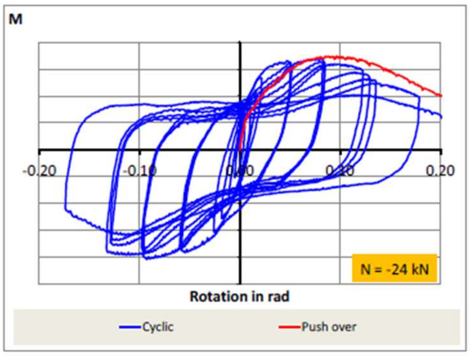
 

Figure 4-6: On the left, a typical example of upright base connection; on the right an example of moment-rotation curve obtained from experimental monotonic and cyclic test in down-aisle direction executed on a specimen of the same typology for SEISRACKS2 research project (C. A. Castiglioni, Kanyilmaz, Bernuzzi, et al. 2014).

Among the previous aspects, it should be pointed out that, considering DA direction, ARSWs, contrarily to standard racks, have always vertical bracings, whose stiffness toward horizontal action is much higher than the one corresponding to upright and pallets beam frames. This implies that seismic actions are taken mainly by vertical bracings, and in seismic conditions the structural response of the warehouse along DA direction is mainly represented by the vertical bracings, their layout, their position with respect to the load planes (that are aligned with the uprights), and the way they are connected to the rest of the structure (§2.2.1). Basically, this means that, with the purpose of modelling and catch the global behaviour of the structure, the assessment of the stiffness of pallet beam-to-upright and upright base connections for DA direction may be not relevant, and, for the verification checks, the strength resistance of particular pieces (as the beam-end-connector for pallet beams) is usually provided by its producer.

Another relevant aspect that should be considered is related to the distribution of loads inside the warehouse. At now, the design of ARSWs considers only the empty/full loaded condition of the entire warehouse, with safe but very penalizing factors for combinations of variable actions at static ULS (γ_Q combination factor for pallet load is equal 1.5). This factor could be considered very penalizing also because the weight of unit loads is always checked at the entrance of the warehouse, and weight limits are fixed for pallet beam and uprights. All these parameters are recognized as boundary conditions for the logistic rules, and so, as an example, even if there is a free spot available but the maximum load on the corresponding upright would be exceeded, the pallet goes in the next available spot. Recognizing the influence of the actual distributions depending on the type of warehouse, loading strategy, and rules imposed by the automation is therefore crucial in order to optimize the design and to give the bases for the definition of the proper design strategies.

Considering seismic conditions, there are several issues that have to be taken into account. From the global point of view, the main aspects are: (i) the contribution of pallet units to the energy dissipation and quantification of participating mass; (ii) based that pallets may be distributed in a non-uniform manner, alternative mass (and load) distribution inside the warehouse may be relevant to be considered; (iii) evaluation of a possible capacity design for ARSWs and definition of the proper behaviour factors.

(i) Seismic acceleration may cause the sliding of pallets, when the dynamic horizontal force of the pallets exceeds the friction between the pallets and the beams where they lie. Unit loads are sub-structures with distinct dynamic characteristics in terms of frequency and damping, and their behaviour affects the response of the whole system. Within SEISRACS2 (C. A. Castiglioni, Kanyilmaz, Bernuzzi, et al. 2014), the movements of the unit loads during low dynamic acceleration on a full scale structure were observed. Then, this phenomenon was analysed with support of numerical simulations, and it was detected that through the small movements that occur when friction is exceeded, pallet units contribute to energy dissipation of the system. The consequence of this phenomenon is the reduction of seismic action on racks, since a lower horizontal action is transferred from the pallets to the structure. Within EN 16681 (2016), the effects of these typical phenomena of racking structures, such as energy dissipation due to the pallet-beam friction, damping due to the movement of the stored products, pallet flexibility, and others, are taken into account through the modification of the seismic response by means of three coefficients: ED_1 and ED_3 are the design spectrum modification factors and ED_2 is the mass modification factor. All of them are minor or equal then 1.0. ED_1 depends basically on the reference period of the structure and on the friction coefficient between pallet load and steel structure (EN 16681 (2016) §7.5.2): given the same material for pallets (the same friction coefficient), the lower is the reference period (higher seismic acceleration, higher inertial forces), the higher is the value of ED_1, so the minor is the reduction of the spectrum. The reduction of forces transferred on beams was indeed detected for low ground accelerations, when the movement of unit loads is small. ED_3 (EN 16681 (2016) §7.5.2) is introduced to account for the dissipative phenomena typical of the dynamic behaviour of racking structures under seismic actions that are observed on racks that have suffered earthquakes, and from the full-scale tests performed on shaking tables. The value of ED_3 equal to 0.8 corresponds to a conventional viscous damping ratio ξ of 10% of the loaded rack describing dissipative mechanisms existing in the whole system. In any case, the maximum value of $ED_1 \cdot ED_3$ is 0.4. The final value of the reduction of the response spectrum is given by the K_D factor, that depend on the product between ED_1 and ED_3, and the ratio between the pallet load and the whole structure load (including dead weight, permanent weight and pallet weight combined as for seismic conditions). If the ratio between the pallet load and the whole structure load is major than 90%, K_D can be taken equal to the product between ED_1 and ED_3, and so it can lead to a maximum reduction of 60%.

Factor ED_2 is used to modify the seismic participating mass associated to pallet units, and it depends on the consistency of the content of the pallets (EN 16681 (2016) §7.5.5). As an example, for liquid and compact content, ED_2 is equal to 1.0 (no mass reduction), while for weak content, ED_2 is equal to 0.8 (20% of mass reduction). In this regard, according to EN 16681 (2016), the formula for the definition of the participating mass is the following:

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

where R_F is the rack filling grade reduction factor, related to the occupancy of stored goods in the rack that can be assumed during the seismic events. Usually, for analysis in CA direction RF equal to 1,0 is assumed (higher probability to have the frame fully occupied), while for analysis in down aisle direction $R_F \geq 0{,}8$ shall be assumed (minor probability to have the full length of the frame occupied).

All of these considerations have been made on studied executed on pallet racks, and the possibility to extend them to ARSWs should be verified, also because they should lead to a strong reduction of seismic design acceleration.

(ii) The distribution of loads inside an ARSWs is driven by logistic strategies. At now, mass distribution is considered uniform along the length and the height of the warehouse, and also the reductions previously introduced (R_F and E_{D2}) are applied to the whole quantity of mass, not altering the distribution. Anyway, the logistic strategies may lead to different mass distributions that could affect the seismic response of the structure. A positive aspect of automated warehouses is that the distribution of the goods, and therefore of the masses, is fully controlled by the automation, so can be tracked and studied for further applications.

(iii) EN 16618 (2016) provides recommendations for the design of steel racks in seismic condition (§8). In particular, two different design concepts are proposed: the low dissipative and the dissipative structural behaviour. Basically, for the low dissipative behaviour, the maximum applicable q factor is equal to 2, while is major for dissipative behaviour, and in this latter case, it is recommended to apply the rules within Eurocode 8 (EN 1998). Dealing with low dissipative behaviour, all the structural schemes can be assumed for the bracings, but the q factor is limited to 1.5 if K, D, Z layouts are assumed, where the contribution of diagonals in compression has to be taken into consideration. Basically, for K, D, Z layouts the elastic design is the only possible choice. For other structural schemes (i.e. concentric X-shaped bracings), although a q factor equal to 2 can be used, no over-resistance of the connection with respect to the dissipating element is requested, and no other hierarchy rules are indicated for the other non-dissipating elements. The only limit is for bolted connection, where the shear strength of bolts is requested to be higher than the bearing resistance of the profile, in order to avoid fragile mechanisms. Regarding ARSWs, since they are building, these regulations for low dissipative racks appear not to be suitable, counterposing and being in conflict with the regulations imposed for buildings. In any case, as previously anticipated in this chapter, also the recommendations within Eurocode 8 (EN 1998) are not suitable for ARSWs, and so specific regulations for such particular structures have to be provided.

4.1. Concluding remarks

ARSWs are huge and complex buildings, different from the ordinary constructions and traditional racks in terms of geometry, load patterns, structural configuration and tight tolerances due to automated storage. To follow the fast upgrade of market requests, they inherited form traditional racks the same structural components, starting from cross-sections of the main elements to the connection details, without being supported by a specific and dedicated regulatory framework. Being actual buildings, they have to guarantee the corresponding safety levels, but rules for buildings (i.e. Eurocodes) are not fully applicable, because ARSWs are characterized by geometrical, structural ad load entity features that are not common for ordinary buildings. At the same time, the technical regulations for pallet racks (EN 15512 and EN 16681) are not suitable, especially for seismic design approaches, since they are dedicated to racks, and the global structural behaviour and necessities cannot be overlapped. This is why ARSWs need proper design specifications that consider all the peculiar features that identifies them.

5. Objectives and Methodology

This PhD thesis aims to develop a new approach for the design of ARSWs. In particular, based on an accurate evaluation of safety levels and the design strategies now adopted in current practice, and given the lack of a regulatory framework for such structures, the main objective is to define specific and innovative design approaches for ARSWs in seismic conditions, focusing on the Double-Depth structural typology. This approach is defined assuming a dissipative behaviour for these structures (depending on the structural schemes) and evaluating different and possible yielding patterns as an alternative to the global collapse mechanism. In any case, the optimization of cost-benefit ratio is always taken into consideration as one of the design goals (cost-benefit ratio consists of costs connected to a wide variety of construction details that may be implied by capacity design; benefits related to dissipative behaviour that allow obtaining controlled yielding pattern and lighter structures).

This new design proposal, which is mainly developed and focused on seismic conditions, has to fulfil all the necessities and performance expected at each Limit State:

- At ULS, the design proposal shall satisfy the resistance and ductility request, optimizing the structure's design both at local and global point of view, always taking into consideration the current structural choices and the cost-benefit ratio.
- At SLS, to allow automated storage, pallets fall and excessive residual deformations shall be prevented. In case they happened, they would not make possible to use the stacker cranes for automatic handling of pallets, and the warehouse would be temporarily unusable. The design of the structure at ULS shall be compatible with the expected performances at SLS, controlling the deformability and the residual deformations of the system, at least for low-medium intensity seismic actions.

The study and the analysis of ARSWs are widely performed inside the European research project "*STEELWAR:* Advanced structural solutions for automated *STEEL* rack supported WARhouses", funded by the Research & Innovation, Research Fund for Coal and Steel (RFCS) and coordinated by the University of Pisa. Thanks to this project, a series of experimental tests and research will be executed to support the results of the present PhD thesis. Besides university institutes, the research group participating in this project is formed by two engineering companies with specific competencies on design and inspection of rack systems and knowledge of the logistic industry, by five big rack producers (selling their rack systems solutions in Italy, Europe and Overseas), and by a supplier of storage technology systems.

To achieve the objectives above presented, the research is organized in the following steps.

(i) *Analysis of the more suitable typologies of numerical analyses for the structural assessment of double-depth ARSWs structures*

An ARSW case study is dimensioned following Eurocode 8 directions, both adopting elastic and dissipative approaches. The final aims of this study are: (i) evaluate the applicability of Eurocodes prescriptions for steel buildings to ARSWs, focusing in particular to the capacity design rules; (ii) give a comparison of the two design approaches in terms of structural performances, also looking at the post-elastic behaviour of the two structures; (iii) evaluate the more suitable and efficient non-linear structural analysis for ARSWs. The methodology adopted to achieve these aims is organised in the following steps: (1) execution of non-linear numerical analyses of both structures, using lamped plasticity models; (2) implementation of the contribution of braces in compression within the non-linear numerical analyses of the dissipative structure only. In both cases, non-linear static analyses are carried out on bi-dimensional plane frames representing the typical transversal CA section of the case study building.

(ii) *Assessment of structural response of case study structures in static and seismic conditions*

The structural assessment of five case studies is carried out to comprehend the existing structures' current design strategies. These case studies are designed in a high seismicity area by the Industrial Partners that participate in STEELWAR research project. The structural assessment of these structures is done through 3D or 2D models – based on the possible and doable simplification of the geometry of the system - and executing dynamic analysis, where the seismic input is defined according to natural accelerograms that are selected from the available database to obtain the worst damage scenario for the structural typologies considered, in relation to the seismic intensity level considered (geometric nonlinearities are included). Besides the structural assessment, the analysis of the design strategies currently adopted is executed to point out the positive or questionable aspects.

The results obtained from the structural assessment and the analysis of the current design strategies are used as a starting point to define the possible strategies to optimize the design of ARSWs.

(iii) *Development of a new design methodology*

A new design approach is developed starting from the critical issues found in the analysed structures. The starting design rules are in line with those within Eurocode 8 (prEN 1998:2019). This new approach starts from the optimization from a global point of view, focusing on reducing eccentricities and all possible geometrical aspects that may negatively influence the system's structural behaviour. Then, the design inputs are discussed – as, for example, the definition of loads and participating masses - to point out the righter strategy. The more appropriate structural typologies to assure the desired dissipative structural behaviour are individuated. In this regard, the possibility to involve in the plastic mechanism mainly the lower part of the structure is investigated, while the higher remains in the elastic field. In this lower part, the more restrictive rules corresponding to the medium seismicity class – as defined by Eurocode 8 (prEN 1998:2019) – are applied. Finally, based on the previous step's issues, optimization at local point of view is made, focusing on elements and structural details optimization.

(i) *Numerical validation of the re-designed structure, according to the new approach*

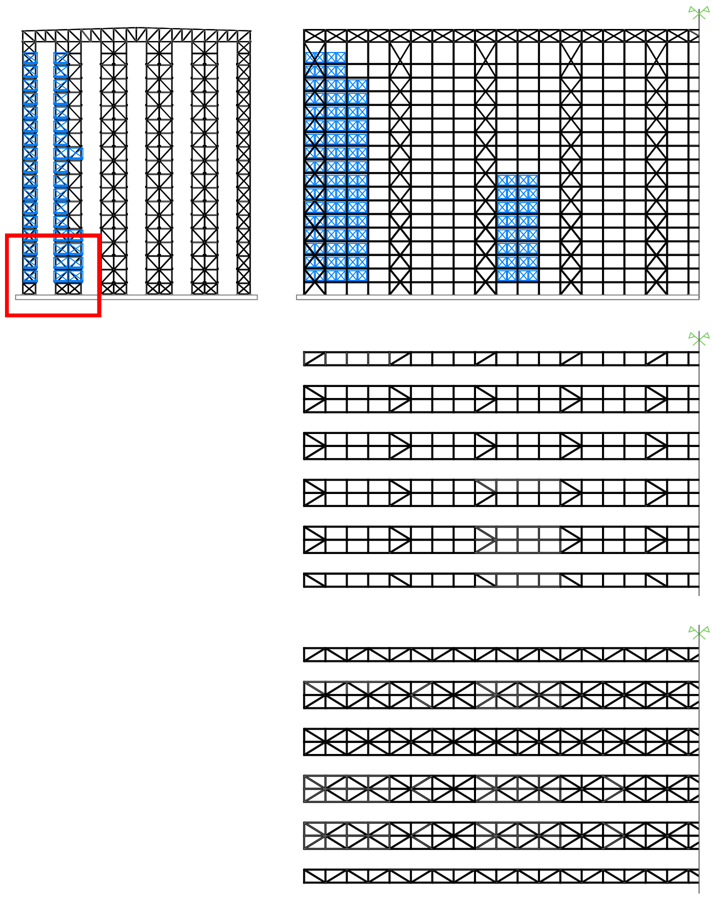
The structural assessment of the structures designed with the new approach is made by executing non-linear dynamic analyses on 2D models, considering both geometrical non-linearities and the structural behaviour of dissipative elements. The other elements are modelled as elastic and then checked in the post-process, through the execution of safety checks. A critical analysis of the new design approach is made.

As future development of this research, further analyses have to be performed to assess the structure's effective global ductility level. Besides, experimental validation of the structural behaviour and the local and global ductility level reached with the proposed design must be performed. Some substructures will be extracted from the re-designed structures, and tested. Experimental testing must also be carried out on the dissipative elements to assess the actual behaviour under cyclic load, which within this research is numerically determined. The experimental validation will be executed in the laboratories of the University of Pisa and of Aachen, in the framework of the RFCS STEELWAR Project. This experimental campaign results are used to re-calibrate the numerical models and finally adjust and define the design method.

6. Analysis of the more suitable typologies of numerical analyses for the structural assessment of double-depth ARSWs structures

The preliminary studies of this research deal with the structural assessment of an ARSW case study designed following European standards (EN 1993 and EN 1998), considering both elastic and capacity design. The main goal is to evaluate the applicability of Eurocodes standards to the design of ARSWs, taking as a benchmark structure a double depth warehouse. Besides, possible typology of numerical analyses for the assessment of post-elastic structural behaviour of this structural typology of warehouses is evaluated. The main results of this studies have been illustrated within Caprili et al (2018).

The geometry of the structure was defined as common to existing ARSWs (Figure 6-1), but it was chosen to use hot-rolled elements instead of typical cold formed sections, with the aim of evaluating possible valid alternative structural solutions for these structures. Besides, all the structural choices (from the global to the local aspects) were made in order to maximise the structural performances of the building (Table 6-1). Along CA direction, the diagonal composing the steel trusses were arranged in the X layout, and also along DA direction, the stability toward horizontal actions was given through X-shaped arranged diagonals placed as bracing towers every 3 spans. The tension only strategy was adopted for the design of the bracing systems, both for CA and DA direction.



a)

b)

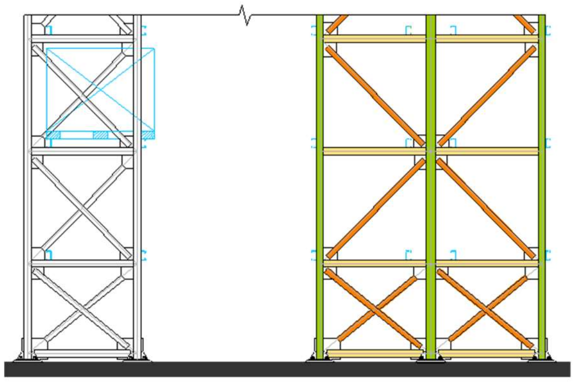
c)

Figure 6-1: a) Transversal section and longitudinal section of the ARSW case of study. With the blue colour an example of pallets disposition is presented; b) general plan view of the ARSW case of study, where horizontal bracing elements are placed only in correspondence of the vertical bracings; c) plan view of the 6th and 14th storeys (at height 7.313 and 17.313 metres, respectively) where the planar bracings extend to the full length of the racks.

Two approaches (elastic and dissipative) were adopted for design of the structure. In both cases, the design was executed performing linear dynamic analyses with unitary response spectrum. The three-dimensional models of the structure were realized using SAP2000® software. All the steel members (uprights, beams, braces) were modelled as mono-dimensional frame elements. Connections were designed and modelled as pinned joints, except for the upright base connection, fixed in both directions. In particular, regarding the design phase of the dissipative approach, the capacity design philosophy was followed to develop a global ductile collapse mechanism through the localization of plastic deformations in correspondence of braces in tension. A behaviour factor equal to 2.0 was adopted for the definition of the design response spectrum, in agreement with the lower value suggested by Eurocode 8 (EN 1998) for such structural typologies. In the final dimensioning of the structure, it was not possible to apply all the restrictions corresponding to X-shaped braces in terms of maximum over-strength factor variation[6] and non-dimensional slenderness ratio $\bar{\lambda}$[7] (that are mandatory to be respected), neither applying a great variation in section, thickness and steel grade of bracing profiles (Table 6-2 and Figure 6-2). This impossibility is due to the reduced inter-storey height and the high number of storeys, that make ARSW structural typology very different from the common steel buildings. This consideration actually highlights the difficulty to apply Eurocodes rules to the design of ARSWs, that are characterized by structural features that make them very different from common buildings.

The comparison between the structural solutions achieved with elastic and dissipative approaches highlight positive and negative aspects, mainly concerning the total weight of the structures (i.e. material cost) and the typology of joints and connections (i.e. manpower and construction costs). The weight of each transversal frame was equal to 10,47 tons and 9,28 tons in the case of, respectively, elastic and dissipative approaches. The resulting base shear forces acting at Ultimate Limit State (ULS) were respectively equal to 435 kN and 197 kN for elastic and dissipative approaches, because of the different seismic design approach. Since no specific capacity design rules are prescribed for structural details within elastic design, connections are easier in this case, especially if compared to structural details required for the dissipative solution: all the differences in sections and materials that were adopted for the dissipative design (Table 6-2 and Figure 6-2) resulted in a wide range of construction details and to the consequent increase of construction costs.

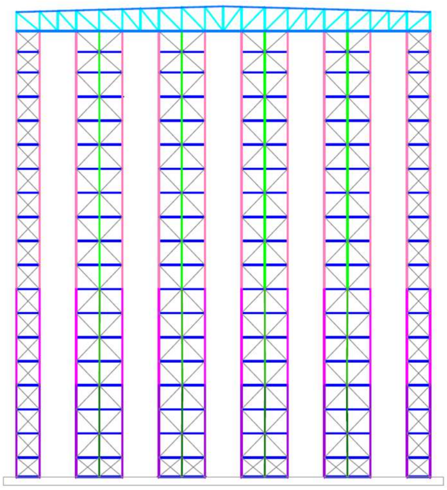
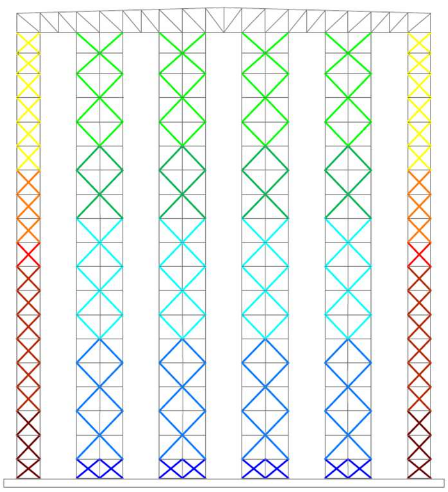
Table 6-1: Main elements and corresponding section profiles.



Colour	Element	Section profile shape
	Uprights	Rectangular Hollowed Section
	Beams	Double Channel section
	Bracing elements	Angle section (*) - EA (Class 3) Hollowed Sections - DA
	Longitudinal beams	C-Shaped Cold formed profiles (Class 4)

[7] The non-dimensional slenderness factor $\bar{\lambda}$ is defined as the square root of the ratio between the Eulerian critical stress σ_{crE} and the yield stress σ_y : $\bar{\lambda} = \sqrt{\sigma_{crE}/\sigma_y}$.

Table 6-2: Summary table with cross-sections and materials adopted for the different structural elements; ARSW designed with dissipative approach.

Transversal Section	*Colour*	*Cross-section*	*Material*
	***Roof Truss***		
		Double Angular Section 45x45x4	S355
		Double Angular Section 55x55x4	S355
	Uprights		
		Rectangular Hollowed Section 120x80x4	S355
		Rectangular Hollowed Section 120x80x6	S355
		Rectangular Hollowed Section 120x80x10	S355
		Rectangular Hollowed Section 150x100x4	S355
		Rectangular Hollowed Section 150x100x6	S355
		Rectangular Hollowed Section 150x100x10	S355
	Beams		
		Double Channel Section 80x50x3	S355
	***Bracing elements***		
		Angular Section 30x30x4	S235
		Angular Section 35x35x4	S235
		Angular Section 35x35x4	S275
		Angular Section 40x40x4	S275
		Angular Section 40x40x5	S355
		Rectangular Hollowed Section 30x30x2	S235
		Rectangular Hollowed Section 30x30x2.5	S235
		Rectangular Hollowed Section 30x30x2.5	S275
		Rectangular Hollowed Section 30x30x2.5	S355
		Angular Section 40x40x4	S355

Non Dissipative Zone: $\Delta\Omega > 25\%$

Capacity design check: not satisfied

Dissipative Zone: $\Delta\Omega \leq 25\%$

Capacity design check: satisfied

Non Dissipative Zone: $\Delta\Omega > 25\%$

Capacity design check: not satisfied

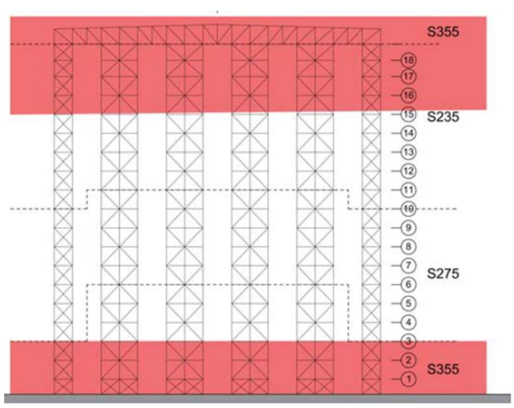


Figure 6-2: Dissipative structure, transversal frame: variation of steel grade and non-dissipative zones (image from Caprili et al (2018)).

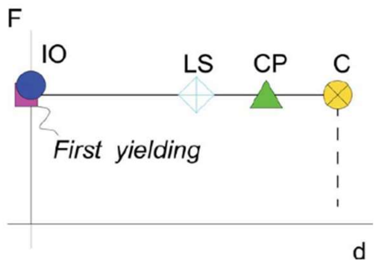
These results were used as a starting point for the structural assessment of the designed ARSWs. This study has two main objectives: (i) give a comparison of the two design approaches in terms of structural performances, looking also at the post-elastic behaviour of the two structures; (ii) evaluate the more suitable and efficient non-linear structural analysis for ARSWs. The methodology that was adopted is organised in the following steps:

1. Execution of non-linear numerical analyses of both structures, using lamped plasticity models;
2. Implementation of the contribution of braces in compression within the non-linear numerical analyses of the dissipative structure only.

In both cases, non-linear static analyses were carried out on bi-dimensional plane frames representing the typical transversal CA section of the case study building. Analyses were performed using SAP2000®.

6.1. Non-linear analyses of structures with only braces in tension

Plastic hinges were located in braces in tension. A zero-compression limit was applied to each brace to neglect its contribution when compressed. An elastic perfectly plastic behaviour characterized plastic hinges. Figure 6-3 shows the adopted force vs displacement relationship; different colours and symbols are associated to the achievement of different Limit States (immediate occupancy ‑ IO, life safety ‑ LS and collapse prevention ‑ CP). The model of the plastic hinge included also the first yielding (represented by a magenta square) and a point beyond CP introduced to solve convergence problems (represented by a yellow crossed circle). The degrading branch, needed by the software, was not relevant for the analysis. The acceptance criteria for each limit state are provided by EN1998-3:2005, being Δ_t the elongation of the considered element (Figure 6-3).



Limit for Immediate Occupancy - IO (blue circle): $0{,}25 \cdot \Delta_t$

Limit for Life Safety - LS (light blue rhombus): $7{,}0 \cdot \Delta_t$

Limit for Collapse Prevention - CP (green triangle): $9{,}0 \cdot \Delta_t$

Figure 6-3: Force vs displacement diagram adopted for plastic hinges in tensile braces (image from Caprili et al (2018)).

Pushover analysis adopted two different load distributions: the 1st group is characterized by incremental horizontal actions proportional to the shear forces at each level of the ARSW, resulting from the design phase; the 2nd group is characterized by incremental horizontal forces proportional to the masses.
Analyses were performed in displacement control; positive force was directed from left to right. Results are presented in terms of capacity curves: the control point was fixed in correspondence of the top storey of the structure. For each dissipative element, the achievement of the different limit states of Figure 6-3 was assessed. In the case of the dissipative ARSW, forces acting on non-dissipative members were checked to confirm the efficiency of the capacity design. Figure 6-5 shows the capacity curve obtained for elastic ARSW using the 1st group distribution and the yielding pattern achieved at collapse limit state. As visible, considering the seismic design base shear (435 kN), the structure was still in the elastic field. The collapse condition was achieved in correspondence of several braces in the bottom part of the building for a displacement equal to about 37 cm and base shear equal to 710 kN.

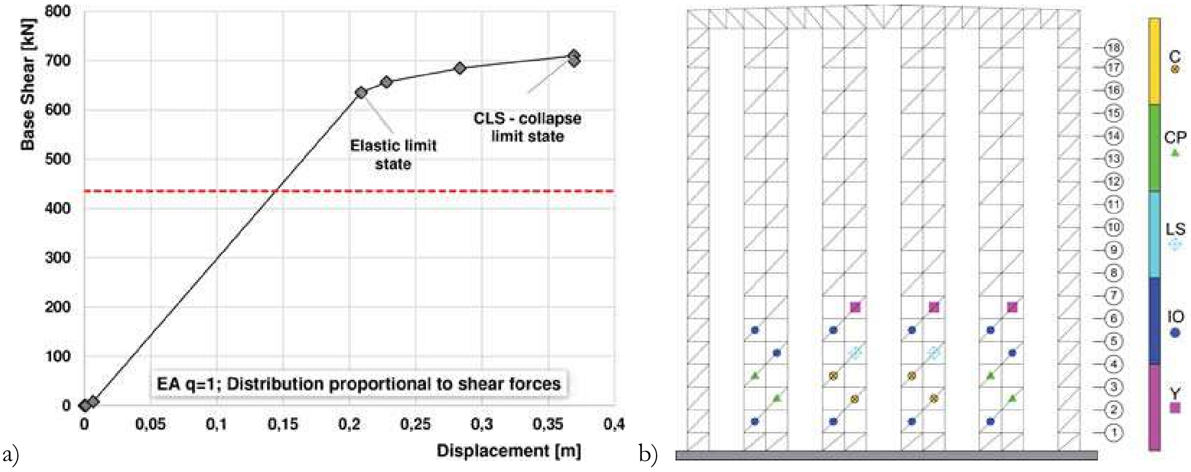


Figure 6-4: a) Capacity curve of ARSW designed for q=1.0 for 1st group distribution; b) yielding pattern and achievement of different limit states in correspondence of the last step of the analysis (about 37 cm of displacement) (image from Caprili et al (2018)).

Figure 6-5 shows the capacity curve obtained for the elastic ARSW using the 2nd group distribution and the yielding pattern achieved at collapse limit state. In this case also, considering the seismic design base shear (435 kN), the structure was still in the elastic field. The collapse condition was achieved in correspondence of several braces in the bottom part of the building for a displacement equal to about 35 cm and base shear equal to 745 kN.

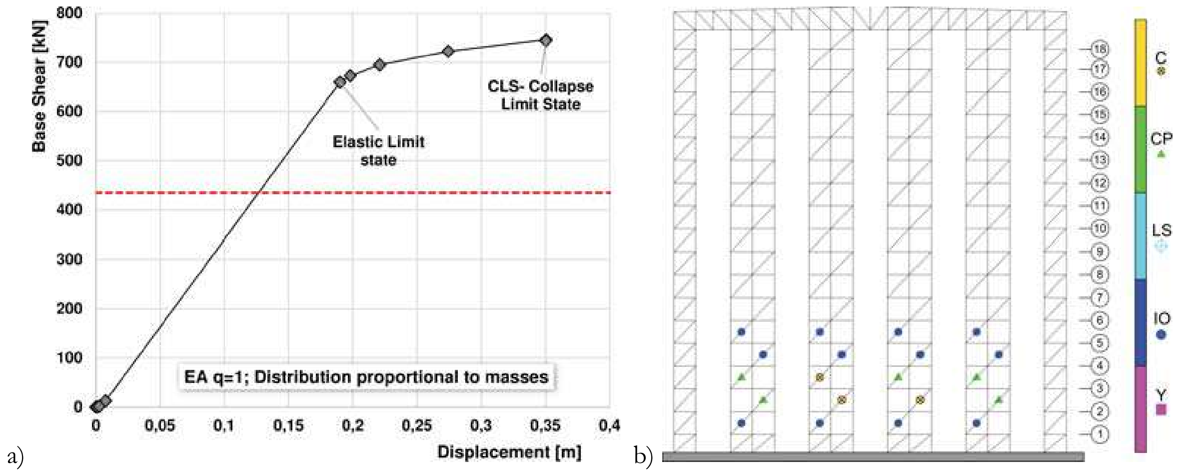


Figure 6-5: a) Capacity curve of ARSW designed for q=1.0 for 2nd group distribution; b) yielding pattern and achievement of different limit states in correspondence of the last step of the analysis (about 35 cm of displacement) (image from Caprili et al (2018)).

Similar considerations are valid in the case of pushover analysis on dissipative ARSW. Figure 6-6 and Figure 6-7 highlight, once again, that for a base shear equal to the design one (about 197 kN) the structure exhibited an elastic behaviour. The collapse condition is achieved for values of displacement and base shear respectively equal to 34 cm and 333 kN for 1st group forces and to 35 cm and 363 kN for 2nd group forces. A wider distribution of plastic hinges can be noted, in correspondence of the central bottom part of the building and, in parallel, of the external uprights.

Comparing the two structures, relevant differences in the yielding pattern developed were observed (from Figure 6-4 to Figure 6-7). In the elastic structure, plastic hinges mainly developed in the central bottom part of the frame, while in the case of the dissipative one, also lateral braces were involved in the developed mechanism. Table 6-3 gathers the values of base shear and displacement at yielding and collapse; in the table, ELS and CLS are acronyms used, respectively, to show the Elastic and the Collapse Limit States.

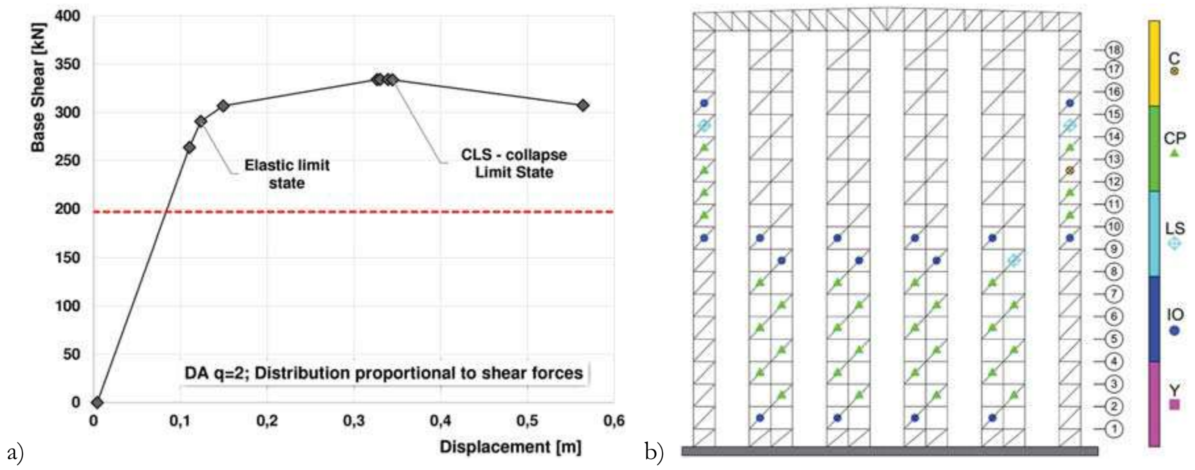


Figure 6-6: a) Capacity curve of ARSW designed for q=2.0 for 1st group distribution; b) yielding pattern and achievement of different limit states in correspondence of the last step of the analysis (about 33 cm of displacement) (image from Caprili et al (2018)).

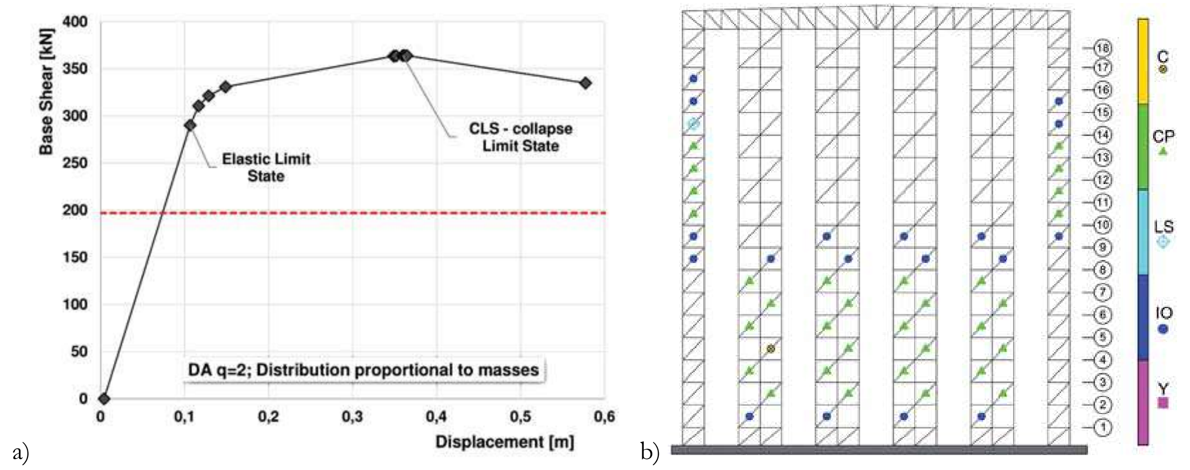


Figure 6-7: a) Capacity curve of ARSW designed for q=2.0 for 1st group distribution; b) yielding pattern and achievement of different limit states in correspondence of the last step of the analysis (about 33 cm of displacement) (image from Caprili et al (2018)).

Table 6-3: Base shear and displacement values corresponding to yielding and collapse for ARSW case study buildings.

Pushover Analysis	Elastic design (q=1) 1st group		Elastic design (q=1) 2nd group		Dissipative design (q=2) 1st group		Dissipative design (q=2) 2nd group	
	ELS	CLS	ELS	CLS	ELS	CLS	ELS	CLS
Base Force [kN]	635	710	660	745	291	333	290	363
Displacement [cm]	21	37	19	3	12	34	11	35

Looking at Figure 6-8, for the same displacement of about 120 mm (*i.e.* value indicating the end of the elastic behaviour of the dissipative ARSW), the elastic structure was in the elastic field, while the first plastic hinges developed in the lower bracing elements of the dissipative structure. It is otherwise to be noted that for the elastic structure, the capacity curve beyond the elastic limit is provided by a mechanical relevance only if the connections among elements are enough over-resistant respect to the bracings. This condition, currently, is not mandatory for non-dissipative structures (EN 1998-1-1). Referring to the yielding patterns, the dissipative approach allowed a major diffusion of plastic hinges: the higher parts of the external lines of racks took part to the mechanism. Although a wider distribution of plastic hinges was reached, a global collapse mechanism was not achieved.

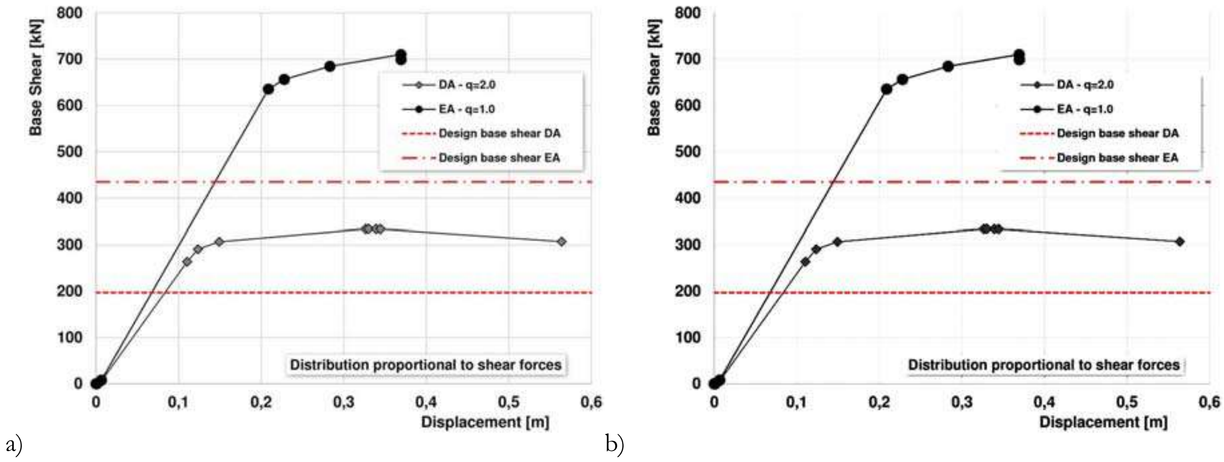


Figure 6-8: Comparison of the capacity curves obtained from the elastic and dissipative structures: a) 1st distribution of horizontal forces; b) 2nd distribution of horizontal forces (image from Caprili et al (2018)).

6.2. Non-linear analyses of the dissipative structure including braces in compression

Further analyses were performed including the contribution of compressed braced for the dissipative ARSW. An asymmetric tension/compression behaviour characterized each plastic hinge: for the tensile force/displacement relationship, the same limitations presented for only-tension braces were adopted (Figure 6-3). The compressive behaviour was calibrated based on preliminary numerical simulation on single brace elements. A simple fibre model was realized using OpenSees® (Mazzoni et al 2017), defining an initial imperfection, equal to 1/500 of the total length of the element, that was applied to model bucking phenomena; a constitutive elastic-perfectly plastic law was adopted for the material. Pushover analysis were performed to simulate the behaviour of compressed members; results in terms of displacement vs base shear are presented for two different sections (Figure 6-9). The equivalent quadri-linear force/displacement law was extracted based on energy equivalence considerations: the continuous curve was obtained from nonlinear analyses on the fibre compressed element, the dashed one represents the approximation to use in the model. The quadri-linear force/displacement law is defined as follows:

- The first branch (O-1) is a parallel to the almost linear part of the curve till the greatest force is reached.
- The second branch (1-2) is the horizontal segment till point 2 is reached (point 2 is the point of the original curve where the maximum force is reached).

- Point 4 is the point of the original curve corresponding to $d_2 = 15 \cdot d_1$, being d_1 the displacement at point 1.
- Point 3 is defined based on the energy equivalence considerations between the two curves.

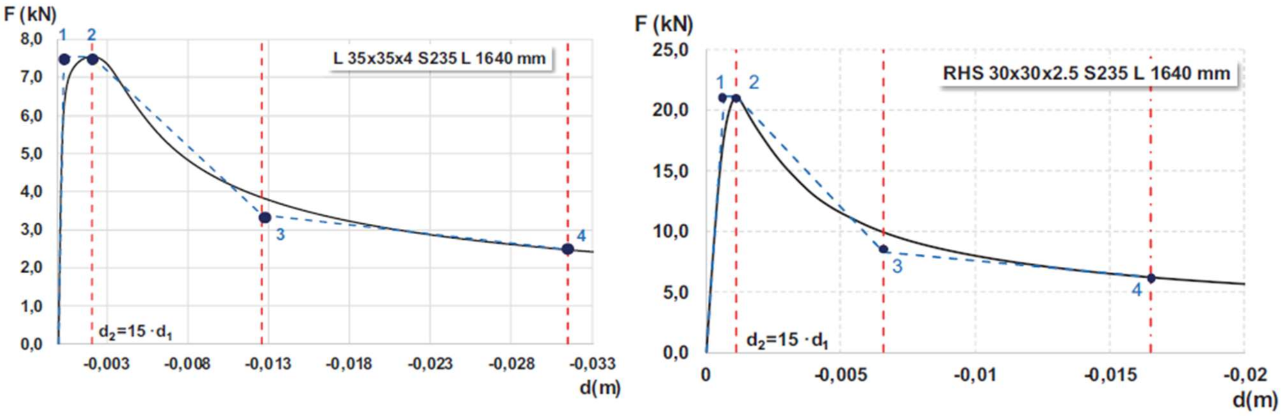
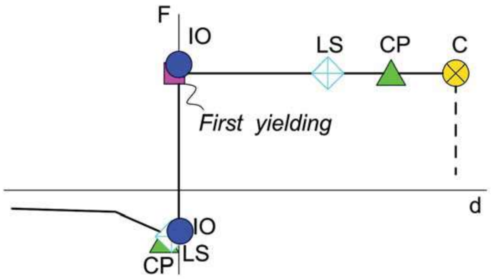


Figure 6-9: Determination of the equivalent quadri-linear relationship for: a) angular L35X35X4 profile, length 1640 mm and steel grade S235; b) rectangular hollow section 30x30x25, length 1640 mm and steel grade S235 (images from Caprili et al (2018)).

Figure 6-10 shows the force/displacement relationship adopted for each plastic hinge. The different colours correspond to the different limit states. The acceptance criteria are provided by Eurocode 8 (EN1998), being Δ_c the shortening of the considered element.



Limit for IO (blue circle): $0{,}25 \cdot \Delta_c$ for profiles of class 1 and 2.

Limit for LS (light blue rhombus): $4{,}0 \cdot \Delta_c$ for profiles of class 1 and $1{,}0 \cdot \Delta_c$ for profiles of class 2.

Limit for CP (green triangle): $6{,}0 \cdot \Delta c$ for profiles of class 1 and $1{,}00 \cdot \Delta_c$ for profiles of class 2.

Figure 6-10: Force vs displacement diagram adopted for each plastic hinge defined for braces (image from Caprili et al (2018)).

The lumped plasticity bi-dimensional model of dissipative structure was developed with SAP2000®; the model included braces in both tension and compression. Pushover analyses were performed using the same forces distribution described in §6.1. Figure 6-11a shows the capacity curve for the 1st distribution of forces: points A, B, C and D refer to the relevant conditions achieved; Table 6-4 gives values of corresponding base shear and displacement. More in details:

- Point A corresponds to a base shear of about 51 kN and a displacement of 1.4 cm. In this condition, several compressed members of the upper part of the external right shelve showed buckling phenomena.
- Point B corresponds to a base shear equal to 258 kN and a displacement of 8.1 cm. In this condition, CP limit state was reached in several compressed braces; buckling was observed in most of the braces, while tensile members did not show any specific structural problem.
- Points C and D correspond to values of shear base of about 360 kN and displacement equal to 12 cm. Most of compressed braces reached CP limit state; strength resistance was achieved in several braces in tension.

Similar results were observed adopting the 2nd forces group Figure 6-11b. In particular:

- Point A corresponds to a base shear force equal to 50 kN and displacement of 1.2 cm. In this condition, several compressed braces of the upper part of the external right shelve underwent buckling phenomena.
- Points B and C correspond to values of the base shear and displacement of about 380 kN and 10-11 cm. CP was achieved in most of the braces in compression, whilst only few tensile braces reached their strength capacity.
- Point D corresponds to a displacement of about 15.5 cm. The collapse condition was reached for most of the tensile braces in the bottom part of the ARSW structure.

Table 6-4: Relevant points achieved in the analysis with corresponding values of base shear forces and displacements.

	1st distribution of forces – Shear Base Forces				2nd distribution of forces – Masses			
	A	B	C	D	A	B	C	D
Base Force [kN]	51	258	354	362	49.5	381	392	384
Displacement [cm]	1.4	8.1	11.6	12.0	1.2	10.8	11.8	15.5

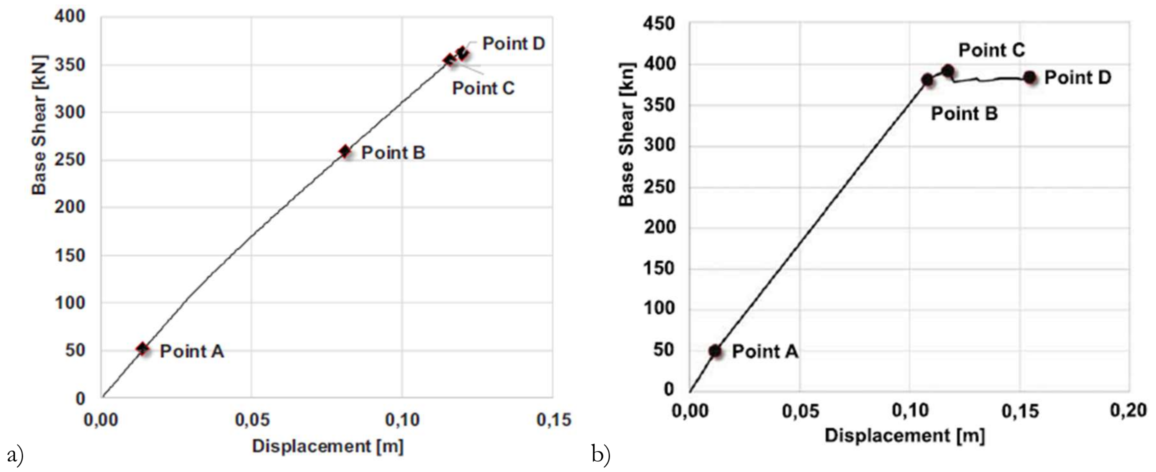


Figure 6-11: Pushover analysis on dissipative structure including members in compression: a) 1st distribution of forces; b) 2nd distribution of forces (images from Caprili et al (2018)).

Figure 6-12 shows the distribution of achieved limit states (i.e. yielding pattern) at the end of the analysis (point D) for the dissipative building and the two distributions of forces. As visible, no global dissipative mechanisms were achieved: ultimate limit states were reached only in correspondence of several elements, while most of them still exhibited ‐ at the end of the analysis ‐ an elastic behaviour. This situation, together with the reduced absolute displacement (lower than 15 cm), was related to the concentration of deformations in correspondence of the bottom part of the dissipative. This is visible from Figure 6-13a, in terms of both absolute displacement and inter-storey drift. High values of inter-storey drift (up to 2.3%) were achieved in correspondence of the 4th-5th storeys of the middle shelves, while in the other areas, lower and differently distributed values were observed.
Several considerations shall be made concerning this situation. One is related to the analysis method: the monitoring of the roof displacement ‐ as commonly used in pushover analyses ‐ is not fully representative in case of ARSW, where the shelves are connected only in correspondence of the roof,

being independent along the whole height. This is evident considering the completely different behaviour of middle and lateral shelves (Figure 6-13b). Besides, the development of dissipative mechanisms takes place mainly in the bottom part of the dissipative structure, where the overstrength variation limit is satisfied (Figure 6-2): this indicates that Eurocode 8 prescriptions are not able, for this particular structural typology, to provide the full exploitation of structural ductile performance as required.

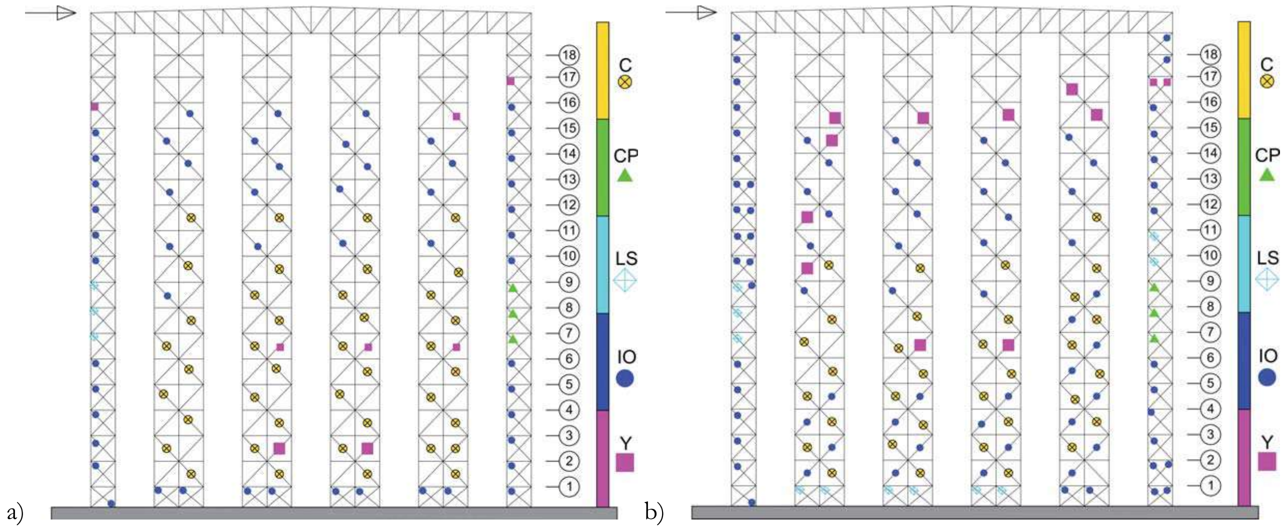


Figure 6-12: Yielding patterns associated to the last step (point D) of analysis on the dissipative building adopting: a) 1st and b) 2nd group distribution of forces (images from Caprili et al (2018)).

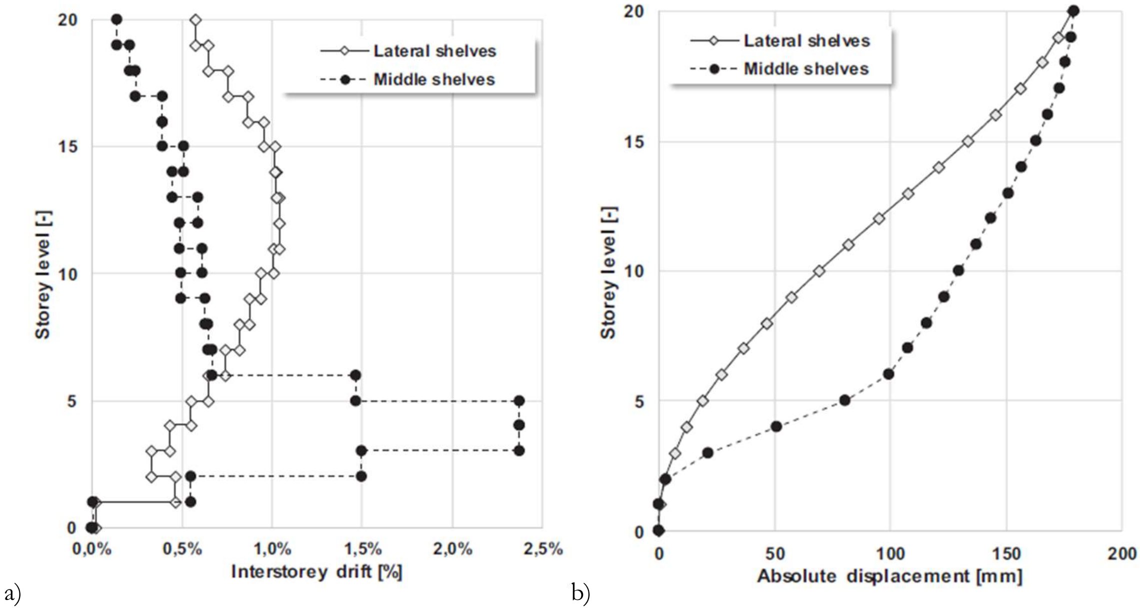


Figure 6-13: a) Absolute displacements and b) inter-storey drifts in correspondence of the different storeys of the ARSW for the middle and the lateral shelves (2nd group of forces) (images from Caprili et al (2018)).

6.3. Conclusive considerations

Automated Rack Supported Warehouses differ from traditional pallet racks, being designed to resist besides self-weight and stored goods, non-structural components and equipment, environmental and seismic actions. EN 16681 (2016) is then not valid for the design and the only possible reference standards are Eurocodes 3 and 8 for steel structures. But ARSW also differ from steel buildings mainly due to their geometry (e.g. high number of storeys of reduced height), pallet load to dead load ratio and structural choices (i.e. cold-formed elements). In this field, the efficacy of Eurocodes design rules when applied to ARSW has been assessed through the application of those rules to design an ARSW case-study building. Both elastic and a dissipative approach were used, and hot-rolled elements were used, contrarily to nowadays common structural choices, but with the aim both to attempt to satisfy all Eurocode 8 (EN 1998) prescriptions for dissipative steel structures and also to evaluate alternative valid solutions. The two strategies were compared in terms of design procedure, construction feasibility, costs and structural performance. In particular, in the dissipative design approach, the satisfaction of the overstrength variation limit along the height imposed by Eurocode 8 (EN 1998) was not always possible: this was mainly due to the geometry of ARSW, being the inter-storey height very limited, and resulted in two non-dissipative portions located in correspondence of the bottom and of the top of the structure (Figure 6-2). The respect of non-dimensional slenderness limits ($1.3 \leq \bar{\lambda} \leq 2.0$) was pursued through a wide variation of profiles cross-sections and steel grade material along the height. If costs related to steel material are lower in the case of using dissipative approach (9.28 tons vs 10.47 tons in the elastic case), connections in this case show higher variability and higher difficulty of realization due to capacity design requirements, increasing construction costs and manpower effort.

Nonlinear static pushover analyses were adopted to assess the structural performance of both structures. Elastic and dissipative structure experienced a non-uniform collapse mechanism involving only the bottom and middle parts of the structures, then disagreeing with the capacity design philosophy (from Figure 6-4 to Figure 6-7). The application of a monotonic increasing load in correspondence to the roof storey led to the concentration of deformation/displacement at the 4th and 5th levels, well represented looking at the inter-storey drift distributions obtained (Figure 6-13). The global ductile behaviour imposed by Eurocode 8 is then not achieved. This situation depends, from one side, on the structural building typology and, on the other side, on the analysis method. Being the shelves connected only at the roof and independent in the bottom and intermediate portions, the representation of the behaviour by the monitoring of a single point is not strictly meaningful. To study with more accuracy the problem, nonlinear dynamic analyses with representative accelerograms should be developed, and may represent the more useful instrument for the structural assessment of such structures.

Results highlight then the need to develop specific design rules for ARSW, since the traditional approach proposed by Eurocodes do not allow to fully exploit the structural performance of such structures. At the same time, the necessity of improving analysis techniques to better understand and exploit the behaviour of ARSW becomes evident.

7. Assessment of structural response of case study structures in static and seismic conditions

The first part of the research deals with the analysis of the current design strategies and structural behaviour of ARSWs through the structural assessment of 5 case studies. These benchmark structures have been designed by five big European companies that nowadays design and produce ARSWs. The final aims of this study are to comprehend the current design strategies, structural choices, and resulting structural behaviour. The analysis of these factors will highlight the positive and questionable aspect of the current practice and will give the basis for the possible strategies for the optimization of the design of these structures. The design of the case studies has been performed within the framework of the STEELWAR research project, where each of the five industrial partners participating designed one multi-depth and one double depth case studies in three different locations that have been chosen to maximize the effects of horizontal actions (wind and seismic): the first one corresponds to a non-seismicity but high windy area, the second one corresponds to a medium seismicity and medium windy area, and finally, the third one consists in the high seismicity area, where wind effects are negligible. In the framework of this thesis, attention is focused on the design in high seismic conditions, and double depth structural typology is taken into consideration.

To reach these aims, the following strategy has been adopted:

1. Analysis and comparison of the design strategies, structural choices (elements, cross-section shapes, type of connections) of the case studies;
2. Structural assessment of the structure through the execution of non-linear dynamic analyses (for this first part of the research, only geometrical non-linearities have been taken into consideration) and safety checks of components (structural element and connections). 3D or 2D models – based on the possible and doable simplification of the structure's geometry – have been adopted to execute the numerical analyses. The seismic input has been defined according to natural accelerograms that have been selected from the available database to obtain the worst damage scenario for the structural typologies considered, in relation to the seismic intensity level considered, that in this case is medium-high. Safety checks have been executed following Eurocodes prescriptions (EN 1993-1-1 and EN 1993-1-3).
3. Comparison of the case studies' structural behaviour through the "hierarchy of criticalities" representation, which consists of representing the chain of mechanisms that may occur in the structure. This chain is obtained by putting in order – from the highest to the lower - the Demand-Capacity (D/C) ratios obtained by executing the safety checks previously carried out.

In the following, more details about the obtained results are given.

7.1. Common and free design input parameters

The five case studies that have been used as benchmark structures in the framework of this thesis are all double depth warehouses. This structural typology is characterized and recognizable from the other main one (the multi-depth one) because two or more aisles parallel to DA direction can be found. The aisles allow the movement of stacker cranes, which are the devices used in this structural typology for the automatic handling of goods. This feature implies that there cannot be any connection between the shelves separated by aisles but the one at the top through the roof truss.

Some input parameters have been fixed to be commonly adopted by all the designers to have comparable structures. These common and equally shared parameters are listed below:

(i) input geometry parameters (i.e. global dimensions as width, length and height of the whole warehouse) and number of defining components (i.e. number of upright lines, number of shelves – coupled uprights - and number of aisles);

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

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

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

Table 7-1 gathers all the shared design input parameters in terms of global dimensions. Besides these overall parameters, the defining geometrical dimensions are given for each direction (CA and DA). For instance, for CA direction, relative distance between each couple upright, distance from consecutive couples of uprights, aisles quantity and width are given. For DA direction, the distance between two successive CA frames is provided, as well as load levels position and minimum clearances between pallet loads and structural elements (uprights, pallet beams). In brief, all the geometrical dimensions and distances are given as common input parameters, as well as the number of uprights, shelves, and aisles. Figure 7-1 gathers the general parameters that have been fixed referring to CA direction.

Table 7-1: Main dimensions for double depth warehouses.

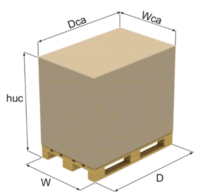
	Double-depth crane
Height (H) x Width (W) x Length (L)	26.2 m × 14.5 m × 96.0 m
Number of shelves along CA direction	8
Warehouse storage capacity	10080 load units

Table 7-2 gathers pallets' characteristics in terms of dimensions and maximum weight: the dimensions are necessary for the definition of the height of each load level and the width of each shelve, while the weight is used both to define gravitational load and participating seismic mass. A maximum of 3 (but at least 2) pallets have to be placed at each load level per each pallet beam couple. The total number of load levels is 14.

Referring to the design force input, in the framework of this thesis, attention has been focused on the design of double depth case studies in high seismicity conditions and the location that has been taken as a reference for the definition of the seismic input is Van. This city is located in Turkey and, according to the Turkish seismic zoning, it belongs to seismic zone 2, being characterized by a Peak Ground Acceleration (PGA) equal to 0.30g for a return period equal to 475 years. Figure 7-2 represent the horizontal acceleration elastic response spectrum defined for Van, and the two corresponding design response spectra assuming a behaviour factor equal to 1.5 or 2. This response spectrum has been defined considering and importance class I, which is associated with warehouses with fully automated storage operations, and a design life equal to 50 years, according to EN16681 (2016).

Table 7-2: Pallet properties.

Type	**Pallet dimensions Width x Depth (mm)**	**Load dimensions Width x Depth x Height (mm)**	**Maximum Weight (kg)**	**Load levels**
1	800x1200	900x1300x800	1000	From 1^{st} to 3^{rd}
2	800x1200	900x1300x1300	800	From 4^{th} to 11^{th}
3	800x1200	900x1300x1800	600	From 12^{th} to 14^{th}



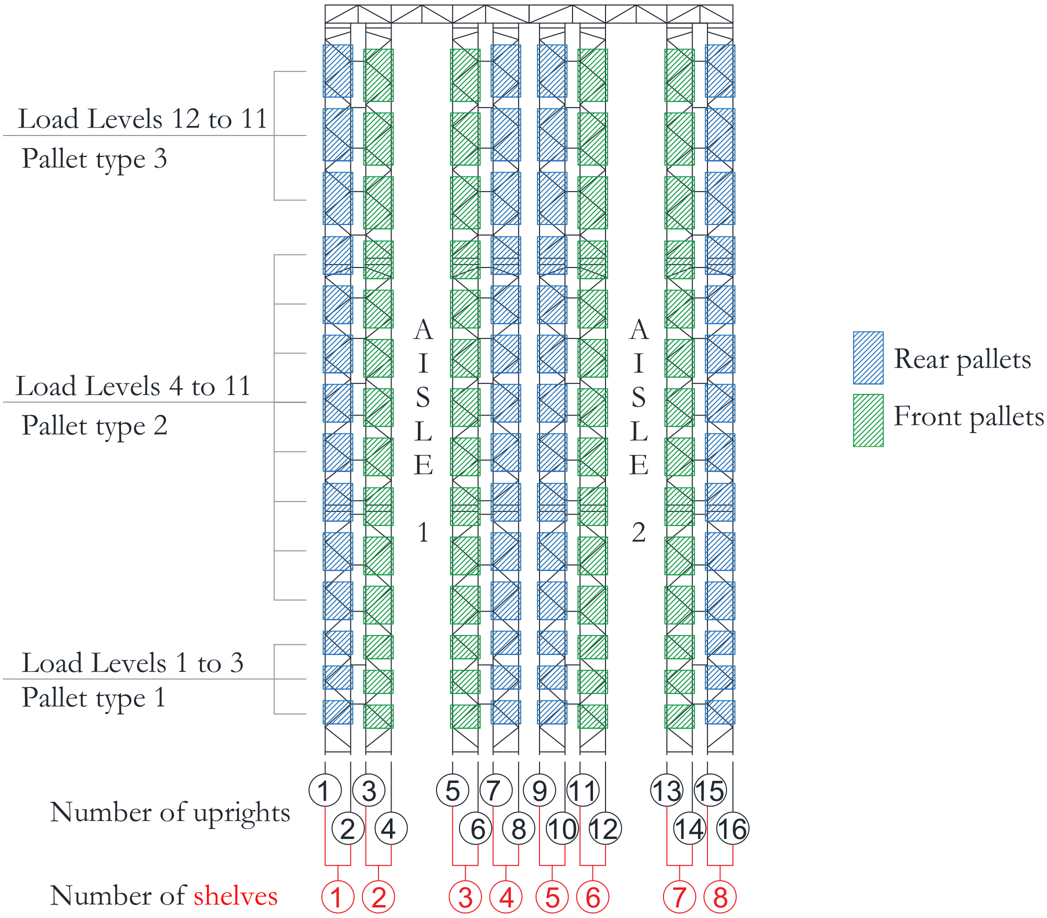


Figure 7-1: Schematic representation of the transversal CA view of the general double depth case study.

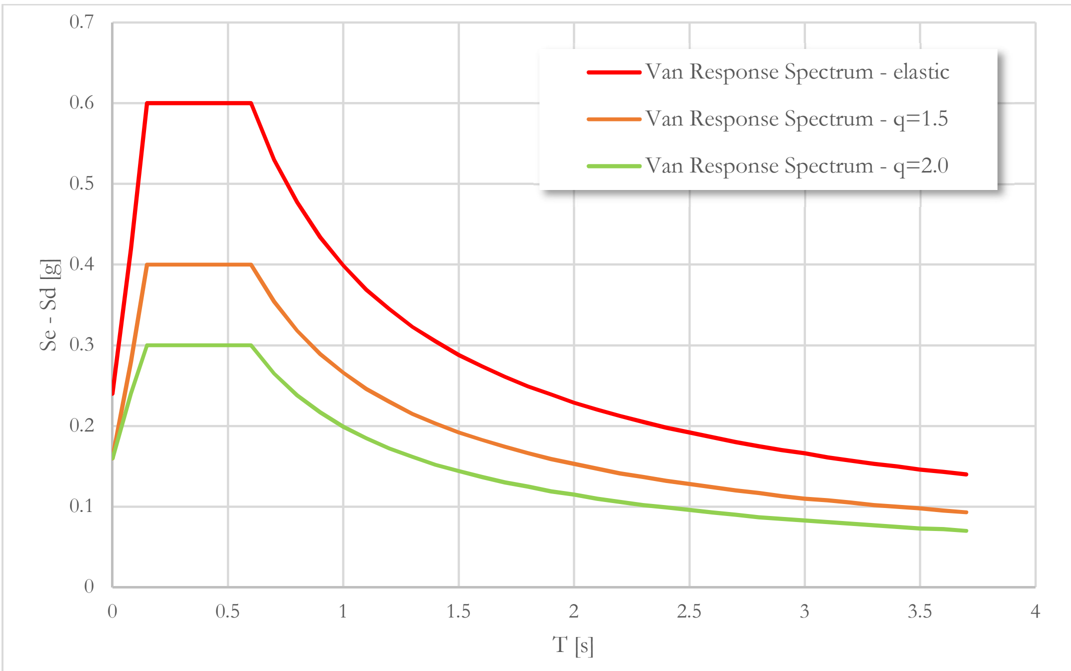


Figure 7-2: Van horizontal acceleration response spectrum (return period 475 years): red line represents the elastic one, while the orange line is the design one, assuming a behaviour factor q equal to 1.5, and the green line represents the horizontal acceleration design response spectrum assuming a q factor equal to 2.0.

Load combination factors have also been fixed for load combinations both for static and seismic design, as well as material factors to calculate design capacity forces. These factors have been defined in agreement with Eurocodes. As regards the design in seismic conditions, the combination of vertical loads to be used for the definition of seismic mass has been defined as follows:

$$G_1 + G_2 + \Psi_2 \cdot (RF) \cdot Q_1 \tag{7-I}$$

where:

- G_1 and G_2 are the masses associated with permanent actions corresponding respectively to the weight of structural and non-structural elements;
- Q_1 is the mass corresponding to the main load, that in this case is represented by pallets;
- Ψ_2 is the combination factor to reduce the characteristic value of the pallets to the corresponding quasi-permanent one. The value of Ψ_2 is equal to 0.8, referring to Van location design conditions and, so, Turkish National Application Documents.
- RF is the Reduction Factor that is applied in warehouses design to take into consideration the probability of pallet full load conditions in case of occurrence of a seismic event. RF may vary depending on structural typologies and the direction of application of seismic action. In particular, for double-depth warehouses, referring to CA direction, RF is taken equal to 1.0, considering that it is more probable to have full load conditions in CA frames. On the other hand, RF is taken equal to 0.8 for DA direction, considering that the probability of having full load conditions is much lower. In fact, the places in the first lines – those closer to the warehouse's front area - are usually more occupied than those closer to the rear area of the warehouse.

This reduces pallet participating mass up to 20% in the CA direction and 46% in the DA direction.
Dealing with seismic load combination, this has been defined as following (in line with EN16681 (2016)):

$$G_1 + G_2 + Q_1 + E \tag{7-II}$$

where E is the effects of seismic action. Concerning loads definitions, full load condition is always considered, so no pallet load reduction is applied.

Besides the fixed design parameters and strategies, several free parameters have been set free to be chosen by the designers to highlight the current trends. For example, among these free parameters, there are: cross-sections of the main structural elements, connections layout, type of numerical analysis, guidelines and prescriptions adopted for the execution of verification checks. In this regard, in the following, the free design parameters are gathered:

(i) Structural types: the structural types of the warehouse can be chosen by the designer of the case study. Different horizontal forces resisting systems can be adopted for CA and DA directions, and different behaviour factors for the two directions can be assumed. The disposition and the layout of the bracings can be freely chosen, too.

(ii) Components characteristics: cross-section shapes, steel grade and connection typology and layout can be freely chosen by the designer according to those that are considered most appropriate to balance structural performances and costs minimization.

(iii) The number of pallets per beam pair: according to what previously said, the number of pallets per each couple of beams can be chosen between a minimum of 2 and a maximum of 3 pallets. So, the total number of pallets, corresponding gravity loads, and participating seismic mass can vary (and, consequently, the stiffness of the structure and so the value of the design seismic force changes) (Figure 7-3).

Finite Element Modelling (FEM) has been used to perform numerical analyses of the structures. 2D or 3D models of the whole system or a portion of it are adopted, depending on possible simplifications accordingly to the structure's geometry. For the seismic design, all the industrial partners have performed the Modal Response Spectrum Analysis (MRSA), and to take into consideration II order effects, two possible strategies have been adopted: a direct one, using as the starting condition of the MRSA analysis a deformed configuration, resulting from the execution of a non-linear P-Delta analysis with the vertical loads applied; an indirect one, by the amplification of the horizontal actions by the factor $1/(1-\theta)$, where θ is the inter-storey drift sensitivity coefficient, in case of having $0.1< \theta<0.2$, as prescribed by Eurocode 8 part 1-2 (prEN 1998-1-2:2019). In any case, only geometrical non-linearities have been taken into considerations, and material behaviour has been defined as linear elastic.

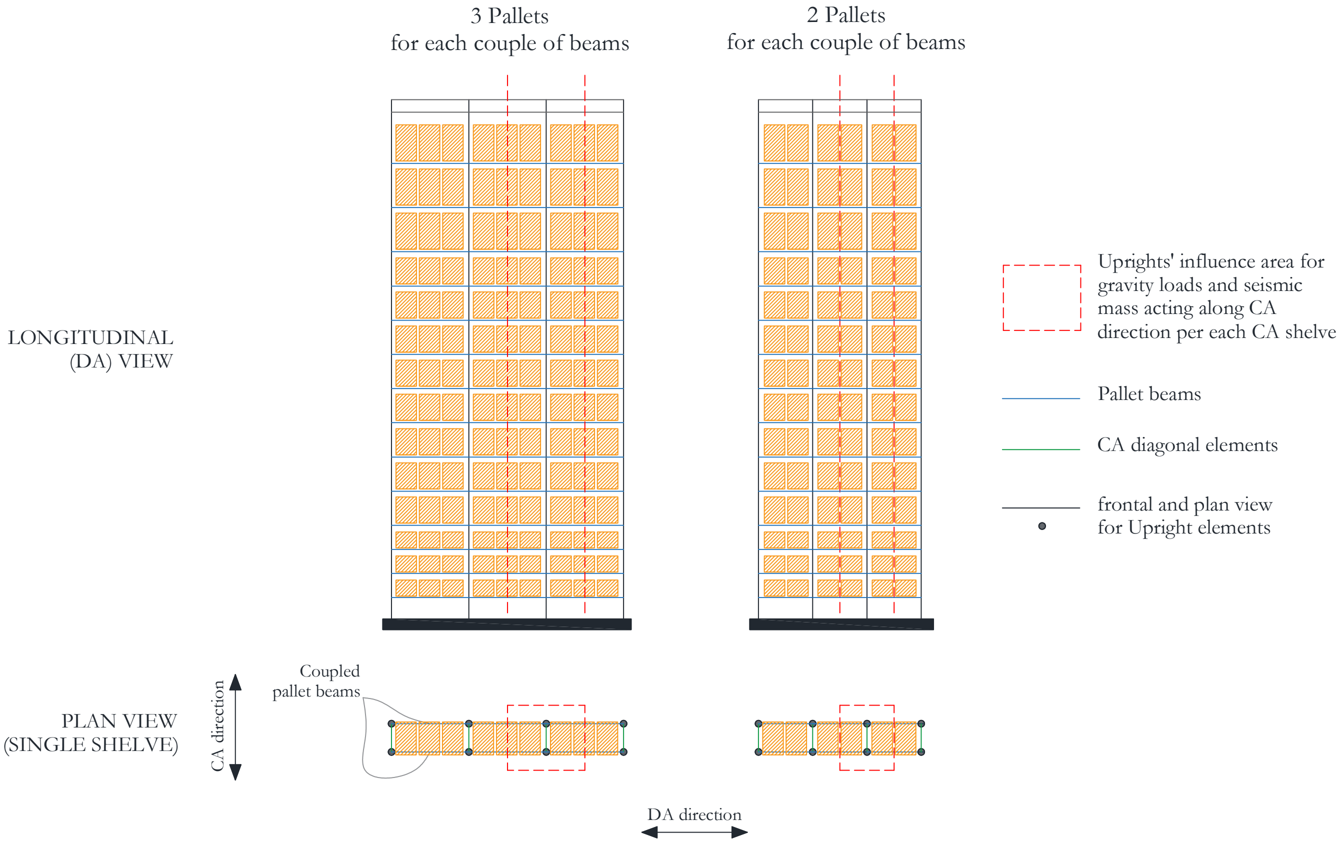


Figure 7-3: Influence areas depending on the number of pallets per each couple of pallets beam considered in the case studies design.

7.2. Configuration, structural choices and design strategies of the case studies

The 5 case studies that have been taken as benchmark structures for the analysis of the current design strategies and resulting structural behaviour of ARSWs, will be individuated from now on as Case Study 1 (CS1), Case Study 2 (CS2), Case Study 3 (CS3), Case Study 4 (CS4), Case Study 5 (CS5). The free design parameters adopted for each case study's design are highlighted in the following. In particular, for each case study, details are given concerning:

- Structural typologies adopted;
- Cross-sections and steel grades for each main structural element;
- Number of pallets for each couple of beams;
- Definition of participating seismic mass;
- Other relevant design parameters taken into consideration by the designer.

7.2.1 **Case Study 1**

The structure is about 26.50 metres high, 14.00 m long along CA direction and about 105 m long along DA direction. Along DA direction, there are in total 46 CA frames, among which there are 42 (from 3 to 23 and from 26 to 46, Figure 7-4) that can be considered "standard", and the other 4 (1, 2, 24 and 25, Figure 7-4) that are considered as "not standard". The "not standard" frames are placed in the zones where bracing towers are. Bracing towers are specific zones where braces for DA direction are placed, and in these zones, columns are different in terms of steel preparation process, sections and grade. Dealing with CA frames, each couple of uprights is organized in a truss scheme, where diagonals are designed to work both in tension and compression. Horizontal elements are placed at each intersection between two consecutive diagonals. Along DA direction, bracing towers constitute the horizontal resisting system, where diagonals are arranged in the X-layout and are designed to work only in tension. Each bracing tower is aligned with the uprights: for each line of uprights (16 lines in total, see Figure 7-5), there are 2 bracing towers (Figure 7-4), and there is no eccentricity between the horizontal resisting system and the centre of mass of the shelves.

Structural typology and characteristics of the main components

Each CA frame is constituted by 16 uprights (Figure 7-5). Each couple of uprights is connected by diagonal elements and horizontal profiles. The consecutive shelves (those not divided by the aisles) are connected one to another by horizontal profiles named "spacers". All the "standard" CA frames are constituted by cold formed profiles, and "not standard" CA frames are constituted by the same profiles of the "standard" ones, with the exception of the columns, for which hot-rolled profiles are used. Table 7-3 summarizes the sections of the main structural elements constituting the "standard" CA frames. Basically, two parts can be individuated: the lower and the upper one. The main difference is that, although the shape of the cross-section each element type (diagonals, uprights) is the same, different thicknesses are used, in order to optimize performances-to-costs ratio. Besides, uprights are continuously holed along their length and are reinforced at the bottom of the lower part. This reinforcement is made through the addition of an external C-shaped element that encases the upright profile. This solution allows changing the upright section only once, without introducing any other discontinuity but providing higher resistance where needed.

Dealing with connections, uprights are continuous from the bottom to the top (but at the interface between the lower and the upper part, where the connection provides continuity to the two profiles), while diagonal and horizontal profiles are each one connected to uprights through a single bolt (Figure 7-6).

Along DA direction, each DA frame is constituted by uprights, pallet beams and bracing towers. Uprights are hinged to the foundation and the pallet beams are connected to uprights through semi-rigid connections (Figure 7-7).

The horizontal forces resisting systems (bracing towers) are placed at the beginning and at the middle length of the frame (Figure 7-4). All the elements of the bracing towers - diagonals, horizontal beams and columns - are constituted by hot-rolled profiles. The pallet load zones are placed between bracing tower 1 and bracing tower 2, and beyond bracing tower 2. All the elements belonging to these parts are constituted by cold-formed profiles. Table 7-4 and Table 7-5 summarize the sections of the main structural elements that constitute DA frames. Dealing with the bracing towers, to optimize the design of the structure, two parts can be individuated: the lower and the upper one. The main difference is that, although the shape of the cross-section adopted for the same element is the same, different thicknesses and sometimes different steel strength are used. With regard to the pallet beams, there are different sections for the different load levels (same shape but different dimensions and thicknesses). Finally, dealing with connections, uprights are continuous from the bottom to the top, while pallet beam-to-uprights connections are the hooked type. Besides, dealing with connections in the bracing towers, uprights are continuous from the bottom to the top, while diagonals are connected to the columns and to the beam through a bolted joint (Figure 7-7). The end sections of the horizontal beams are welded to end plates that are connected to the column through a bolted connection. All the uprights' base connections, with the exception of those corresponding to the uprights constituting the bracing towers, are fixed to the concrete slab with 4 post-installed bonded anchors. Due to the high uplift, it is not possible to use post-installed anchors for the base connections of uprights belonging to the bracing towers. An alternative solution with 4 threaded bars is indeed suggested.

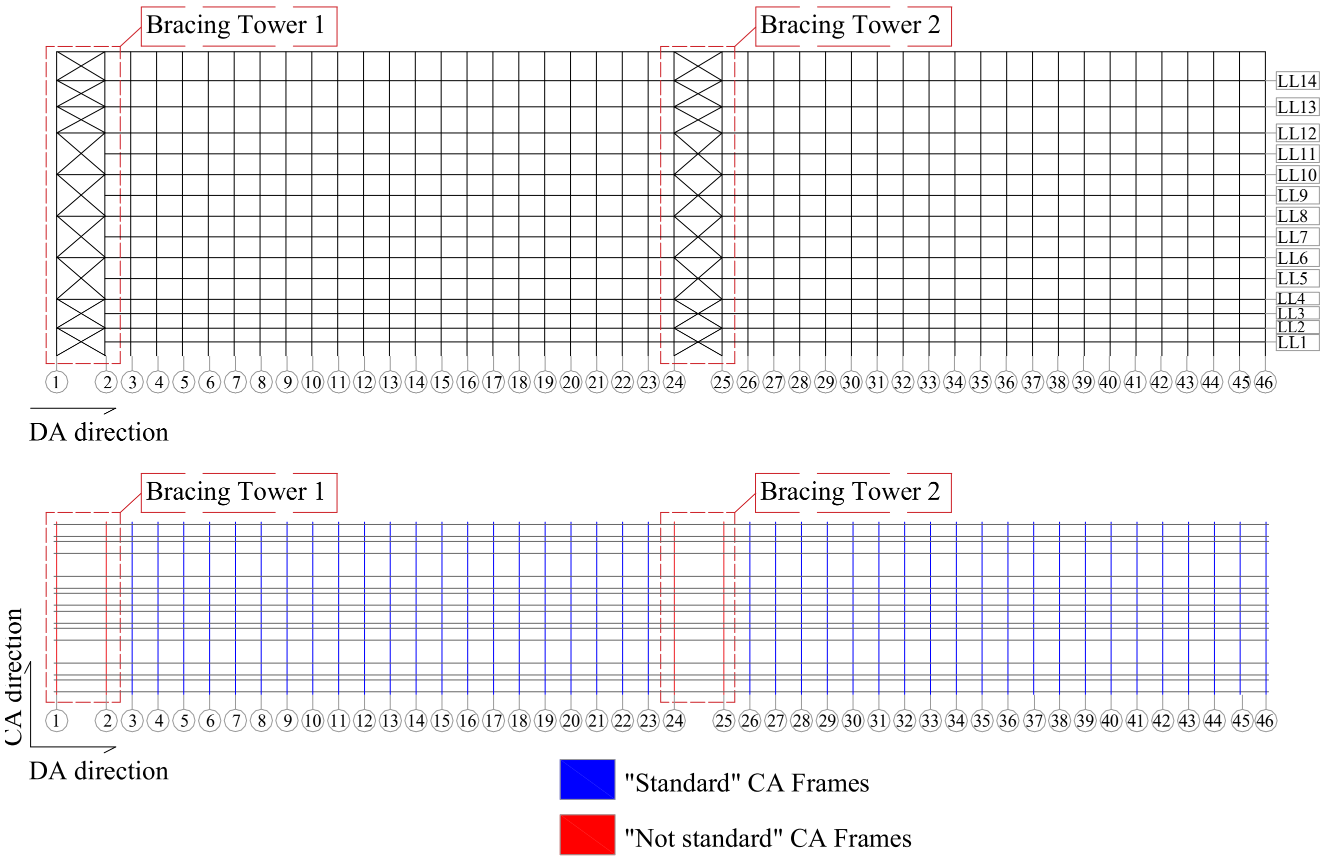


Figure 7-4: Longitudinal (DA) and plan view of CS1.

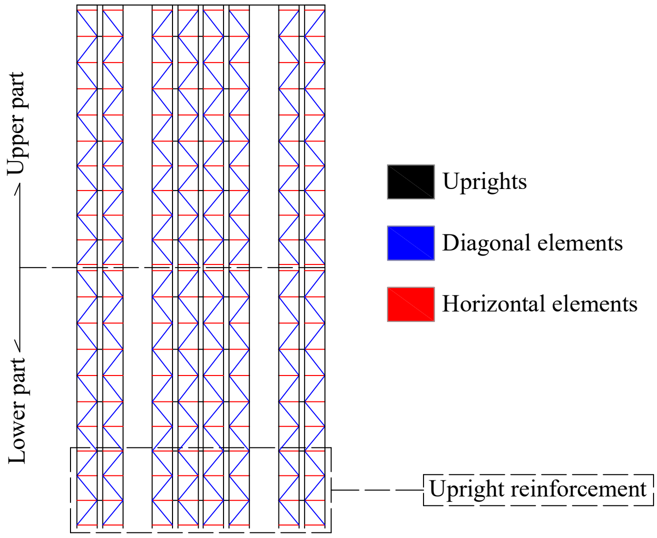
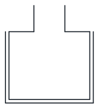
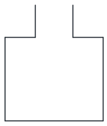
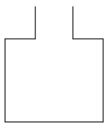


Figure 7-5: Transversal (CA) view of CS1.

Table 7-3: "Standard" CA frames of CS1: sections of structural elements.

Location	Element	Steel preparation process	Section profile shape	Steel grade
Lower part	Reinforced Uprights	Cold-formed	 120x145x4.0 **Continuously holed**	S350GD
	Uprights	Cold-formed	 120x145x4.0 **Continuously holed**	S350GD
	Diagonal elements	Cold-formed	 64x50x3.0	S350GD
	Horizontal elements	Cold-formed	 64x50x1.5	S350GD
	Spacers	Cold-formed	 U 80x15x1.5	S250GD
Upper part	Uprights	Cold-formed	 120x145x2.5 **Continuously holed**	S350GD
	Diagonal elements	Cold-formed	 64x50x2.0	S350GD
	Horizontal elements	Cold-formed	 64x50x1.5	S350GD
	Spacers	Cold-formed	 U 80x15x1.5	S250GD

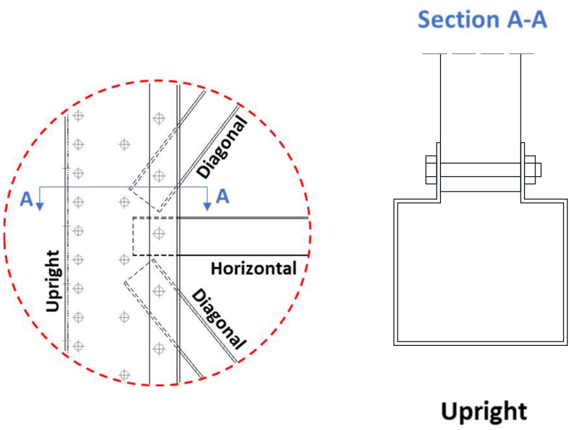


Figure 7-6: Diagonal and horizontal profile – to – upright connection within CS1 (schematic version).

Table 7-4: CS1: DA frames sections of structural elements.

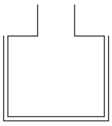
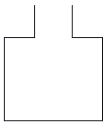
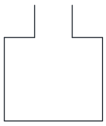
Element	Location	Steel preparation process	Section profile shape	Steel grade
Uprights	Lower part: Reinforced	Cold-formed	 120x145x4.0 **Continuously holed**	S350GD
	Lower part: Not reinforced	Cold-formed	 120x145x4.0 **Continuously holed**	S350GD
	Upper part	Cold-formed	 120x145x2.5 **Continuously holed**	S350GD
Pallet beams	1 to 3	Cold-formed	 110x50x2.0	S355JR
	4 to 11	Cold-formed	 100x50x2.0	S355JR
	4 to 11 "special sections" (used in a few parts only, when necessary)	Cold-formed	 100x50x2.5	S355JR
	12 to 14	Cold-formed	 90x50x2.0	S355JR

Table 7-5: CS1 bracing towers: sections of structural elements.

Location	Element	Steel preparation process	Section profile shape	Steel grade
Lower part	Uprights	Hot-rolled	HE140B	S355JR
	Braces (diagonal elements)	Hot-rolled	L 70x70x8.0	S355JR
	Horizontal profiles	Hot-rolled	HE140A	S275JR
Upper part	Uprights	Hot-rolled	HE140A	S355JR
	Diagonal elements	Hot-rolled	L 70x70x6.0	S355JR
	Horizontal elements	Hot-rolled	HE120A	S275JR

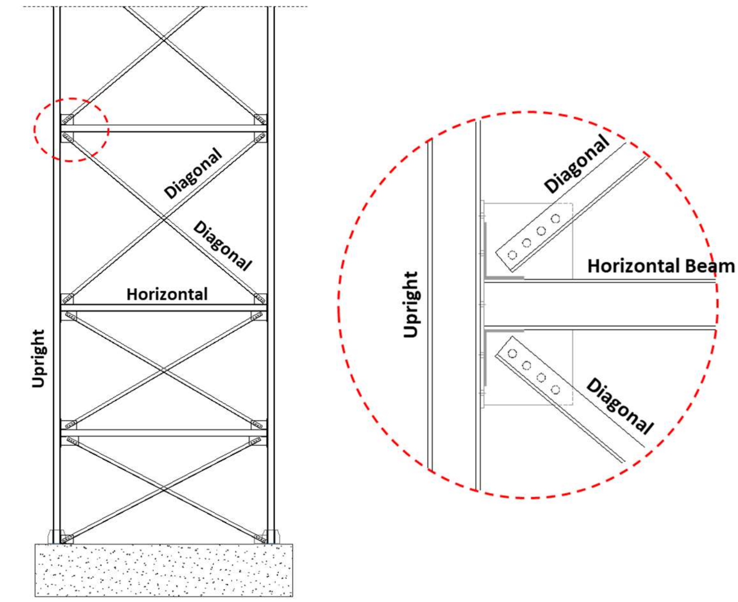


Figure 7-7: DA connections for CS1: bracing tower, diagonal and horizontal – to – upright connection

Design parameters

Table 7-6 gathers the definition of the free design parameters within CS1. In particular, it can be noticed that only 2 pallets for each couple of beams have been considered; besides, the definition of the participating seismic mass is made considering only the reduction factor RF and not the Ψ_2 factor, differently from what has been suggested within the common input design parameters (§ 7.1, equation (7-I)).

The structure has been modelled as low dissipative in both directions. In particular, a behaviour factor q equal to 1.5 ha been used both for CA and DA direction (respectively characterized by a truss scheme and a X-braces structural typologies). The design response spectrum has been defined by reducing the elastic one through the behaviour factor and the additional factor K_d that is defined within EN 16681 (2016) as:

$$K_d = 1 - \frac{P_{E,prod}}{P_E} \cdot (1 - E_{D1} \cdot E_{D3}) \tag{7-III}$$

where P_E is the total weight of the rack in the seismic design situation, including dead weight, permanent weight, and the stored product weight; $P_{E,prod}$ is the weight of the stored products; E_{D1} is depend on the intensity of seismic actions, the number of load levels, and the maximum horizontal force that can be transmitted by the unit loads to the beams (basically, this coefficient takes into consideration the capability of pallets to dissipate energy through their movement on the beams, when friction is overcome); E_{D3} is equal to 0.8, and is a introduced to account for the dissipative phenomena typical of the dynamic behaviour of racking structures under seismic actions that are not included in the mathematical formulation presented within EN16681 (2016), but that are observed on racks that have suffered earthquakes, and from tests performed on shaking tables (the value of 0.8 corresponds to a conventional viscous damping ratio equal to 10% to be applied to the loaded rack, and fits the dissipative mechanism that involves the whole system). The K_d factor is equal to 0.8 in CS1 and it has been applied as an additional reduction factor for the determination of the design response spectrum for both CA and DA direction.

Finally, along CA direction, within this case study has been applied a reduction of stiffness due to the eccentricity of the connection of diagonal element with respect to the centre line of uprights (Figure 7-6), as suggested by EN15512 and EN16681 (2009 and 2016). The size of the reduction of this lateral stiffness has been determined by the execution of shear experimental tests on the shelves constituting the structure. The procedure for the execution of this test is described within EN15512 (2009). The reduction of stiffness has been modelled by the designer within the numerical model trough equivalent axial springs placed at the connection between the diagonal elements and the uprights.

A unique 3D model has been used by the industrial partner for the static and seismic design.

Table 7-6: Definition of free design parameters within CS1.

Horizontal resisting systems	CA direction: truss scheme.
	DA direction: X-shaped braces.
Number of pallets for each couple of beams	2.
Mass definition	CA direction: the participating mass has been calculated as $G_1 + G_2 + Q_1$ DA direction: the participating mass has been calculated as $G_1 + G_2 + 0.8 \cdot Q_1$
Definition of design response spectrum	Q factor = 1.5 for both CA and DA direction.
	K_d =0.8 reduction factor considered.
Reduction of stiffness along CA direction	Considered through the modelling of axial springs in correspondence with the connection between uprights and diagonals. Determination of the size of the reduction according to experimental shear test on the shelve.
Modelling strategy	3D full model.

7.2.2 **Case Study 2**

The structure is about 26.00 metres high, 14.00 m long along CA direction and about 110 m long along DA direction. Along DA direction, there are in total 50 CA frames, among which there are 42 (from 3 to 16, from 19 to 32, and from 35 to 48, Figure 7-8) that can be considered "standard" and the other 8 (1, 2, 17, 18, 33, 34, 49 and 50, Figure 7-8) that are considered as "not standard", since they are placed in the zones where the bracing towers are. The CA frames located in these zones are so called "not-standard". For CA frames, each couple of uprights is organized in a truss scheme, where the diagonals are designed to work both in tension and compression. Along DA direction, bracing towers constitute the horizontal resisting system, where diagonals are arranged in the X-layout and are deigned to work only in tension. Each bracing tower is aligned with uprights: for each line of uprights (16 lines in total, see Figure 7-9), there are 4 bracing towers (Figure 7-8), and there is no eccentricity between the horizontal resisting system and the centre of mass of the shelves.

<u>Structural typology and characteristics of the main components</u>

Each CA frame is constituted by 16 uprights (Figure 7-9). Each couple of uprights is connected by diagonal elements only, that are arranged in a K-shaped layout. There is no connection between the consecutive uprights, but the one at the top through the roof truss. All the CA frames are constituted by cold formed profiles, and "not standard" ones are characterized by having the uprights with different cross-sections in terms of thickness, with respect to those belonging to the "standard" CA frames. Table 7-7 summarizes the sections of the main structural elements of the elements constituting the "standard" CA frames. Basically, two parts can be individuated: the lower and the upper one. The main difference is that, although the shape of the cross-section adopted for the same element type is the same, different thicknesses and sizes are used, in order to optimize performances-to-costs ratio. The uprights are continuously holed along their length. Dealing with connections, uprights are continuous from the bottom to the top (a connection between the lower and the upper upright provides continuity among the two profiles), while diagonal and horizontal profiles are each one connected to uprights through a single bolt.

Along DA direction, each DA frame is constituted by uprights, pallet beams and 4 bracing towers. Uprights are hinged to the foundation and the pallet beams are connected to them through bolted connections. The bracing towers are placed one at the front, another one at the rear, and two along the length of the structure (Figure 7-8). The diagonal elements of the bracing towers are the only hot-rolled ones. The pallet load zones are placed between the 1st and the 2nd, the 2nd and the 3rd, and the 3rd and the 4th bracing tower. All the elements belonging to these parts are constituted by cold-formed profiles.

Table 7-8 and

Table 7-9 summarize the sections of the main structural elements that constitute DA frames. Dealing with the bracing towers, uprights are cold-formed elements, and the same cross-section is kept for the whole height, while for the braces, a smaller section is used in the higher part of the towers. With regard to the pallet beams, the same cross-section is used for all the load levels.

Finally, dealing with connections, the uprights belonging to the pallet load zones are continuous from the bottom to the top, and pallet beam-to-uprights connections are bolted (they are modelled as hinged to the uprights). Besides, dealing with connections in the bracing towers, uprights are continuous from the bottom to the top, while diagonals are connected to the columns through a bolted joint. An additional steel plate is necessary to realize this connection: both diagonals and horizontal beams are bolted to a plate, that is then bolted to the column. All the uprights' base connections are fixed to the concrete slab with post-installed bonded anchors. Bigger anchors are required for the uprights belonging to the bracing towers, due to the higher uplift that arises there when seismic action occurs along DA direction.

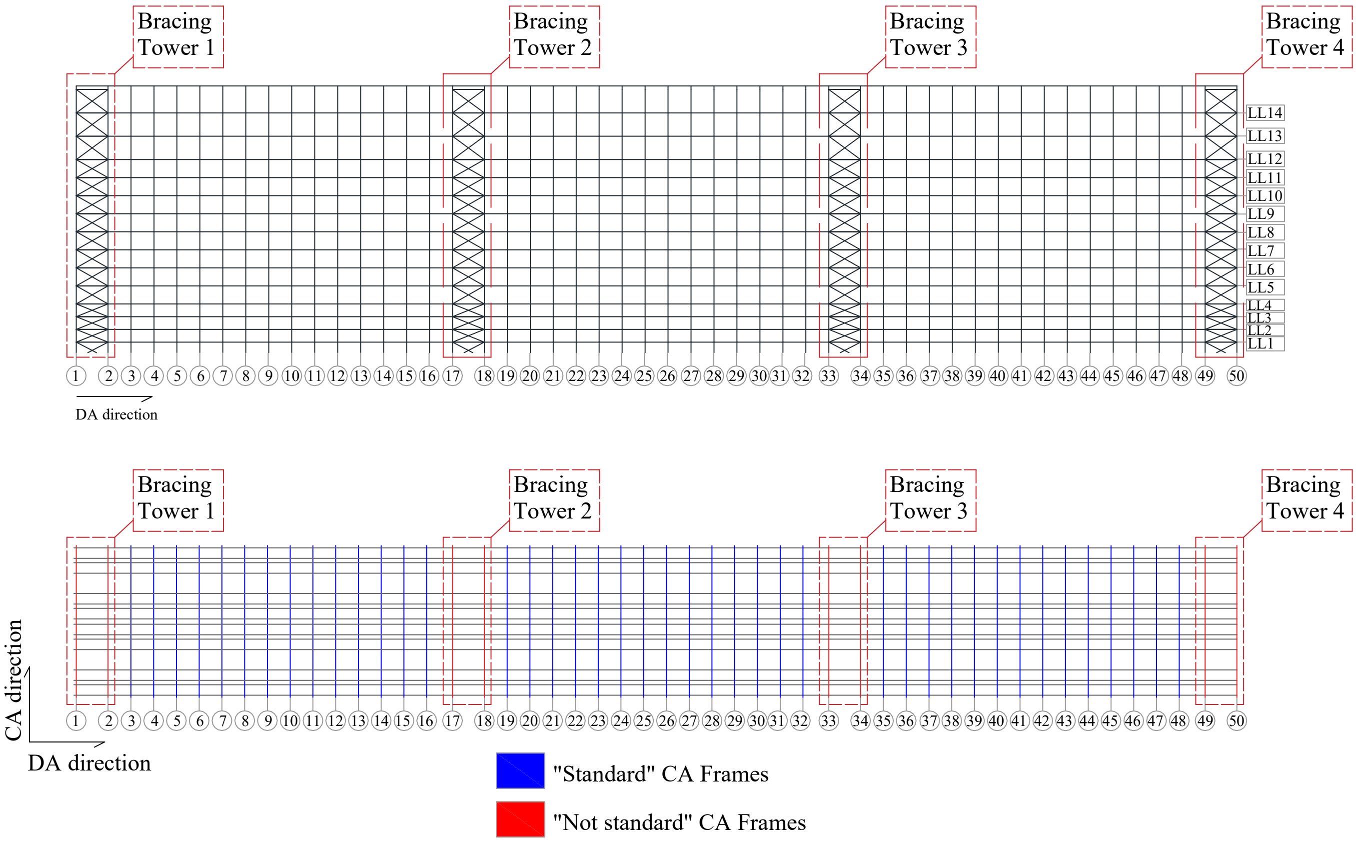


Figure 7-8: Longitudinal (DA) and plan view of CS2.

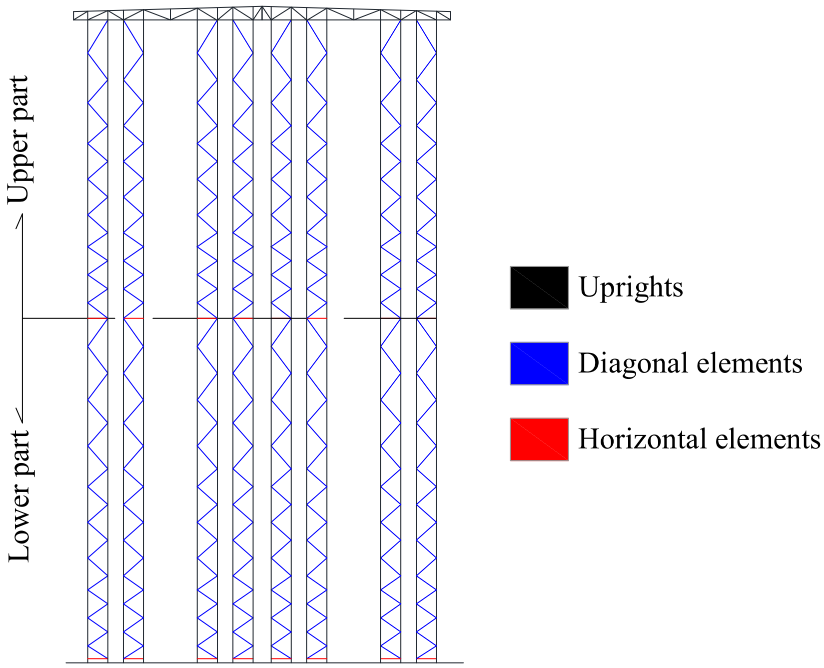


Figure 7-9: Transversal (CA) view of CS2.

Table 7-7: "Standard" CA frames of CS2: sections of structural elements.

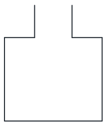
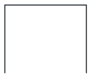
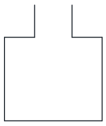
Location	Element	Steel preparation process	Section profile shape		Steel grade
Lower part	Uprights	Cold-formed		140x140x3.5 **Continuously holed**	S355MC
	Diagonal elements	Cold-formed		49x70x2.5	S350GD
Upper part	Uprights	Cold-formed		83x140x2.5 **Continuously holed**	S355MC
	Diagonal elements	Cold-formed		49x90x2.0	S350GD

Table 7-8: CS2: DA frames sections of structural elements.

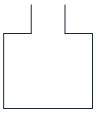
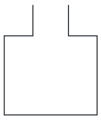
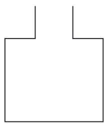
Element	Location	Steel preparation process	Section profile shape		Steel grade
Uprights (not belonging to bracing towers)	Lower part	Cold-formed		140x140x3.5 **Continuously holed**	S355MC
	Upper part	Cold-formed		83x140x2.5 **Continuously holed**	S355MC
Pallet beams	All levels	Cold-formed		50x150x2.5	S350GD

Table 7-9: CS2 bracing towers: sections of structural elements.

Element	Location	Steel preparation process	Section profile shape		Steel grade
Upright	All height	Cold-formed		140x140x3.5 **Continuously holed**	S420 MC
Diagonal elements (braces)	Lower part	Hot-rolled	L 80x80x8		S355 JR
	Upper part	Hot-rolled	L 70x70x6		S355 JR
Horizontal elements	All height	Cold-formed	SHS 90x4		S275 M

Design parameters

Table 7-10 gathers the definition of the free design parameters within CS2. In particular, it can be noticed that only 2 pallets for each couple of beams have been considered; besides, the definition of the participating seismic mass is made considering only the reduction factor RF and not the Ψ_2 factor, differently from what has been suggested within the common input design parameters (§ 7.1, equation (7-I)).

The structure has been modelled as low dissipative in both directions. In particular, a behaviour factor q equal to 1.5 ha been used for CA direction (characterized by a K-shaped layout for diagonals), and a q factor equal to 2.0 for DA direction (characterized by X-braces structural typology). The design response spectrum has been defined by reducing the elastic one by the behaviour factor and the additional factor K_d (EN 16681 (2016), equation (7-III)), that in this case study is equal to 0.8.

Finally, along CA direction, within this case study a reduction of stiffness due to the eccentricity of the connection of diagonal element with respect to the centre line of uprights has been applied, as suggested by EN15512 and EN16681 (2009 and 2016). The size of the reduction of this lateral stiffness has been determined by the execution of shear experimental tests on the shelves constituting the structure. The procedure for the execution of this test is described within EN15512 (2009). The reduction of stiffness has been modelled by the designer within the numerical model trough equivalent axial springs placed at the connection between the diagonal elements and the uprights. A unique 3D model has been used by the industrial partner for the static and seismic design.

Table 7-10: Definition of free design parameters within CS2.

Horizontal resisting systems	CA direction: K-shaped layout for diagonals.
	DA direction: X-shaped braces.
Number of pallets for each couple of beams	2.
Mass definition	CA direction: the participating mass has been calculated as $G_1 + G_2 + Q_1$ DA direction: the participating mass has been calculated as $G_1 + G_2 + 0.8\cdot Q_1$
Definition of design response spectrum	Q factor = 1.5 for CA direction. Q factor = 2.0 for DA direction.
	K_d =0.8 reduction factor considered.
Reduction of stiffness along CA direction	Considered through the modelling of axial springs in correspondence with the connection between uprights and diagonals. Determination of the size of the reduction according to experimental shear test on the shelve.
Modelling strategy	3D full model.

7.2.3 Case Study 3

The structure is about 26.50 metres high, 14.00 m long along CA direction and about 102 m long along DA direction. Along DA direction, there are in total 33 CA frames, and there are 4 bracing towers (Figure 7-10). All the elements are cold-formed. Focusing on each CA frame, there are 8 uprights that are coupled through horizontal and diagonal elements, that are organized in a V-shaped layout (Figure 7-11). Diagonals are designed to work both in tension and compression. Along DA direction, bracing towers constitute the horizontal resisting system, where diagonals are arranged in the X-shaped layout and are designed to work only in tension. The bracing towers are placed in an eccentric position with respect to the upright shelves: within each group of bracing towers (numbered from 1 to 4 within Figure 7-10), there are two "external" bracing towers that are placed on the outer side of both the more external uprights, and there is a central bracing tower that is placed in the middle of the frame between the two central shelves (Figure 7-11) . This results in eccentricity between to centre of mass of the external shelves and the respective horizontal resisting system, while no eccentricity can be found between the central shelves and the respective bracing tower (the central one). The connection between the bracing towers and the respective shelves is assured at each load level by both the horizontal bracing system and transversal beams.

Structural typology and characteristics of the main components

Each CA frame is constituted by 8 uprights, that are coupled thought horizontal and diagonal elements arranged in the V-shaped layout (Figure 7-9).
Table 7-11 summarizes the sections of the main structural elements that constitute each CA frame. As usual, in order to optimize performances-to-costs ratio, different thicknesses have been used along the height of the structure, both for the uprights and diagonal profiles. The uprights are not continuously holed along their length. In fact, they are drilled only at the connections with diagonal and horizontal elements and, for the lower uprights, only at the bottom, where reinforcement is placed, that is indeed directly bolted to the column. Dealing with connections, uprights are continuous from the bottom to the top, while diagonal and horizontal profiles are each one directly connected to uprights through a single bolt. Diagonal and horizontal profiles are also mutually connected through bolts at the mid span of the horizontal profile with additional supporting C profiles.
Along DA direction, each CA frame is connected to the following one through pallet beams, that are continuous and connected to the uprights or to the horizontal beams: for each shelve, that is composed of two uprights, and for each load level (that is placed at the same height of the horizontal beam), there are 4 pallet beams: the two outer ones are aligned with the uprights and are bolted to them, while the central ones are supported by the horizontal beam. The bracing towers are connected to the respective shelves through horizontal braces and additional angular elements (Figure 7-10).
Table 7-12 gathers the sections of the main structural elements that constitute the bracing towers and the pallet beams: with regard to the bracing towers, the cross-section shape of the elements is kept the same but different thicknesses are used for the central and the external ones; with respect to the pallet beams, the same cross-section is used for all the load levels. Dealing with connections in the bracing towers, uprights are continuous from the bottom to the top, while diagonals are connected to the columns through a bolted joint. An additional steel plate is necessary to realize this connection: both diagonals and horizontal beams are bolted to the plate, that is then bolted to the column. All the uprights' base connections are fixed to the concrete slab with threaded bars, with the only exception of the foundation of the bracing towers, that is constituted by a rigid beam to which the uprights belonging to the towers are connected, and the beam is then connected to the concrete slab through threaded bars. More resistant and stiffer connections are required for bracing towers, due to the higher uplift that arises there when seismic action occurs along DA direction.

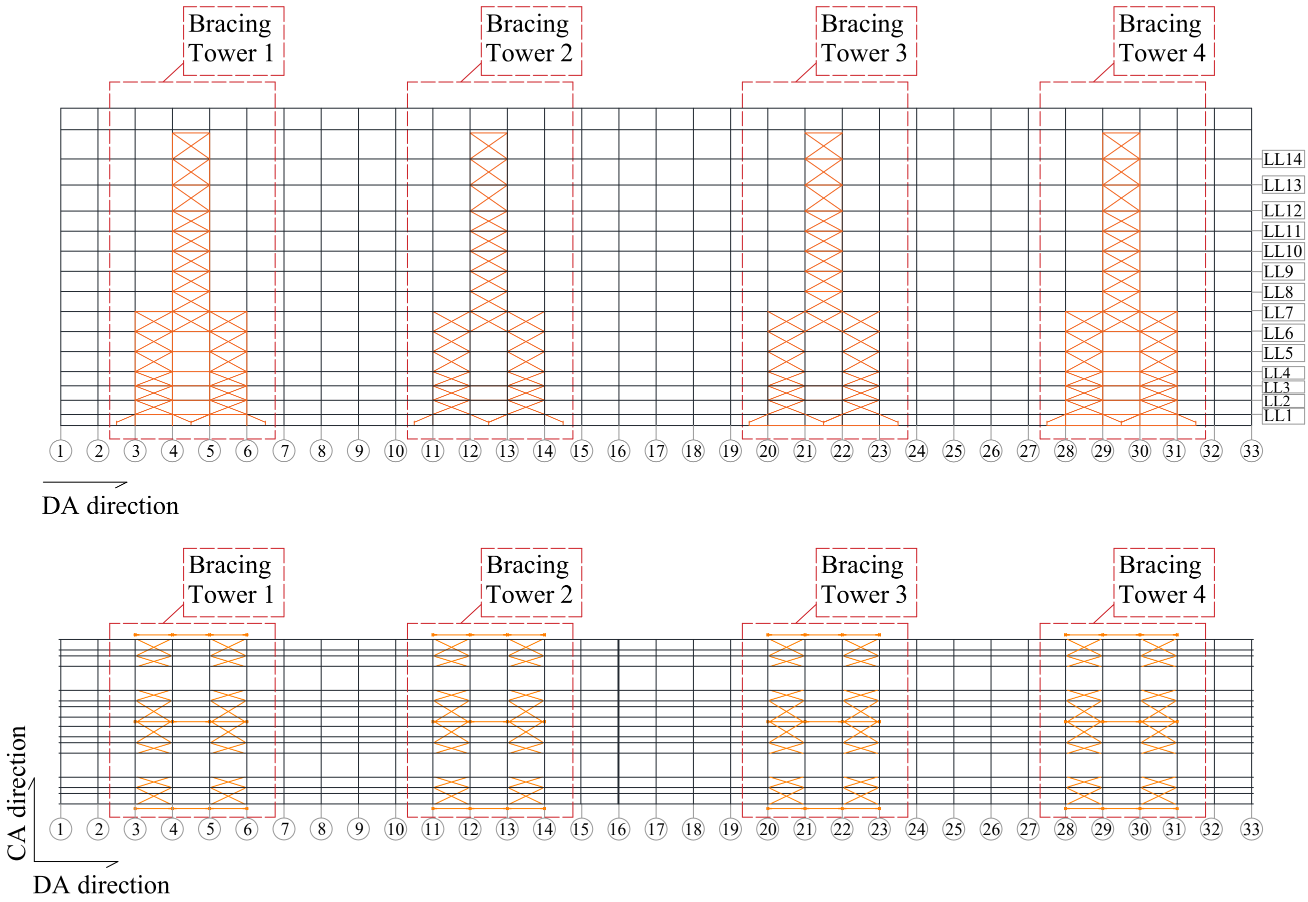


Figure 7-10: Longitudinal (DA) and plan view of CS3.

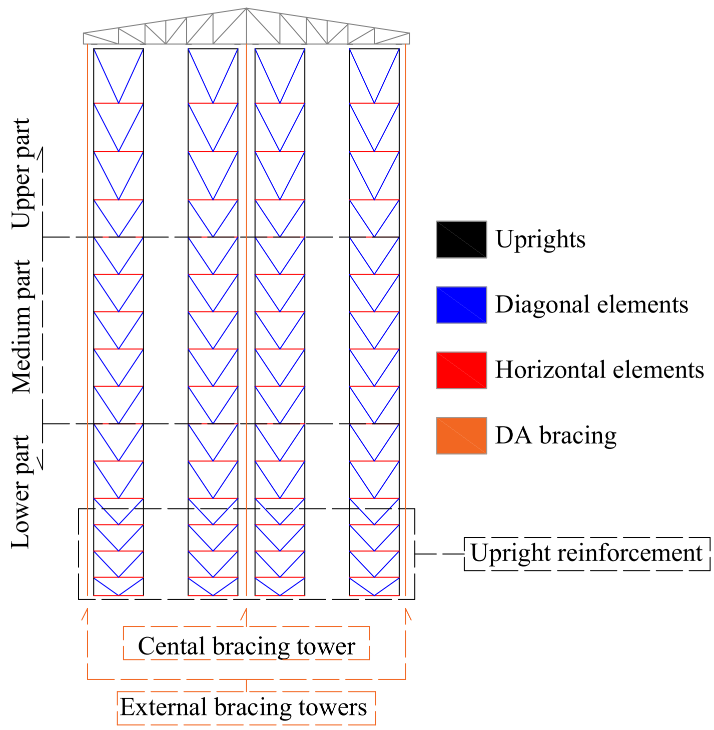


Figure 7-11: Transversal (CA) view of CS3.

Table 7-11: CA frames of CS3: sections of structural elements.

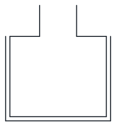
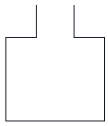
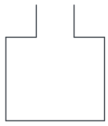
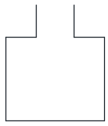
Element	Location	Steel preparation process	Section profile shape	Steel grade
Uprights	Lower part	Cold-formed	 150x175x4 + 100x142x4 (Reinforced) **Continuously Holed**	S420MC
		Cold-formed	 150x175x4 **Not Holed**	S420MC
	Medium part	Cold-formed	 150x175x3 **Not Holed**	S420MC
	Upper part	Cold-formed	 150x175x2 **Not Holed**	S420MC
Diagonal elements	Lower and medium parts	Cold-formed	 70x70x20x4.0	S350GD
	Upper part	Cold-formed	 70x70x20x3.0	S350GD

Table 7-12: CS3 bracing towers and load levels: sections of structural elements.

Element	Location	Steel preparation process	Section profile shape	Steel grade
Upright	Both external and central bracing tower	Cold-formed	Lower part: 2x C150x80x20x5.0 **Not Holed** Upper part: 2x C150x80x20x3.0 **Not Holed**	S350GD
Diagonal elements (braces)	External bracing tower	Cold-formed	Lower part: 2x C120x140x3.0 Upper part: 2x C120x140x2.0	S350GD
	Central bracing tower	Cold-formed	Lower part: 2x C120x140x3.0 and 4.0 Upper part: 2x C120x140x2.0	S350GD
Horizontal elements	External bracing tower	Cold-formed	Whole height: 2x C120x60x20x3.0	S350GD
	Central bracing tower	Cold-formed	Whole height: 2x C120x60x20x3.0 and 4.0	S350GD
Pallet beams	All load levels	Cold-formed	2x C100x150x3.0	S350GD

<u>Design parameters</u>

Table 7-13Table 7-10 gathers the definition of the free design parameters within CS3. In particular, it can be noticed that 3 pallets for each couple of beams have been considered; besides, the definition of the participating seismic mass is made considering both the reduction factor RF and the Ψ_2 factor, according to what has been suggested within the common input design parameters (§ 7.1). The structure has been modelled as low dissipative in both directions. In particular, a behaviour factor q equal to 1.5 ha been used for CA direction (characterized by a V-shaped layout for diagonals), and a q factor equal to 2.0 for DA direction (characterized by X-braces structural typology). The design response spectrum has been defined by reducing the elastic one by the behaviour factor and the additional factor K_d (EN 16681 (2016), equation (7-III)), that in this case study is equal to 0.8. Finally, along CA direction, no reduction of stiffness has been applied to the CA frames.

Table 7-13: Definition of free design parameters within CS3.

Horizontal resisting systems	CA direction: V-shaped layout for diagonals.
	DA direction: X-shaped braces.
Number of pallets for each couple of beams	3.
Mass definition	CA direction: the participating mass has been calculated as $G_1 + G_2 + 0.8 \cdot Q_1$ DA direction: the participating mass has been calculated as $G_1 + G_2 + 0.8 \cdot 0.8 \cdot Q_1$
Definition of design response spectrum	Q factor = 1.5 for CA direction. Q factor = 2.0 for DA direction.
	K_d =0.8 reduction factor considered.
Reduction of stiffness along CA direction	Not applied.
Modelling strategy	2D models for both CA and DA direction.

7.2.4 Case Study 4

The structure is about 26.50 metres high, 14.50 m long along CA direction and about 100 m long along DA direction. Along DA direction, there are in total 33 CA frames, and the horizontal forces resisting system is diffused along the whole length of the structure (Figure 7-12). All the elements are cold-formed. Focusing on each CA frame, there are 16 uprights that are coupled through diagonal elements, that are organized in a K-shaped layout (Figure 7-13). Diagonals are designed to work both in tension and compression. Along DA direction, braces are diffused all along the length of the structure, diagonals are arranged in the X-shaped layout and only work in tension. The horizontal resisting systems are placed in an eccentric position with respect to the upright shelves: there are two "external" bracing systems that are placed on the outer side of both the more external uprights, and there is a central bracing system that is placed in the middle of the frame between the two central shelves (Figure 7-13). This results in eccentricity between to centre of mass of the external shelves and the respective horizontal resisting system.

Structural typology and characteristics of the main components

Each CA frame is constituted by 16 uprights, that are coupled thought diagonal elements arranged in the K-shaped layout (Figure 7-13). Table 7-14 summarizes the sections of the main structural elements that constitute the CA frames. As usual, in order to optimize performances-to-costs ratio, different thicknesses have been used along the height of the structure, both for the uprights and diagonal profiles. Some differences can also be found between the external and the central shelves. The uprights are continuously holed along their length. Dealing with connections, uprights are continuous from the bottom to the top, while diagonal profiles are directly connected to uprights through a single bolt. Along DA direction, each CA frame is connected to the following one through pallet beams, that are connected to the uprights through semi-rigid hooked connections. All the bracing systems are connected to the respective shelves through rigid tube elements (highlighted in red within Figure 7-13). Looking at the external shelves, tubes are placed to connect both the consecutive shelves and the more external upright to the one belonging to the bracing systems. Looking at the central shelves, the tubes connect the uprights of the consecutive shelves, and, at the middle, the diagonal of the central bracings are directly hinged to the tubes (no upright is placed in the plane of the central bracing system). The tubes are fixed to the uprights through bolted connections, coupling the consecutive shelves.

Table 7-15 gathers the sections of the main structural elements that constitute the bracing towers and the pallet beams.

All the uprights' base connections are fixed to the concrete slab with post-installed bonded anchors. Bigger anchors are required for the uprights belonging to the bracing towers, due to the higher uplift that arises there when seismic action occurs along DA direction.

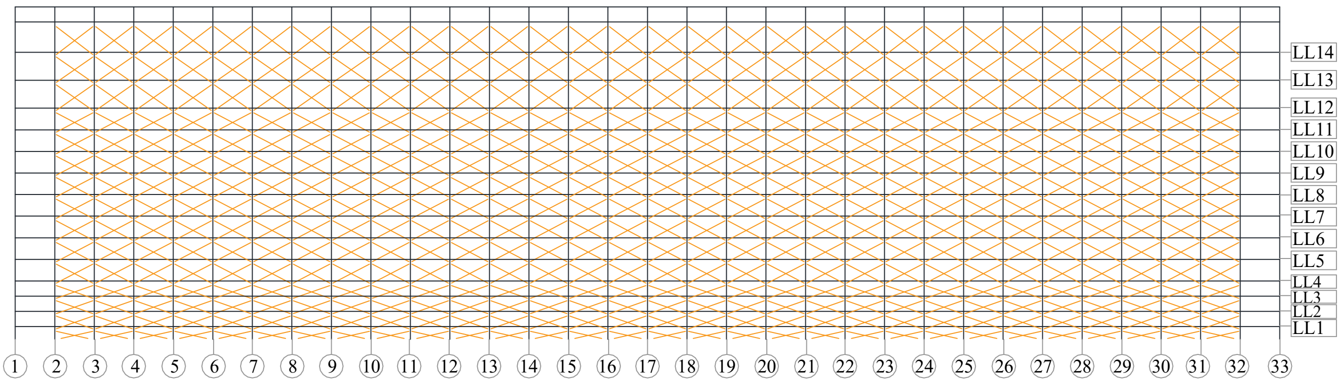


Figure 7-12: Longitudinal (DA) view of CS4.

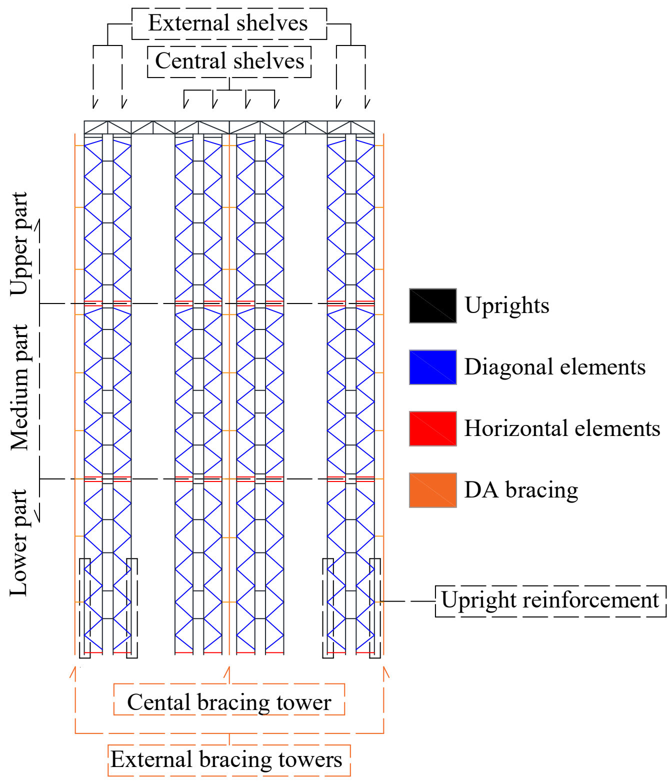


Figure 7-13: Transversal (CA) view of CS4.

Table 7-14: CA frames of CS4: sections of structural elements.

	Element	Location	Steel preparation process	Section profile shape	Steel grade
External shelves - Outer uprights	Uprights	Lower part	Cold-formed	280x120x4.0/2.5 (Reinforced) **Continuously Holed**	S500MC
			Cold-formed	140x120x4.0 **Continuously Holed**	S500MC
		Medium part	Cold-formed	140x120x3.5 **Continuously Holed**	S500MC
		Upper part	Cold-formed	140x120x2.5 **Continuously Holed**	S500MC
	Diagonal elements	Lower part	Cold-formed	2x C45x70x3.0	S355MC
		Medium and upper part	Cold-formed	C45x70x3.0	S355MC
External shelves	Uprights	Lower part	Cold-formed	140x120x4.0	S500MC

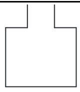
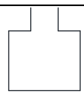
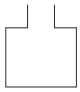
- Central uprights and Central shelves uprights		Medium part	Cold-formed		140x120x3.5	S500MC
		Upper part	Cold-formed		140x120x2.5	S500MC
	Diagonal elements	Whole height	Cold-formed		C45x70x3.0	S355MC

Table 7-15: CS4 bracing towers and load levels: sections of structural elements.

Element	Location	Steel preparation process	Section profile shape		Steel grade
Upright	Both external and central bracing tower	Cold-formed		140x120x4.0 **Continuously Holed**	S500MC
Diagonal elements (braces)	Both external and central bracing tower	Cold-formed	2x SHS 40x40x3.0		S355MC
Pallet beams	Load levels 1-3	Cold-formed	RHS 130x50x1.8		S275JR
	Load levels 4-11	Cold-formed	RHS 130x50x1.5		S275JR
	Load levels 12-14	Cold-formed	RHS 120x50x1.5		S275JR

Design parameters

Table 7-16 gathers the definition of the free design parameters within CS4. In particular, it can be noticed that 3 pallets for each couple of beams have been considered; besides, the definition of the participating seismic mass is made considering both the reduction factor RF and the Ψ_2 factor, according to what has been suggested within the common input design parameters (§ 7.1). The structure has been modelled as low dissipative in both directions. In particular, a behaviour factor q equal to 1.5 ha been used for CA direction (characterized by a K-shaped layout for diagonals), and a q factor equal to 2.0 for DA direction (characterized by X-braces structural typology). The design response spectrum has been defined by reducing the elastic one by the behaviour factor and the additional factor K_d (EN 16681 (2016), equation (7-III)), that in this case study is equal to 0.8. Finally, along CA direction, no reduction of stiffness has been applied to the CA frames.

Table 7-16: Definition of free design parameters within CS4.

Horizontal resisting systems	CA direction: K-shaped layout for diagonals.
	DA direction: X-shaped braces.
Number of pallets for each couple of beams	3.
Mass definition	CA direction: the participating mass has been calculated as $G_1 + G_2 + 0.8 \cdot Q_1$ DA direction: the participating mass has been calculated as $G_1 + G_2 + 0.8 \cdot 0.8 \cdot Q_1$

Definition of design response spectrum	Q factor = 1.5 for CA direction. Q factor = 2.0 for DA direction.	
	K_d =0.8 reduction factor considered.	
Reduction of stiffness along CA direction	Not applied.	
Modelling strategy	3D full model both for static and seismic design.	

7.2.5 Case Study 5

The structure is about 26.30 metres high, 14.90 m long along CA direction and about 101 m long along DA direction. Along DA direction, there are in total 33 CA frames, and there are 6 bracing towers, constituting the horizontal force resisting system along that direction (Figure 7-14). All the elements are cold-formed. Focusing on each CA frame, there are 16 uprights that are coupled through diagonal elements, that are organized in a K-shaped layout (Figure 7-15). Diagonals are designed to work both in tension and compression. Along DA direction, within each bracing tower, diagonals are arranged in the X-shaped layout and only work in tension. The horizontal resisting systems are placed in an eccentric position with respect to the upright shelves: there are two "external" bracing systems that are placed on the outer side of both the more external uprights, and there is a central bracing system that is placed in the middle of the frame between the two central shelves (Figure 7-15). This results in eccentricity between to centre of mass of the external shelves and the respective horizontal resisting system.

Structural typology and characteristics of the main components

Each CA frame is constituted by 16 uprights, that are coupled thought diagonal elements arranged in the K-shaped layout (Figure 7-15).

Table 7-17 summarizes the sections of the main structural elements that constitute the CA frames. In this case, in order to optimize performances-to-costs ratio, different thicknesses have been used for the external and central shelves, while the same sections are kept along the height of the structure. Uprights are continuously holed along their length. Dealing with connections, uprights are continuous from the bottom to the top, while diagonal profiles are directly connected to uprights through a single bolt. Along DA direction, each CA frame is connected to the following one through pallet beams, that are connected to the uprights through semi-rigid hooked connections. All the bracing systems are connected to the respective shelves through rigid tube elements (highlighted in red within Figure 7-15). Looking at the external shelves, the tubes connect the consecutive shelves and also the more external upright to the respective bracing system; in particular, from the external side of the more external upright, the tubes are fixed to the upright and, at the opposite end section, diagonal braces are directly connected to the tubes (no upright is placed in the plane of the bracing). Looking at the central shelves, the configuration is similar: the tubes connect the uprights of the consecutive shelves, and, at the middle, the diagonals of the central bracings are directly hinged to the tubes (also in this case, no upright is placed in the plane of the central bracing system). Both for the external and the central shelves, the tubes that connect the consecutive shelves are fixed at both end sections to the uprights through bolted connections, coupling the consecutive shelves.

Table 7-15 gathers the sections of the main structural elements that constitute the bracing towers and the pallet beams. All the elements base connections (uprights and DA braces of the bracing towers) are fixed to the concrete slab through post-installed bonded anchors.

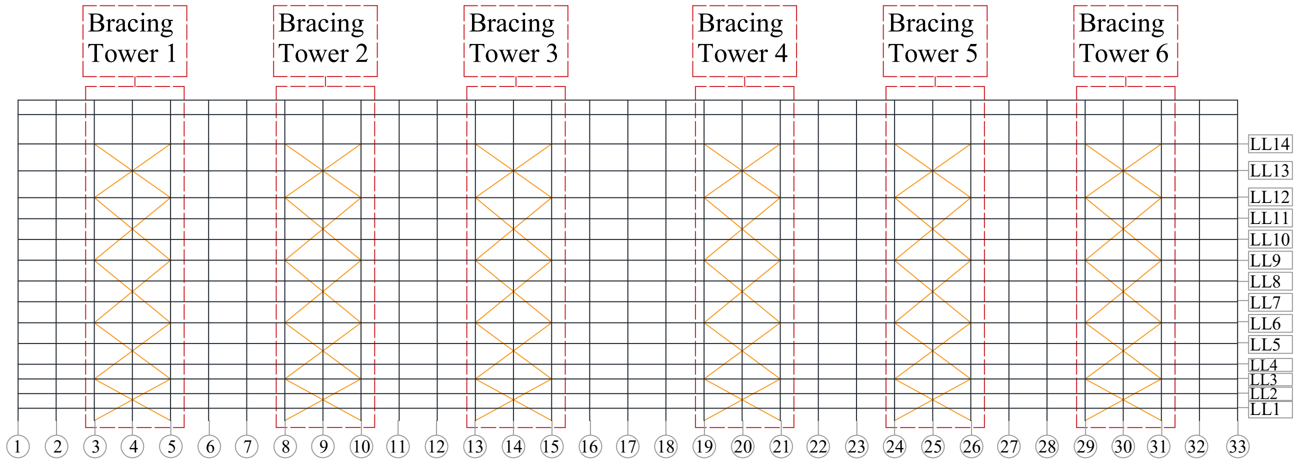


Figure 7-14: Longitudinal (DA) view of CS5.

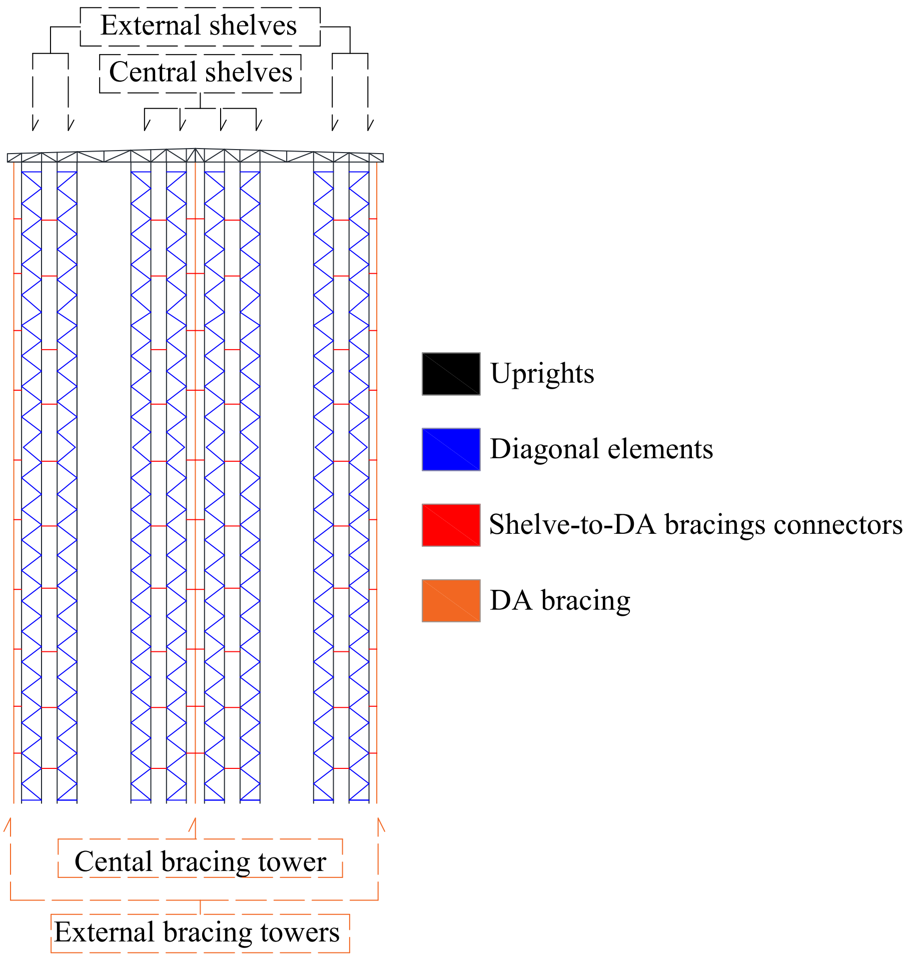


Figure 7-15: Transversal (CA) view of CS5.

Table 7-17: CA frames of CS5: sections of structural elements.

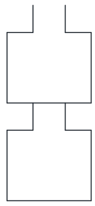
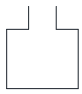
	Element	Location	Steel preparation process	Section profile shape	Steel grade
External shelves	Uprights	Whole height	Cold-formed	 254x115x5.0/3.5 **Continuously Holed**	S500MC
	Diagonal elements	Whole height	Hot-rolled	SHS 60x60x5.0	S235JR
Central shelves	Uprights	Whole height	Cold-formed	 126x115x5.0 **Continuously Holed**	S500MC
	Diagonal elements	Whole height	Cold-formed	SHS 50x50x2.0	S355MC

Table 7-18: CS5 bracing towers and load levels: sections of structural elements.

Element	Location	Steel preparation process	Section profile shape	Steel grade
Diagonal elements	All bracing towers	Hot-rolled	SHS 100x100x10.0	S275JR
Pallet beams	All load levels	Cold-formed	RHS 50x150x2.0	S235JR

Design parameters

Table 7-19 gathers the definition of the free design parameters within CS5. In particular, it can be noticed that 3 pallets for each couple of beams have been considered; besides, the definition of the participating seismic mass is made considering both the reduction factor RF and the Ψ_2 factor, according to what has been suggested within the common input design parameters (§ 7.1). The structure has been modelled as low dissipative in both directions. In particular, a behaviour factor q equal to 1.5 ha been used for both CA and DA directions (that are characterized respectively by a K-shaped and a X-shaped layout for diagonals). The design response spectrum has been defined by reducing the elastic one by the behaviour factor, while the additional factor K_d (EN 16681 (2016), equation (7-III)), has not been taken into consideration. Finally, along CA direction, no reduction of stiffness has been applied to the CA frames.

Table 7-19: Definition of free design parameters within CS5.

Horizontal resisting systems	CA direction: K-shaped layout for diagonals.
	DA direction: X-shaped braces.

Number of pallets for each couple of beams	3.
Mass definition	CA direction: the participating mass has been calculated as $G_1 + G_2 + 0.8 \cdot Q_1$ DA direction: the participating mass has been calculated as $G_1 + G_2 + 0.8 \cdot 0.8 \cdot Q_1$
Definition of design response spectrum	Q factor = 1.5 for CA direction. Q factor = 1.5 for DA direction.
	K_d not applied.
Reduction of stiffness along CA direction	Not applied.
Modelling strategy	3D full model both for static and seismic design.

7.3. Analysis of the configurations, of the structural choices and of the design strategies adopted within the case studies

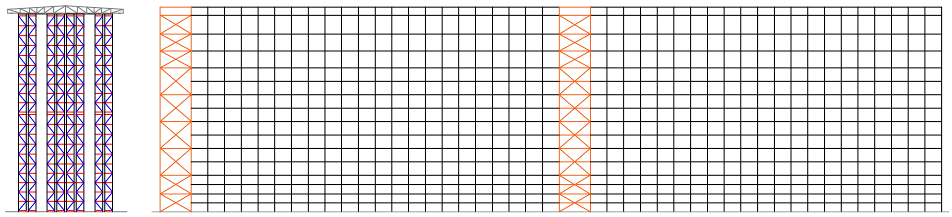
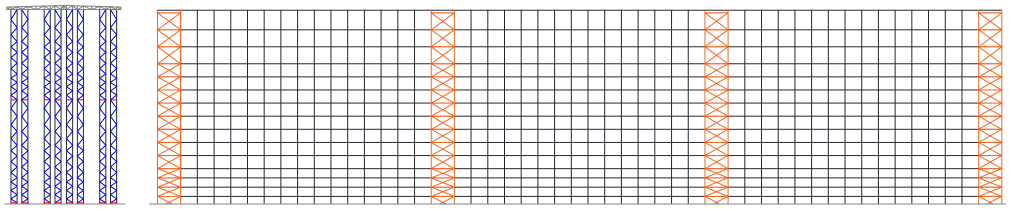
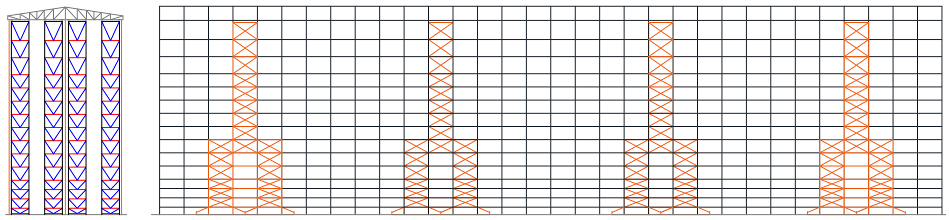
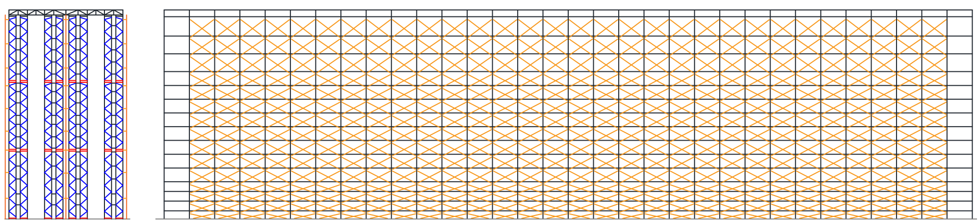
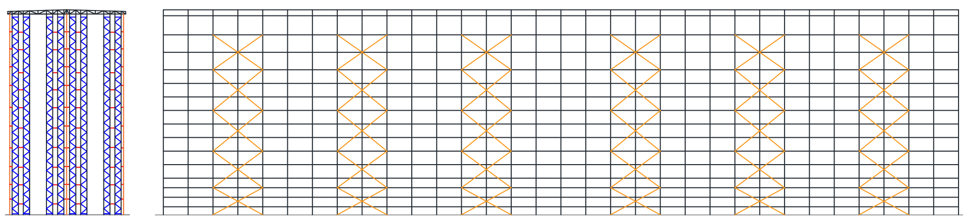
The analysis of the configurations, of the structural choices and of the design strategies adopted for the 5 case studies previously illustrated highlights that there are some common paths and distinguishing features that identify the current design approach for double depth ARSWs.

Concerning the structural choices, from the global point of view, different configurations can be identified along CA and DA direction (Table 7-20) the CA frames are in general constituted by repeated modular shelves, each one constituted of two uprights that are coupled through diagonals that are arranged in different layouts (truss, X, V, K schemes). The consecutive adjacent shelves are usually reciprocally connected through transversal elements, that are called "spacers", that can be hinged or fixed to the uprights (in case of using a moment resisting connection, also the shelves result coupled, and this choice affects significantly the stiffness of the structure along CA direction, as well as the distribution of forces in the uprights in case of horizontal forces acting). This configuration allows the shelves to resist both to vertical and horizontal loads.

- Along DA direction, the CA frames are repeated and connected by the pallet beams. The horizontal forces resisting system can be diffused along the length of the structure or located in strategic positions (in this case, each bracing system is called "bracing tower"), and the only structural scheme adopted within the analysed case studies is the X-shaped one (diagonal working in tension only). In any case, the bracing system can be aligned with the uprights constituting the shelves or placed in an eccentric position (usually, the one for the external shelves is placed outside the outermost upright, and, for the central bracing, it is placed in the middle of the central uprights). In this second configuration, the connection of the bracing system to the respective shelves is made through rigid transversal elements, that are usually placed at the height of the load levels and connect all the uprights of the shelves. This kind of solution implies, for the external shelves, an eccentricity of the centre of mass with respect to the centre of stiffness, and so, there may be, in addition to the translational modes, also not negligible rotational modes that can influence the response of the structure to horizontal forces along DA direction.

In general, the horizontal bracing system is placed in line with the load levels, directly connected to the pallet beams or to the uprights, and so, in case of having the eccentric bracing towers, it is aligned with the elements that connect the shelves to the bracing towers. In any case, no rigid plane can be found in such structures: the group of shelves that are separated by the aisles, can be considered almost one independent for the other ones: the only connection that involves all the shelves is the one at the top of the structure (constituted of the roof truss), that in any case in very far from the base of the uprights, and so its influence in the coupling of the shelves is very low.

Table 7-20: Global configurations and structural schemes of the case studies (BT means Bracing Tower).

CS	CA and DA direction views	Peculiar characteristics
1		CA: Truss scheme. Not coupled shelves (hinged spacers). DA: X-shaped bracing towers, aligned with the uprights.
2		CA: K-shaped scheme. No connection between consecutive adjacent shelves. DA: X-shaped bracing towers, aligned with the uprights.
3		CA: V-shaped scheme → reduced number of uprights and shelves. DA: X-shaped bracing towers placed in an eccentric position.
4		CA: K-shaped scheme. Coupled shelves (fixed spacers used also for the connection of the BT to the shelves) DA: X-shaped diffused bracing system placed in an eccentric position.
5		CA: K-shaped scheme. Coupled shelves (fixed spacers used also for the connection of the BT to the shelves). DA: X-shaped diffused bracing system placed in an eccentric position.

Key		Diagonals		Horizontal profiles		DA bracing system		Uprights and pallet beams

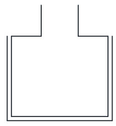
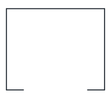
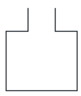
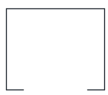
From the local point of view, the case studies share similar choices in terms of main profiles cross-sections and type of connections. Table 7-21 gathers and compares all the structural choices in terms of cross-section, type of connection and possible peculiarities of all the case studies. All the uprights are characterized by a C-shaped with lips cross-section and are holed along their length (with the only exception of those used within CS3 that are drilled only when necessary in correspondence with connections with diagonals and pallet beams). The C-shaped section with lips, that is opened in the inner side, allows faster and easy connection of diagonals, that are directly bolted to the uprights in correspondence with the lips without using additional sheets. All the diagonals used for the CA frames are characterized by a C-shaped section.

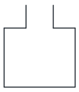
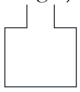
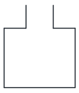
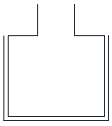
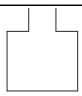
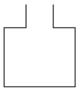
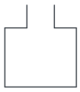
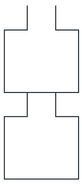
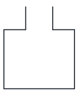
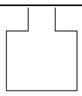
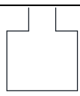
Along DA direction, different sections are used for the diagonals belonging to the bracing system, but these sections are actually very commonly used for this type of element, as C section, double C section, SHS. Very typical and commonly used sections in warehouses filed are also adopted for the pallet beam elements (E section, C section, double C section, rectangular section). The typical semi-rigid hooked connection is used when the C and rectangular sections are adopted for the pallet beam element, while, in the other cases, the pallet beams are directly bolted to the uprights or to other supporting elements. In any case, all the connections are always realized with the aim of limiting additional sheets and/or additional processes at the workshop (welds are barely used, and bolted joints are always preferred). According to the analysed case studies, it seems that the main path that guides all these structural choices is the structural optimization, that aims to balance the structural needs with: (i) limiting the costs connected to the necessary amount of steel and to the additional processes at workshops (i.e. welds or additional sheets for connections are very limited); (ii) for the same element (i.e. diagonals, uprights, pallet beams), limiting the number of different cross-sections needed (that implies an easier and cost effective production and a less probability of mistakes during the construction phases, since the thicknesses involved are very low, and so, for the same cross-section shape, it couldn't be so easy to distinguish very close thicknesses). In fact, the cross-section is always kept the same for each type of element, thicknesses change a maximum of three times along the height of the structure, that is not too much thinking that these structures are huge, the frames are repeated many times, and so the total number of elements is quite high.

Dealing with the design parameters to be freely adopted by the designers of the case studies, those that mainly influence the structural behaviour and the magnitude of the design forces of the structure are: the definition of the design response spectrum, the definition of the participant mass and the possible reduction of the lateral stiffness of the frames along CA direction. As regards the definition of the design response spectrum, it is obtained through the reduction of the elastic one by the behaviour factor, whose value depends on the structural typology used, and by the adoption of the K_d factor (EN16681 (2016), equation (7-III)). This last factor depends on the two coefficients E_{D1} and E_{D3} that respectively represent the capability of the pallets to dissipate energy through their sliding on the pallet beams and the dissipative phenomena that are typical of the dynamic behaviour of the whole racking system under seismic action. Assuming for K_d the value of 0.8 (corresponding to assuming E_{D1} equal to 1 and E_{D3} equal to 0.8, and a very reduced value of the weight of the structure with respect to the weight of the pallets, see equation (7-III)), Figure 7-16 shows the final design response spectra. In particular, a decrease of the seismic acceleration up to 46% corresponds to the assumption of K_d equal to 0.8 and q-factor equal to 1.5, while a decrease up to 60% can be reached adopting a q-factor equal to 2.0. Dealing with the definition of the seismic mass (§7.1, equation (7-I)), if both Ψ_2 and RF factors are assumed, in line with the common input deign parameters and with what indicated within EN16681 (2016), a reduction of the seismic mass up to 20% is obtained along CA direction and up to 46% is obtained along DA direction. This assumption directly affects the size of the total base due to seismic action, that is reduced by this reduction of mass, but is also increased due to the increment of the seismic acceleration (since, assuming the same stiffness, the mass decreases, and so also the fundamental natural period of the structure). As a consequence, it is not possible to say *a priori* if these assumptions finally determine an increase or a decrease of the design seismic base share. In any case, it is necessary to observe that not all the designers assumed the Ψ_2 factor equal to 0.8, believing that this could result in a too less conservative design input parameter. Finally, along CA direction, it is possible to take into consideration the reduction of the lateral shear stiffness of the frames due to the eccentricity of the diagonal-to-upright connections with respect to the centroid of the uprights. The size of the reduction of this lateral stiffness can be determined by the execution of shear experimental tests on the shelves constituting the structure. The procedure for the execution of this test is described within EN15512 (2009). This assumption basically can make the fundamental period of the structure (along CA direction) increase also up to 30-40%, being the structure more flexible, and determining a reduction of the seismic acceleration. It is necessary to observe that not all the designers have assumed this reduction, since not all of them have the experimental tests involving the elements and profiles adopted for the case study.

Table 7-23 highlights the effects of the definition of the design response spectrum and of the possible reduction of the lateral stiffness of the frames along CA direction in the final determination of the seismic base shear. The effects of the definition of the participant mass is not considered since it is not possible to assess *a priori* if this determines a reduction or an increase of the seismic base shear. In any case, it can be noticed that, in some cases, quite high reductions are reached (up to 60%), and could get worsen in case that also the reduction of the participant mass would be not conservative. In general, it is necessary to evaluate if these assumptions, that are justified for traditional racks, are also suitable for ARSWs or if they actually lead to an unsafe and not conservative design.

Table 7-21: Cross-section of the main structural elements, relative distinguishing characteristics and types of connection used.

CS	**CA frame**				**DA frame**			
	Optimization strategy		Upright	Diagonal		Bracing System		Pallet beams
						Upright	Braces	
1	Reduction of thickness of the cross-section from the bottom to the top (sections kept the same for all the shelves) → **2 different zones.**	**Lower part**	**C-shaped section with lips** 	**C-shaped section** 	**Lower part**	**HEB**	**L**	**C-shaped section**  (changing through the load levels)
		Upper part			**Upper part**	**HEA**	**L**	
		Info	• Uprights have an additional C-reinforcement at the bottom. • Uprights are continuously holed. • Diagonals are bolted directly to the uprights to avoid additional sheets.		**Info**	• Reduction of thickness from the bottom to the top. • Hot rolled elements are used for the bracing system. • Pallet beams are connected to the uprights through semi-rigid hooked connections.		

CS	CA frame: Optimization strategy		CA frame: Upright	CA frame: Diagonal	DA frame	DA frame: Bracing System: Upright	DA frame: Bracing System: Braces	DA frame: Pallet beams
2	Reduction of thickness of the cross-section from the bottom to the top (sections kept the same for all the shelves) **→ 2 different zones.**	**Lower part**	**C-shaped section with lips** 	**U-shaped section** 	**Lower part**	**C-shaped section with lips** (the same for the whole height) 	**L**	**E-shaped section**  (the same for all the load levels)
		Upper part			**Upper part**		**L**	
		Info	• Uprights are continuously holed. • Diagonals are bolted directly to the uprights to avoid additional sheets.		**Info**	• Reduction of thickness from the bottom to the top only for the braces. • Pallet beams are continuous and connected to the uprights through bolted connections.		
3	Reduction of thickness of the cross-section from the bottom to the top (sections kept the same for all the shelves) **→ 3 different zones for the uprights and 2 for the diagonals.**	**Lower part**	**C-shaped section with lips** 	**C-shaped section** 	**Lower part**	**Double C section**	**Double C section**	**Double C section** (the same for all the load levels)
		Middle part						
		Upper part			**Upper part**	**Double C section**	**Double C section**	
		Info	• Lower part uprights have an additional C-reinforcement at the bottom. • Uprights are NOT continuously holed, with the only exception of the bottom part of the lower uprights, to allow the connection of the reinforcement to the upright. • Diagonals are bolted directly to the uprights to avoid additional sheets.		**Info**	• Reduction of thickness from the bottom to the top. Higher thicknesses for the central bracing tower. • Pallet beams are continuous and connected to the uprights through bolted connections.		
4	Reduction of thickness of the cross-section from the bottom to the top and differences in thicknesses and cross-section between the central and the external shelves. **→ within each shelve, 3 different zones for the**		**C-shaped section with lips** **Ext. Shelve (Ext. Upright s)** / **Central Shelve and central uprights of ext. shelve**	**Ext. Shelve** / **Central Shelve**	**Whole height**	**C-shaped section with lips** 	**Double SHS cold-formed profiles**	**Rectangular section** (the same for all the load levels)
		Lower part	 / 	**Double C section** / **C section**				
		Middle part	 / 					

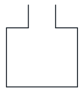
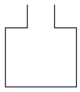
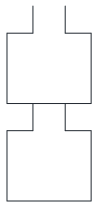
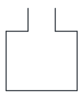
CS	CA frame: Optimization strategy		CA frame: Upright	CA frame: Diagonal	DA frame	DA frame: Bracing System – Upright	DA frame: Bracing System – Braces	DA frame: Pallet beams
	uprights and 2 for the diagonals of the external shelves.	**Upper part**	 	**C section**				
		Info	• Lower part uprights are reinforced through doubling the section. • Uprights are continuously holed. • Diagonals are bolted directly to the uprights to avoid additional sheets.		**Info**	• No reduction of thicknesses. • Pallet beams are connected to the uprights through semi-rigid hooked connections.		
5	Differences of thicknesses and cross-section between the central and the external shelves (sections are kept the same along the height of the structure).	**Lateral shelves**	**C-shaped section with lips** 	**C-shaped section** **SHS**	**Whole height**	**Not used**	**SHS**	**Rectangular section** (the same for all the load levels)
		Central shelves		SHS				
		Info	• Bigger sections belong to the external shelves. • Uprights are continuously holed. • Diagonals are bolted directly to the uprights to avoid additional sheets.		**Info**	• No reduction of thickness. • Braces are directly connected to rigid tube elements that are fixed to the uprights composing the shelves. • Hot rolled elements are used for the bracing system. • Pallet beams are connected to the uprights through semi-rigid hooked connections.		

Table 7-22: Summary of the free design parameters as adopted by the designers of the case studies (*).

		Horizontal resisting systems	Number of pallets for each couple of beams	Mass definition	Definition of design response spectrum: Behaviour factor	Definition of design response spectrum: K_d factor	Reduction of stiffness along CA direction	Modelling strategy
CS1	**CA**	Truss scheme	2	$G_1 + G_2 + Q_1$	1.5	0.8	Considered	3D full model
	DA	X-shaped scheme **BT**		$G_1 + G_2 + 0.8 \cdot Q_1$	1.5			
CS2	**CA**	K-shaped scheme	2	$G_1 + G_2 + Q_1$	1.5	0.8	Considered	3D full model
	DA	X-shaped scheme **BT**		$G_1 + G_2 + 0.8 \cdot Q_1$	2.0			
CS3	**CA**	V-shaped scheme	3	$G_1 + G_2 + 0.8 \cdot Q_1$	1.5	0.8	Not applied	2D models representative of the two directions
	DA	X-shaped scheme **BT**		$G_1 + G_2 + 0.8 \cdot 0.8 \cdot Q_1$	2.0			
CS4	**CA**	K-shaped scheme	3	$G_1 + G_2 + 0.8 \cdot Q_1$	1.5	0.8	Not applied	3D full model
	DA	X-shaped scheme **D**		$G_1 + G_2 + 0.8 \cdot 0.8 \cdot Q_1$	2.0			
CS5	**CA**	K-shaped scheme	3	$G_1 + G_2 + 0.8 \cdot Q_1$	1.5	1	Not applied	3D full model
	DA	X-shaped scheme **BT**		$G_1 + G_2 + 0.8 \cdot 0.8 \cdot Q_1$	1.5			

(*) D means "Diffused", while BT means "Bracing Tower".

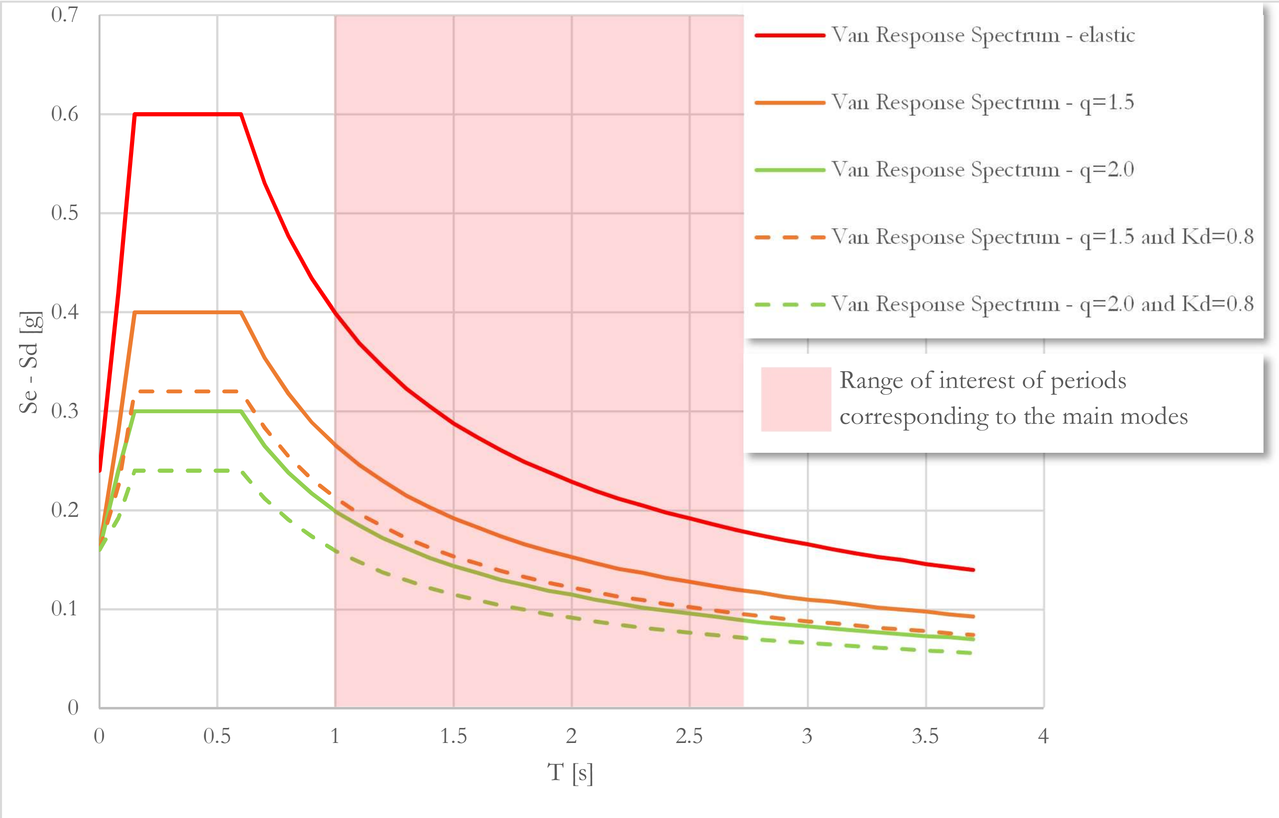


Figure 7-16: Definition of the design response spectrum according to all the possible reducing parameters to be assumed.

Table 7-23: Influence of the design assumptions in the reduction of the seismic design base shear.

Case Study	Direction	Seismic acceleration			Mass		Total Seismic base shear reduction [%]
		q	Kd	Lateral stiffness	Ψ_2	RF	
1	CA	1,5	0,8	YES	1,00	1,00	**52**
	DA	1,5	0,8	NO reduction	1,00	0,80	**46**
2	CA	1,5	0,8	YES	1,00	1,00	**52**
	DA	2,0	0,8	NO reduction	1,00	0,80	**60**
3	CA	1,5	0,8	NO reduction	0,80	1,00	**46**
	DA	2,0	0,8	NO reduction	0,80	0,80	**60**
4	CA	1,5	0,8	NO reduction	0,80	1,00	**46**
	DA	2,0	0,8	NO reduction	0,80	0,80	**60**
5	CA	1,5	1,0	NO reduction	0,80	1,00	**33**
	DA	1,5	1,0	NO reduction	0,80	0,80	**33**

7.4. Structural assessment of the case studies

The structural assessment of the case studies has been done through the execution of Finite Element Method (FEM) numerical analyses and the evaluation safety level at Life Safety Limit State (LSLS) through the execution of safety checks of the main components (elements and connections).
All the input data for the definition of the numerical model have been kept the same as defined by the designers of the case studies. At first, for each case study, modal analysis has been executed, in order to point out the main modes and the corresponding natural frequencies. The modal analyses have been executed starting from a deformed condition, that has been obtained by the execution of a non-linear analysis (only geometrical non-linearities included - P-Delta effect on columns) with only gravitational loads acting. Then, Non-Linear Time History Analyses (NLTHA) (only geometrical non-linearities included) have been carried out using a set of 15 natural accelerograms as seismic input. This set of accelerograms has been selected within the framework of the STEELWAR Research Project from the NGA-West2 database (Bozorgnia et al. 2014) that match the target Conditional Spectra (CS) (J. Baker 2011; Lin, Haselton, and Baker 2013a; 2013b) at a 2475 years return period, or equivalently an exceedance probability of 2% in 50 years. In addition, record selection has also been performed for a wider range of probabilities of exceedance, for reasons of completeness, and in order to be applied to further analyses (Kohrangi M., Tsarpalis D., Vamvatsikos D. Deliverable D.4.2. From Steelwar Research Project). The vulnerability assessment of the case studies has been executed using the set of records corresponding to an exceedance probability of 10% that is, among those available, the one closer to the one corresponding to the design response spectrum: in fact, the design response spectrum is defined for a return period equal to 475 years, that, considering that the warehouse is characterized by the importance class I (importance factor equal to 0.8 and a 50 years design life), corresponds to a probability of exceedance equal to 20%. In any case, the fact that the seismic input for the vulnerability assessment does not match the design response spectrum is not relevant in this phase, since the aim of the vulnerability assessment is to catch the global structural behaviour at LSLS resulting from the application of the current design strategies. In particular, the weaker parts of the structure and the chain of mechanisms are individuated by putting in order (form the highest to the lowest) the demand-capacity ratios obtained from the execution of the safety checks of elements and connections. These re-arranged demand-capacity ratios are illustrated, representing the so called "hierarchy of criticalities". The safety checks have been executed by the application of the prescriptions within Eurocode 3 (in particular prEN 1993-1-1:2019, prEN 1993-1-3:2019 and prEN 1993-1-8:2019) and EOTA documents for base connections, when post-installed anchors are used (2010).

7.4.1 Modal Analysis

For all the case studies, modal analyses of the 3D structures have been carried out. Table 7-24 summarizes the modelling strategies that have been adopted. Basically, to consider possible second order effects, the modal analyses start from a deformed condition resulting from a non-linear analysis (geometrical non-linearities included) with only gravitational loads acting. Material is linear elastic, and all the elements have been modelled as mono-dimensional frame element. Connections are punctually modelled in order to be representative of the structural behaviour of the joints, as drawn within the technical drawings of the case studies. The participant mass has been defined in agreement with the assumption made within each case studies (Table 7-22). With regard to the mass of the pallet load, for each load level, it is modelled as lumped and placed at the right height, in correspondence with center of mass of the pallets, to consider the effects of vertical eccentricity on the axial load of uprights. The connection of the mass with the structure is made through a reverse T-shaped "dummy" substructure (EN16618 (2016)), that is modelled to be sufficiently more rigid than the rest of the structure (to avoid relevant modes of vibration on the substructure when to flexible, or numerical instability when too much stiff) and with no mass associated (Figure 7-17 from Table 7-24).

Table 7-24: Characteristics of the model for the execution of modal analysis.

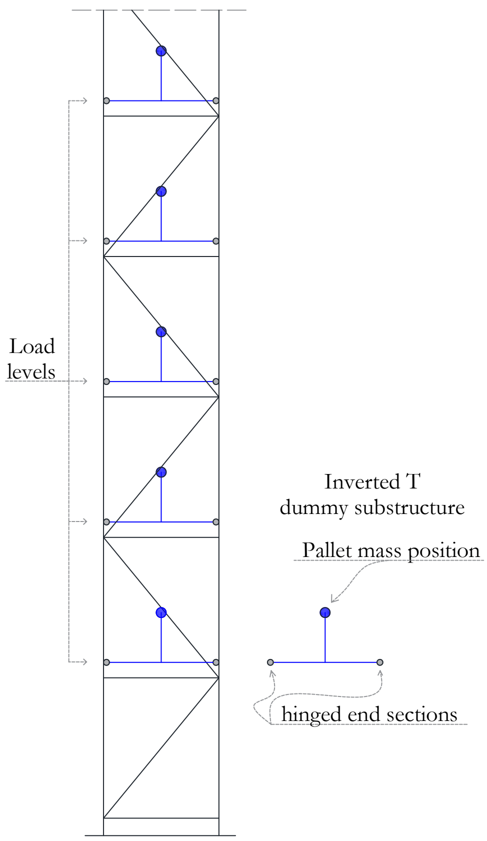


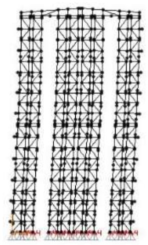
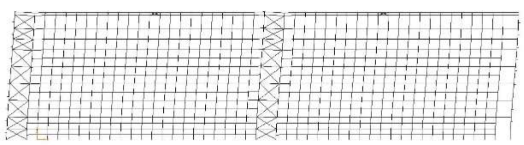
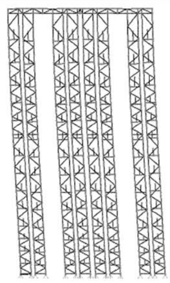
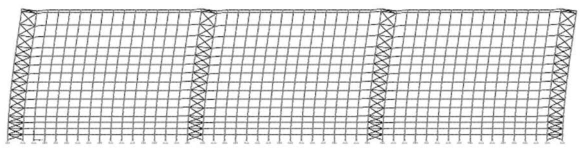
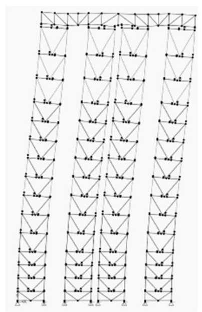
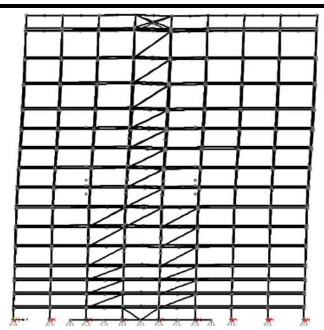
Figure 7-17: Modelling of pallet mass.

Software	SAP2000® (Finite Element modelling).
Type of analysis	Modal analysis (to include P-Delta effects, modal analysis starts from a deformed condition obtained by the execution of a non-linear analysis with only gravitational loads acting, including P-Delta effects on columns).
Model dimension	3D.
Geometry data	From case study drawings.
Material behaviour	Linear elastic.
Definition of elements section	Sections geometry defined according to case study drawings.
Type of element	Mono-dimensional *frame element* used for all the elements.
Elements connections and restrains	The connections are schematized in other to represent the structural behaviour of the joints as drawn in the technical drawings of the case studies.
Load definition	FULL LOAD CONDITION is always considered: beside dead load, pallet load is defined according to the different load levels and schematized as a uniformly distributed load along pallet beam length. Load combination adopted: $G_1 + G_2 + Q_{pallet}$, where G_1 is dead load, G_2 is non-structural elements load (not much relevant), and Q_{pallet} is pallet load.
Mass definition	Pallet load mass is placed in the upper nodes of the T-shaped dummies substructure, as indicated within EN16681 (2016), to take into consideration the actual position of the centre of mass of the pallet, and its consequent effects on the distribution of forces in the elements (especially on uprights). The participating mass is defined according to the assumptions of the designers of the case studies (Table 7-22).

When uprights have holes along their length, the possible resulting reduction of stiffness along CA direction is evaluated by the adoption of the equivalent cross-section for uprights. The equivalent section is determined through the application of the method shown within Annex G of prEN15512 (2018), that

basically consist in the determination of a reduced thickness to be used in correspondence with the holes. This magnitude of reduction of thickness depends on the size of the hole and the interspace of the holes. It is also possible that the pattern of the hole does not imply any reduction of thickness.
The modal analyses have been performed on 3D models. Since the structures are quite big, when possible, a significative and representative portion of the structure has been modelled. The choice of the portion of the structure has been made based mostly on the typology (diffused braces, bracing towers) and position of the vertical bracing along DA direction, and accurately selecting the corresponding influence area of the bracing.

Table 7-25: Relevant eigen modes resulting from modal analysis of the case studies.

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

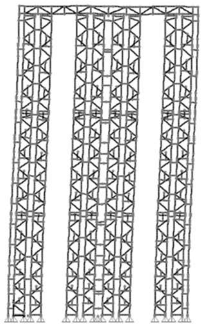
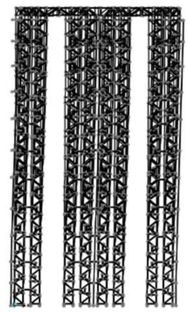
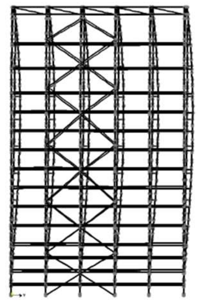
4	Translational, DA direction	1.08	67		
	Translational, CA direction	0.96	64		
				T 0.96s mass 52%	T 1.08s mass 57%
5	Translational, DA direction	1.29	37		
	Translational, CA direction	1.10	11		
	Translational, CA direction	1.09	40		
	Translational, CA direction	1.03	9		
	Translational, DA direction	0.98	29		
	Translational, DA direction	0.61	5	T 1.09s mass 40%	T 1.29s mass 37%

Table 7-25 gathers the relevant eigen modes of all the case studies structures. In particular, it can be noticed that no relevant variability of periods can be found among the main modes of DA direction, especially if we look at the case studies where bracing towers are adopted (all of them but CS4, where diffused braces are used). The case study CS4 is the one characterized by the lowest period along that direction. A relevant variation of period can be found among the main modes along CA direction, and this actually depends on presence and typology of connection between the consecutive shelves. In fact, looking at CS1 and CS2, these are the case studies characterized by the highest periods, and, in the former, the consecutive shelves are connected by hinged and very flexible elements, and, in the latter, there is no connection between the consecutive shelves. On the other hand, in all the other case studies, the consecutive shelves are connected by rigid transversal beams at each load levels, that allow the connection of the shelves to the corresponding bracing tower, that is placed in an eccentric position. These rigid elements couple the shelves, making the structure more rigid along CA direction.

7.4.2 **Non-linear Time History analyses**

Non-Linear Time History analyses (only geometrical non-linearities included) have been carried out using a set of 15 natural accelerograms as seismic input. This set of accelerograms has been selected within the framework of the STEELWAR Research Project from the NGA-West2 database (Bozorgnia et al. 2014) that match the target Conditional Spectra (CS) (J. Baker 2011; Lin, Haselton, and Baker 2013a; 2013b) at a 2475 years return period, or equivalently an exceedance probability of 2% in 50 years. In addition, record selection has also been performed for a wider range of probabilities of exceedance, for reasons of completeness, and in order to be applied to further analyses (Kohrangi M., Tsarpalis D., Vamvatsikos D. Deliverable D.4.2. From Steelwar Research Project). The selection procedure is based on the approximate method of CS (Lin, Haselton, and Baker 2013b) using the geometric mean of spectral accelerations as the Intensity Measure, IM (Kohrangi et al. 2017).The Ground Motion Prediction Equations of Boore and Atkinson (2008) are used for all purposes of the record selection, starting from hazard analysis. The procedure for the selection of the accelerograms is given within Deliverable 4.2 of Steelwar Research Project (Kohrangi M., Tsarpalis D., Vamvatsikos D. 2018). Figure 7-18 shows the record selected for each probability of exceedance for Van city. These figures represent the set of 30 records, but in the framework of this thesis, the set of 15 accelerograms is used. Table 7-26 gathers the scale factors, that are used to scale the 2% probability of exceedance records to obtain the other accelerograms defined for the other probability of exceedance.

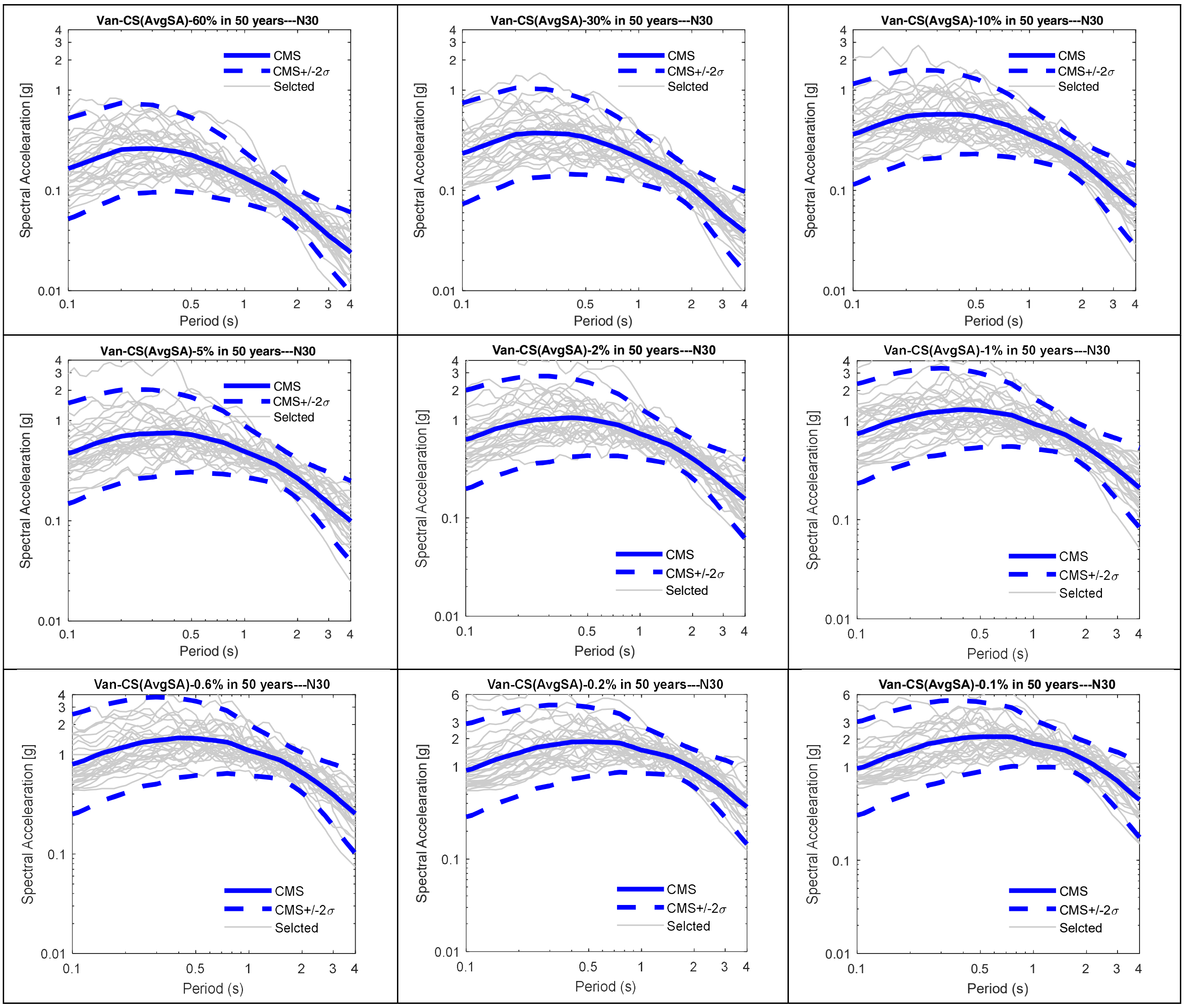


Figure 7-18: Selected records and the 2.5th/50th/97.5th percentiles for Van (these figures are taken from Deliverable 4.2 of Steelwar Research Project (Kohrangi M., Tsarpalis D., Vamvatsikos D. 2018)).

Table 7-26: Scale factors used to scale the records.

	2% / 50 Y	60% / 50 Y	30% /50 Y	10% /50 Y	5% /50 Y	1% /50 Y	0.6% /50 Y	0.2% /50 Y	0.1% /50 Y
AvgSA [g]	0.4990	0.0870	0.1380	0.2410	0.3340	0.6560	0.7890	1.1060	1.3330
Acc.	Scale Factors								
	SF (2%/50)	SF (60%/50)	SF (30%/50)	SF (10%/50)	SF (5%/50)	SF (1%/50)	SF (0.6%/50)	SF (0.2%/50)	SF (0.1%/50)
1	2.3577	0.4111	0.6520	1.1387	1.5781	3.0995	3.7279	5.2257	6.2982
2	4.1442	0.7225	1.1461	2.0015	2.7739	5.4481	6.5527	9.1854	11.0707

3	5.7066	0.9949	1.5782	2.7561	3.8197	7.5021	9.0231	12.6483	15.2443
4	2.3164	0.4039	0.6406	1.1188	1.5505	3.0453	3.6627	5.1343	6.1880
5	9.7842	1.7059	2.7059	4.7255	6.5490	12.8626	15.4705	21.6861	26.1371
6	3.6942	0.6441	1.0216	1.7842	2.4727	4.8565	5.8411	8.1880	9.8685
7	7.9841	1.3920	2.2080	3.8561	5.3441	10.4962	12.6242	17.6963	21.3283
8	5.0573	0.8817	1.3986	2.4425	3.3851	6.6485	7.9965	11.2092	13.5098
9	4.3039	0.7504	1.1903	2.0787	2.8808	5.6581	6.8052	9.5394	11.4973
10	5.5614	0.9696	1.5380	2.6860	3.7225	7.3112	8.7935	12.3265	14.8564
11	0.8991	0.1568	0.2486	0.4342	0.6018	1.1820	1.4216	1.9928	2.4018
12	4.0280	0.7023	1.1139	1.9454	2.6961	5.2953	6.3689	8.9277	10.7601
13	2.5756	0.4490	0.7123	1.2439	1.7239	3.3859	4.0724	5.7086	6.8803
14	5.4605	0.9520	1.5101	2.6372	3.6549	7.1785	8.6339	12.1028	14.5868
15	9.6424	1.6811	2.6666	4.6570	6.4540	12.6762	15.2462	21.3717	25.7582

The set of records that has been used for the vulnerability assessment of the case studies corresponds to an exceedance probability of 10% that is, among those available, the one closer to the one corresponding to the design response spectrum: in fact, the design response spectrum is defined for a return period equal to 475 years, that, considering that the warehouse is characterized by the importance class I (importance factor equal to 0.8 and a 50 years design life), corresponds to a probability of exceedance equal to 20%. In any case, in this phase, it is not relevant that the seismic input for the vulnerability assessment does not match precisely the design response spectrum. In fact, the aim of the vulnerability assessment is to catch the global structural behaviour resulting from the application of the current design strategies and highlight the weaker parts of the structure. The characteristics and strategies adopted to model of the structure are gathered in Table 7-27.

For each case study, the possibility to execute the analyses on reduced parts of the structure has been evaluated, since these structures are huge, and so a very high quantity of elements and nodes would be involved if a 3D model of the entire structure had been done. This would imply an increment of geometrical complexity of the model, higher number of elements and nodes, and so, of the number of equations to be solved to finalize the analyses. Basically, if geometrical simplifications (when possible) were not implemented, the time for the execution of the analyses would significantly elongate, and also the storage of the resulting data would be tricky, since the quantity of data to be saved would be massive. The possible simplifications to be adopted have been carefully pondered for each case study.

Table 7-28 gathers the geometrical simplifications adopted for each case study. In particular:

- CS1: in this case study, the effects of seismic action along CA direction have been evaluated modelling one 2D CA frame, that can be assumed representative since each one can be considered independent from the others (each CA frames is characterized by the same stiffness, the same quantity of associated mass and vertical load). Along DA direction, the same considerations can be made, since for each alignment there are two in-plane bracing towers. So, the effects of seismic action along CA direction have been evaluated modelling one whole 2D DA frame.
- CS2: in this case study, the effects of seismic action along CA direction have been evaluated modelling one 2D CA frame, that can be assumed representative since each one can be considered independent from the others (each CA frames is characterized by the same stiffness, the same quantity of associated mass and vertical load). Along DA direction, the same

considerations can be made, since for each alignment there are two in-plane bracing towers. So, the effects of seismic action along CA direction have been evaluated modelling one whole 2D DA frame.

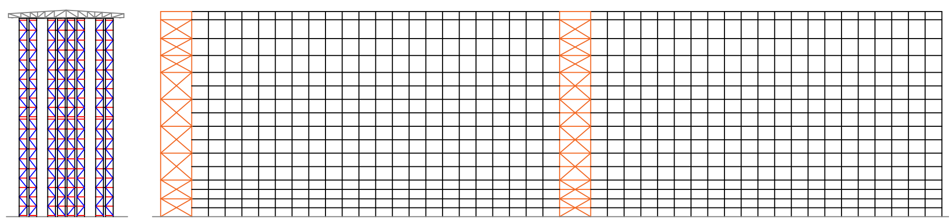
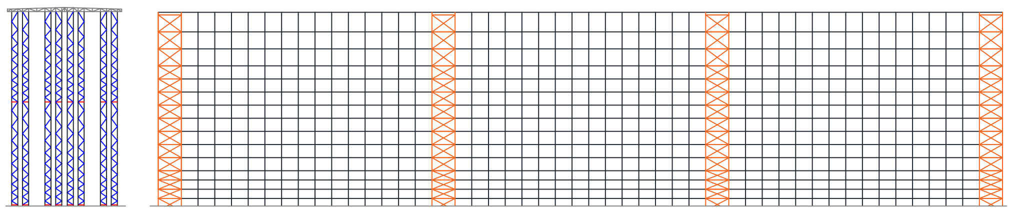
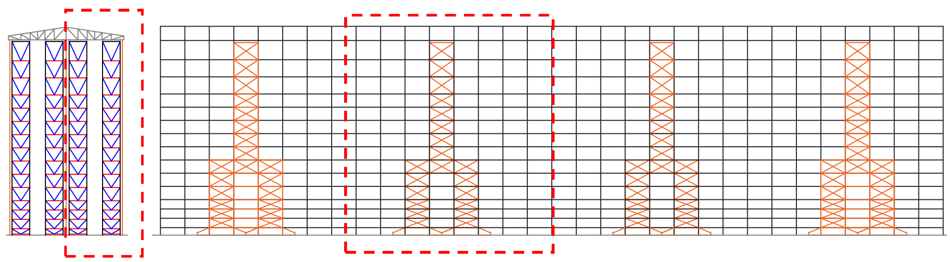
- CS3: in this case study, the effects of seismic action along CA direction have been evaluated modelling one 2D CA frame. All the CA frames are characterized by the same participating mass and vertical load, but not by the same stiffness: in fact, the CA frames in correspondence with the vertical bracing towers are more rigid than the others, since in these ones the shelves are connected by rigid transversal elements that are placed to connect the shelves to the respective bracing tower (that are placed in an eccentric position). This connection among consecutive shelves is not present in all the other CA frames (those that are not in correspondence with the bracing towers). So, to account for the safer assumptions, the CA frame with the shelves connected has been modelled. Along DA direction, a 3D model has been used, analysing the portions of structure that belong to the influence area of the bracing towers, and taking the biggest one. Using a 3D model is the only possible strategy in this case, since the vertical bracings along DA direction are placed in an eccentric position with respect to the uprights belonging to the shelves. An additional simplification has been assumed, modelling half of that portion along CA direction, taking advantage of the longitudinal symmetry of the structure.
- CS4: in this case study, a unique 3D model has been used to evaluate the effects of the seismic action along CA and DA direction. This choice has been made because along DA direction is possible to reduce the whole structure to only two CA frames and the relative vertical bracing, since the braces along DA direction are diffused all along this length.
- CS5: in this case study, the effects of seismic action along CA direction have been evaluated modelling one 2D CA frame, that can be assumed representative since each one can be considered independent from the others (each CA frames is characterized by the same stiffness, the same quantity of associated mass and vertical load). Along DA direction, a 3D model has been used, analysing the portions of structure that belong to the influence area of the bracing towers, and taking the biggest one. Using a 3D model is the only possible strategy in this case, since the vertical bracings along DA direction are placed in an eccentric position with respect to the uprights belonging to the shelves.

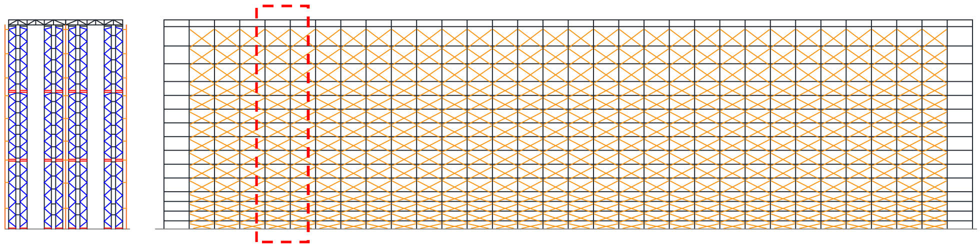
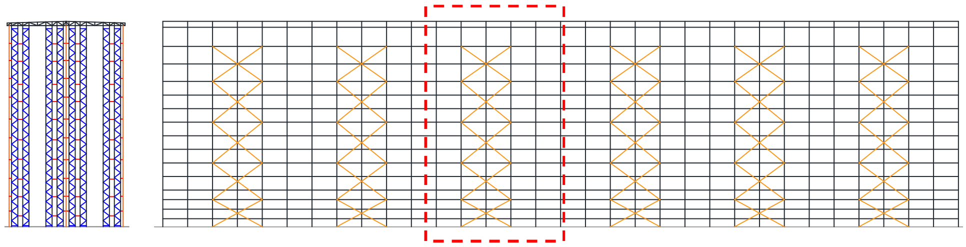
Table 7-27: Characteristics of the model built for the execution of time history analyses.

Software	Opensees ® (Mazzoni et al 2017) (Finite element modelling).
Type of analysis	Non-linear time history analysis, including P-Delta effects on columns.
Models dimension	2D or 3D.
Geometry data	From case study drawings.
Material behaviour	Linear elastic.
Definition of elements sections	Sections geometry defined according to case study drawings.

Definition of elements	Mono-dimensional elements used for all the elements (in general, *elastic-beam-column element* has been used, and, in case of 2D models, *truss element* has been used to model hinged elements).
Elements connections and restrains	The connections are schematized in other to represent the structural behaviour of the joints as drawn in the technical drawings of the case studies.
Load definition	FULL LOAD CONDITION is always considered: beside dead load, pallet load is defined according to the different load levels and schematized as a uniformly distributed load along pallet beam length. Load combination adopted: $G_1 + G_2 + Q_{pallet}$, where G_1 is dead load, G_2 is non-structural elements load (not much relevant), and Q_{pallet} is pallet load. Seismic input consists in a 15 ground motion record set selected in the framework of STEELWAR research project, matching the peak ground acceleration of the area (0.3g in 475 years) and corresponding to a probability of exceedance equal to 10%.
Mass definition	Pallet load mass is placed in the upper nodes of the T-shaped dummies substructure, as indicated within EN16681 (2016), to take into consideration the actual position of the centre of mass of the pallet, and its consequent effects on the distribution of forces in the elements (especially on uprights). The participating mass is defined according to the assumptions of the designers of the case studies (Table 7-22).

Table 7-28: Geometrical simplification adopted for the execution of NLTHA.

CS	CA and DA direction views, portions of the structure considered in the FEM model highlighted	Modelling strategy
1		**CA direction of seismic input:** CA frame → 2D model **DA direction of seismic input:** In-plane vertical bracing → one DA frame fully modelled → 2D model
2		**CA direction of seismic input:** CA frame → 2D model **DA seismic input direction:** In-plane vertical bracing → one DA frame fully modelled → 2D model
3		**CA direction of seismic input:** CA frame → 2D model **DA seismic input direction:** Portion of the structure along DA direction, half of the structure along CA direction (taking advantage of the longitudinal symmetry) → 3D model

CS	CA and DA direction views, portions of the structure considered in the FEM model highlighted	Modelling strategy
4		**CA and DA direction of seismic input:** One single 3D model used for both directions of seismic input (2 CA frames and relative bracing system along DA direction have been modelled).
5		**CA direction of seismic input:** CA frame → 2D model **DA seismic input direction:** Portion of the structure along DA direction → 3D model

Key							
		Diagonals		Horizontal profiles		DA bracing system		Uprights and pallet beams

7.4.3 Vulnerability assessment and definition of the "hierarchy of criticalities"

The vulnerability assessment is carried out through the execution of the safety checks of the main structural elements and connections by the application of the prescriptions within Eurocode 3 (in particular prEN 1993-1-1:2019, prEN 1993-1-3:2019 and prEN 1993-1-8:2019) and EOTA documents for base connections, when post-installed anchors are used (in particular EOTA TR45 and EOTA TR49 (2010)). The huge quantity of data that result from the model, consisting in the record of the forces acting on the elements to be checked, has been handled through an auxiliary software (MATLAB®) that has allowed to automate the extraction of the data from the records (that have been saved on text files) and the execution the safety checks, thanks to a specific script that has been defined accurately for each case study, based on the type of element (or connection), the forces acting, the structural characteristics and all the possible mechanism that can involve the analysed component.

Table 7-29 shows all the mechanisms that have been checked for each typology of element and connection.

Table 7-29: Mechanism checked for each element and connection.

ELEMENTS	
Uprights	Shear resistance Tensile resistance Bending resistance Tensile + bending resistance Buckling (flexural, torsional and torsional flexural when necessary) Bending and axial compression (resistance) Bending and axial compression (stability)
Diagonal and Horizontal (hinged elements, in general)	Tensile resistance Buckling (flexural, torsional and torsional flexural when necessary)

Braces (hinged elements working only in tension)	Tensile resistance
Pallet beams (DA model)	Shear resistance Tensile resistance Bending Resistance Tensile + Bending resistance Flexural Buckling Bending and axial compression (resistance) Bending and axial compression (stability)
CONNECTIONS	
Diagonal and Horizontal elements -to – uprights (CA frame)	Shear resistance of bolt Plastic ovalization Failure of the net section Failure of the ending part
Connections where additional sheets or welds are required	Bolted connection (Diagonal to plate, or plate to upright): - Shear resistance of bolt - Plastic ovalization - Failure of the net section - Failure of the ending part (when applicable) Welded connection (Plate to column or element to plate): - Stress check
Pallet beam – to – Upright (hooked connection)	Bending resistance From the experimental tests on this kind of connection, only maximum negative bending resistance is available (indeed, for ordinary static conditions, where only vertical pallet load is acting, only negative bending force can act on this connection). So, it is not possible to know the maximum positive bending resistance, necessary to check these connections when also seismic load is acting, and positive bending moment can occur.
Base connections	Upright to base plate connection (bolted: shear resistance of bolts, ovalization and net section failure) Base plate checks Anchor checks (Tensile force, Shear force, Tensile + Shear force)

After the execution of safety checks, the weaker parts of the structure and the chain of mechanisms are individuated by putting in order (form the highest to the lowest) the demand-capacity ratio obtained from the execution of the safety checks. These re-arranged demand-capacity ratios are illustrated in the following tables, representing the so called "hierarchy of criticalities". It is important to remark that the hierarchy of criticalities are only a graphical representation of the demand-capacity ratios (D/C ratio), where these ratios have been obtained from the execution of the safety checks, based on the forces resulting from the execution of numerical analyses where all the elements are elastic (only geometrical non-linearities have been included in the models). For each case study, the hierarchies for seismic action in both directions are represented. For each direction, the results are gathered in consecutive steps, and a range of D/C ratio corresponds to each step: the first step is the one with the mechanisms characterized by the highest value of D/C ratios, and, in the following steps, the values of D/C decrease. Each step is so individuated by a range of D/C ratios, that are here proportional to the lowest D/C ratio value represented in the tables (that is the lower value of the last step): taking as an example the first step of Table 7-30 (that gathers the hierarchies of CS1, CA direction), it is individuated by the range 1.59-1.55, where 1.59 is the ratio between the highest D/C value represented in this step and the lowest D/C value represented in the last represented step, and 1.55 is the ratio between the lowest D/C ratio represented in this step and the lowest D/C ratio represented in the last represented step. The mechanisms related

to connections are individuated by a circle, that is placed in correspondence with the element whose connection fails. The mechanisms related to elements are individuated by simply highlighting the element with a non-black color. Within each step, the mechanisms that belong to that same step are characterized by the red color, while the mechanisms that belong to previous steps are marked with the yellow color. Besides, for each step, the first occurring mechanism is pointed out with a blue arrow, while the last occurring one with a green arrow.
Referring to the hierarchy of criticalities corresponding to seismic action along CA direction, a similar behaviour can be individuated among all the case studies:

- the higher forces are concentrated at the bottom of the structure. In fact, the highest D/C ratios are placed there. From the bottom, the mechanisms spread through the height of the structure. A major diffusion of the mechanisms is allowed when different thicknesses along the height of the structure are used for diagonal or upright elements (this is more evident within CS1 (Table 7-30), CS4 (Table 7-36) and CS5 (Table 7-38)).
- In all the case studies, the components that are characterized by the highest D/C ratios are diagonal connections and uprights base connections: the leading mechanism for diagonal connections is plastic ovalization, while the leading one for base connections is failure due to tensile and shear force on anchors, were the mechanism associated to tensile force is the concrete-cone one, and the mechanism connected to shear force is failure of anchor. Both of these mechanisms are fragile. Failure of base connections is relevant in all the case studies where anchors used are the post-installed ones. The only case study where base connections do not fail first is CS3, where traditional base connections through threaded bars have been chosen, allowing better performances.

  As regards diagonal elements, it can be noticed that the ultimate resistance of the element (both tensile and in compression) is always higher that the resistance of the connection (at least 40% higher). This happens because, according to EN16681 (2016), if behaviour factors up to 2 are used, no hierarchy design rules are mandatory to be applied. The only request is to avoid a fragile failure of connection by having the shear resistance of bolts at least 1.20 times the plastic ovalization. This design strategy implies that, although fragile failure of connection is prevented, no over-resistance is provided to connection with respect to diagonal element, and connections are designed directly with the reduced design forces from numerical analysis. Considering that diagonals are directly bolted to uprights, and that these elements are characterized by very low thicknesses, among all the mechanism connected to this kind of connection, the leading one (the one characterized by the lower resistance) is the plastic ovalization. Basically, firstly connection is designed, and then the diagonal, based on the thickness, the diameter of the bolt, and the grade of the steel that is needed to have a sufficient resistance connection.
- The consequent criticality that occurs is buckling failure of uprights at the bottom of the structure, or where the reinforcement stops, if the reinforcement provides sufficient resistance. The highest D/C ratios are obtained for combined axial compression and bending. Axial compression force acting on uprights is in general very high, considering that the first relevant rate is due to the weight of the goods, and the second one is due to seismic action. Form capacity side, a good resistance of uprights in compression is obtained, adopting different strategies (as outside reinforcement, or local reinforcement) to control local and global buckling phenomena. A relevant rate of D/C ratio is due to bending, since, although bending force is moderate, the

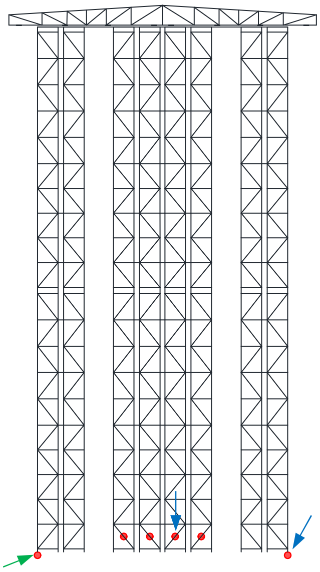
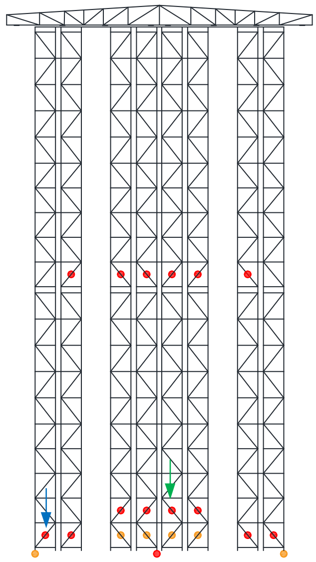
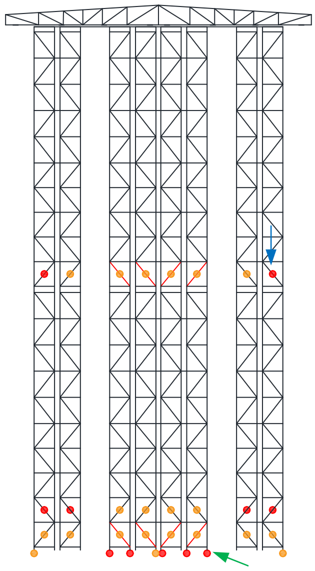
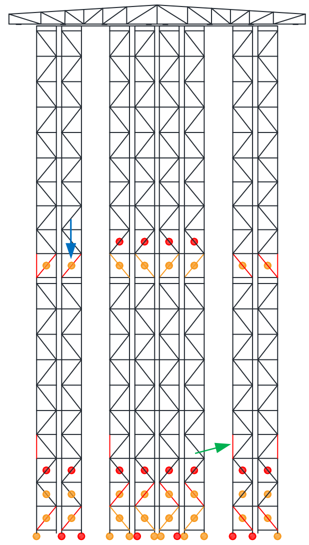
section modulus of uprights sections usually adopted is very low. So, the contribution of bending is relevant and cannot be disregarded.

Along DA direction, the highest D/C ratios are concentrated in the bracing systems, starting from the bottom, and firstly involving diagonal to upright connections and upright base connections. In particular, especially if the same elements are used for both lateral and central bracing systems (CS3 (Table 7-35), CS4 (Table 7-37), and CS5 (Table 7-39)), the central bracing systems are those where the mechanisms characterized by the highest D/C ratios occur. Dealing with diagonal connections, the leading mechanism is most of the time plastic ovalization of the diagonal, or of the plate connecting the diagonal to the upright, where diagonal is bolted. The only case study where shear resistance of bolts is the leading failure is CS2 (Table 7-33). Also for this direction, the leading failure of base connections is due to tensile and shear force on anchors, were the mechanism associated to tensile force is the concrete-cone one, and the mechanism connected to failure of anchor due to shear force. Both these mechanisms are fragile. The immediate consequent criticality occurs in the uprights belonging to the bracing towers, due to axial compression. Unlike uprights belonging to CA frames, the uprights belonging to the bracing system along DA direction, are mainly forced by axial compression/tension due to seismic action, since gravitational pallet load does not act on them. The only gravitational load acting is the dead one due to their own weight, that is not relevant if compared to the one induced by seismic action. All the other elements of the structure, those not belonging to the bracing systems, are characterized by D/C ratios lower than those previously indicated: when the DA bracing system is aligned with the shelves (CS1 and CS2), the elements that are involved after the bracing towers are pallet beams, that work in compression to transfer the forces among the bracing towers, while in case of eccentric bracing system (CS3, CS4 and CS5), the elements that are involved after the bracing systems are the uprights that are closer and directly connected to the bracing system itself, and failure is due to axial compression and bending.

What appears clear by the analysis of the effects of seismic action in both directions of double depth warehouses, is that failure of connections is the one happening first. This is one of the possible consequences of not applying any hierarchy rule in the design of the structure, although a behaviour factor major than 1.5 has been used. This criticality implies that, from the global point of view, the structure has very limited post-elastic sources: if plastic ovalization of diagonal-to-upright connection happens first, these connections would become loose (with a poor dissipative behaviour associated), the lateral deflections of the structure would increase, second order effects may become relevant and cause failure of uprights due to stability issues. If crisis of an upright base connection is the first to happen, this could trigger a series of chain collapses, leading to the collapse of the whole structure (Figure 3-1). In conclusion, the analysis of the case studies highlight that the current design approach can be applied if the structure is designed to remain in the elastic field, considering that, in any case, it is probable that, if crisis in connections are the first that occurs, the whole structure could be involved in the mechanism and be irreparably damaged. If the current design strategies are applied and a dissipative behaviour is expected (a behaviour factor major than 1.5 is adopted), post-elastic sources appear to be very limited, especially if the indications from EN16681 (2016) are applied, suggesting the no need of applying hierarchy rules for the design of the structure for low-dissipative design (behaviour factor between 1.5 and 2).

Table 7-30: Hierarchy of criticalities for CS1: seismic action along CA direction.

Key: ● failure in connection happening in the step; ● failure in connection already happened;
— failure in element happening in the step; — failure in element already happened;
→ first mechanism happening in the step; → last mechanism happening in the step

1st STEP	2nd STEP	3rd STEP	4th STEP
			
1.59 - 1.55	**1.54 - 1.43**	**1.42 - 1.33**	**1.32 - 1.22**
First mechanisms: failure of diagonal connection due to plastic ovalization + failure of base connection in post-installed anchors due to tensile+shear force.	First mechanism: failure of diagonal connection due to plastic ovalization.	First mechanism: failure of diagonal connection due to plastic ovalization.	First mechanism: failure of diagonal element in comrpession (stability).
Last mechanism: failure of base connectors (post-installed anchors) due to tensile+shear force.	Last mechanism: failure of diagonal connection due to plastic ovalization.	Last mechanism: failure of base connectors (post-installed anchors) due to tensile+shear force.	Last mechanism: failure of upright due to axial compressiona and bending (stability).
Note: Only failures of connections involved.	Note: Only failures of connections involved.	Note: Diagonal elements start to buckle (where failure of connection has already happened).	Note: Failure of connections of diagonals spreads, as well as of diagonal elements. Uprights start to buckle.

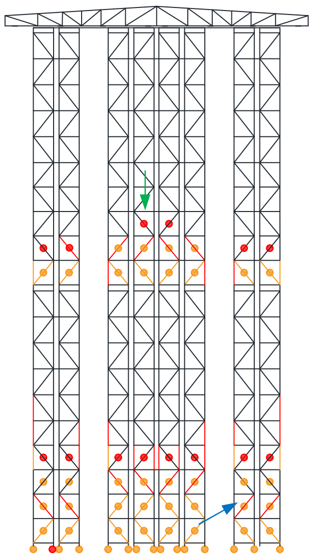
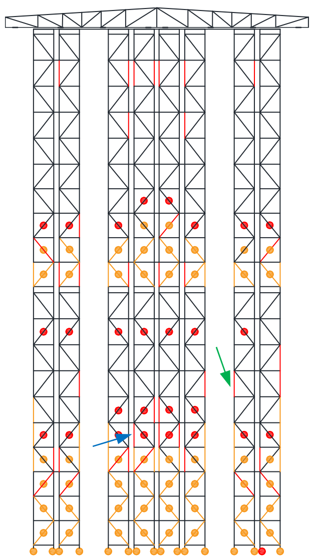
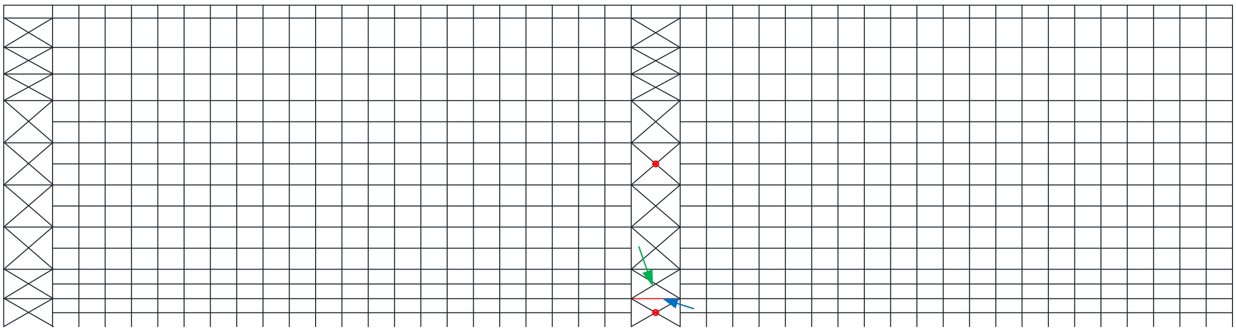
5th STEP		6th STEP	
	**1.21 - 1.10** First mechanism: failure of diagonal element in comrpession (stability). Last mechanism: failure of diagonal connection due to plastic ovalization. Note: failure of connections of diagonals spreads, as well as of diagonal elements and uprights.		**1.08 – 1.00** First mechanism: failure of upright due to axial compressiona and bending (stability). Last mechanism: failure of upright due to axial compression and bending (stability). Note: failure of connections of diagonals spreads, as well as of diagonal elments and uprights. All the base connections reach their ultimate capacity.

Table 7-31: Hierarchy of criticalities for CS1: seismic action along DA direction.

Key: ● failure in connection happening in the step; ● failure in connection already happened;
failure in element happening in the step; failure in element already happened;
first mechanism happening in the step; last mechanism happening in the step

1st STEP



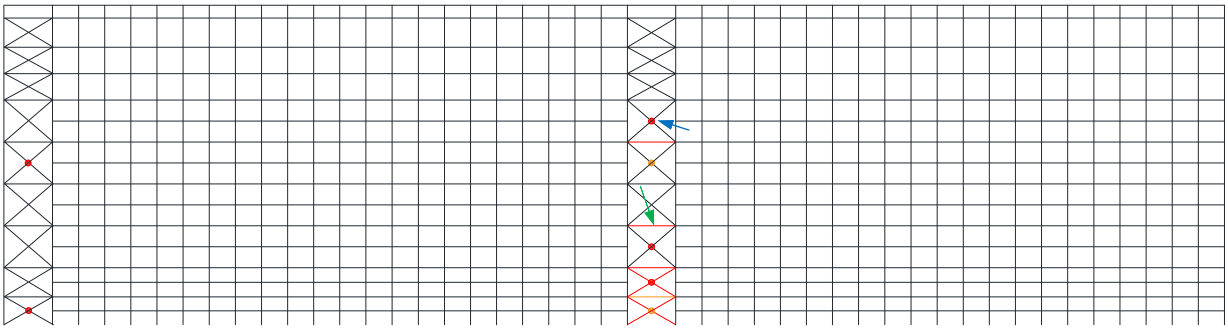
3.00 – 2.60

First mechanisms: failure of diagonal connection due to plastic ovalization (central bracing tower).

Last mechanism: failure of horizontal profile due to axial compression (stability).

Note: Failures concentrated in the central bracing tower.

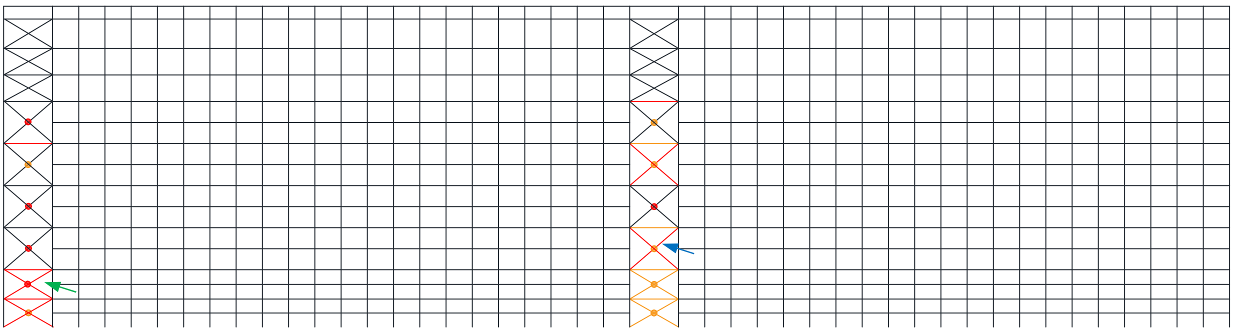
2nd STEP



2.59 – 2.13

First mechanisms: failure of diagonal connection due to plastic ovalization (central bracing tower).
Last mechanism: failure of horizontal profile due to axial compression (stability).
Note: Failures appear also in the lateral bracing tower. In the central bracing tower, braces reach tensile resistance (failure in connection already happened).

3rd STEP



2.12 – 1.81

First mechanisms: failure of diagonal in tension.
Last mechanism: failure of diagonal in tension.
Note: Failures spread in both the bracing towers.

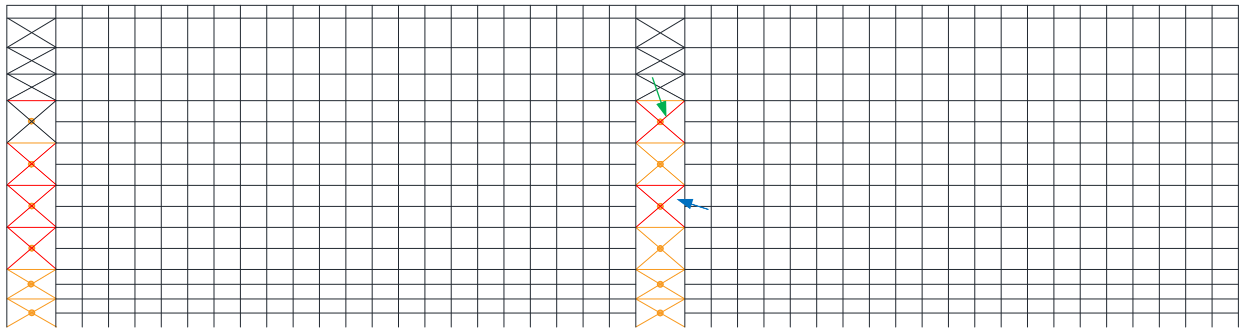
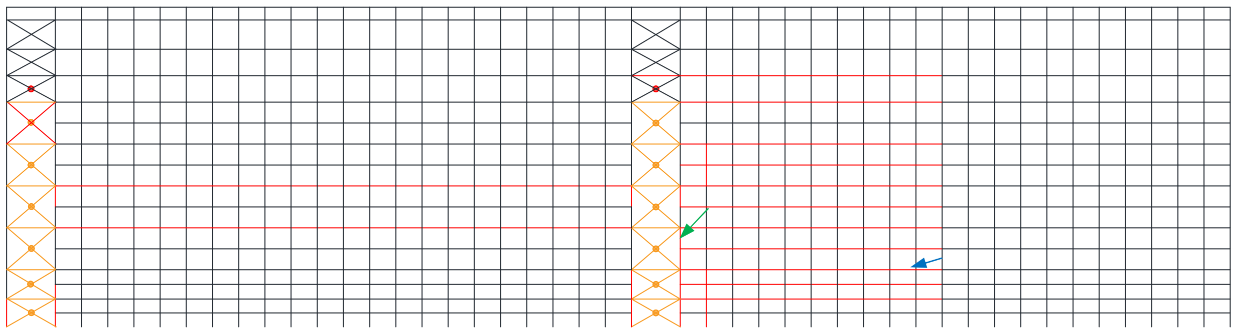
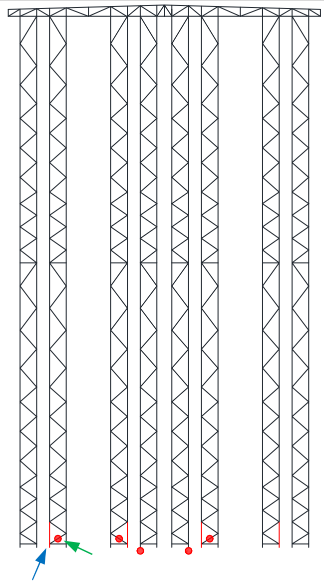
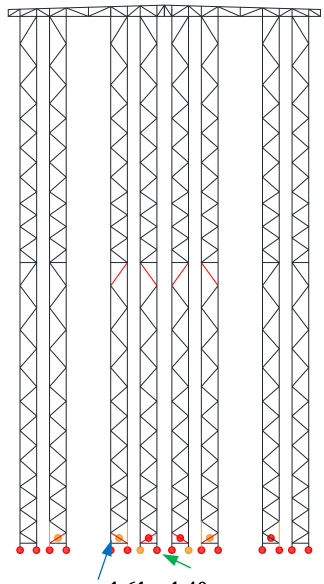
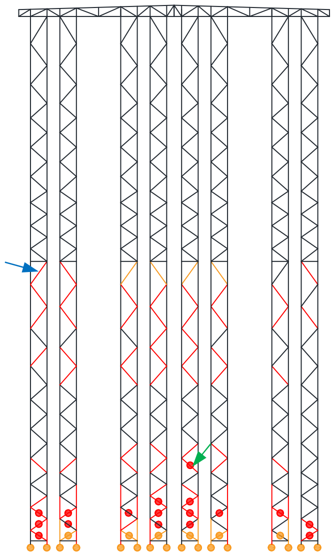
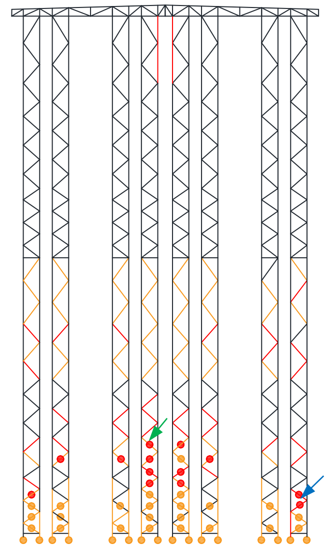
Key: ● failure in connection happening in the step; ● failure in connection already happened; ▬ failure in element happening in the step; ▬ failure in element already happened; → first mechanism happening in the step; → last mechanism happening in the step
4th STEP
 **1.80 – 1.53** First mechanisms: failure of diagonal in tension. Last mechanism: failure of diagonal in tension. Note: Failures spread in both the bracing towers.
5th STEP
 **1.51 – 1.00** First mechanisms: failure of pallet beams due to axial compression (stability). Last mechanism: failure of upright belonging to the central bracing tower due to axial compression (stability). Note: Failure involve both pallet beams and the columns of the bracing towers.

Table 7-32: Hierarchy of criticalities for CS2: seismic action along CA direction.

<table>
<tr><td colspan="4">Key: ● failure in connection happening in the step; ● failure in connection already happened;
▬ failure in element happening in the step; ▬ failure in element already happened;
→ first mechanism happening in the step; → last mechanism happening in the step</td></tr>
<tr><th>1st STEP</th><th>2nd STEP</th><th>3rd STEP</th><th>4th STEP</th></tr>
<tr><td></td><td></td><td></td><td></td></tr>
<tr><td>1.94 – 1.63</td><td>1.61 – 1.40</td><td>1.39 – 1.10</td><td>1.09 – 1.00</td></tr>
<tr><td><u>First mechanisms</u>: failure of upright due to axial compression + bending (stability).

<u>Last mechanism:</u> failure of diagonal connection due to plastic ovalization.

<u>Note:</u> Failure of diagonal connections, base connectors and uprights (at the bottom) are involved.</td><td><u>First mechanism</u>: failure of diagonal element due to axial compression (stability).

<u>Last mechanism:</u> failure of base connectors (post-installed anchors) due to tensile+shear force.

<u>Note</u>: Diagonal elements start to buckle (where failure of connection has already happened).</td><td><u>First mechanism</u>: failure of diagonal element due to axial compression (stability).

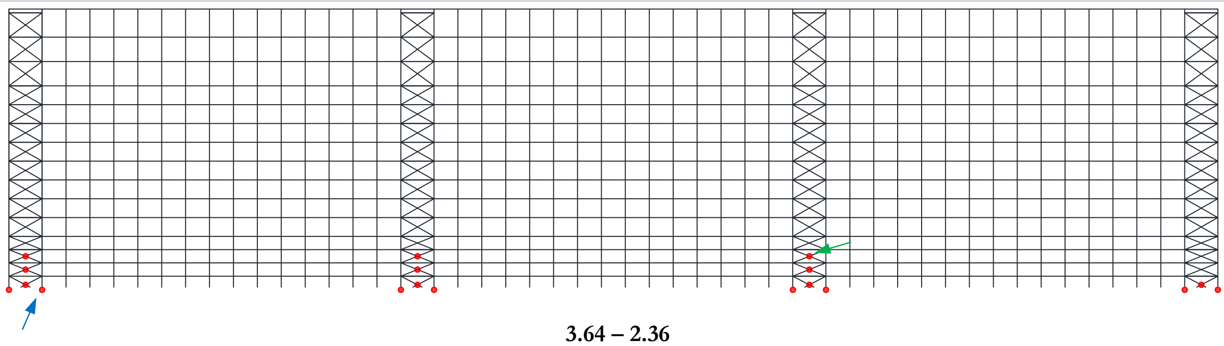
<u>Last mechanism:</u> failure of diagonal connection due to plastic ovalization.

<u>Note</u>: Diagonal elements belonging to higher level start to buckle. Uprights start to buckle.</td><td><u>First mechanism</u>: failure of diagonal element due to axial compression (stability).

<u>Last mechanism</u>: failure of diagonal connection due to plastic ovalization.

<u>Note:</u> Failure in connections of diagonals spreads, as well as in diagonals.</td></tr>
</table>

Table 7-33: Hierarchy of criticalities for CS2: seismic action along DA direction.

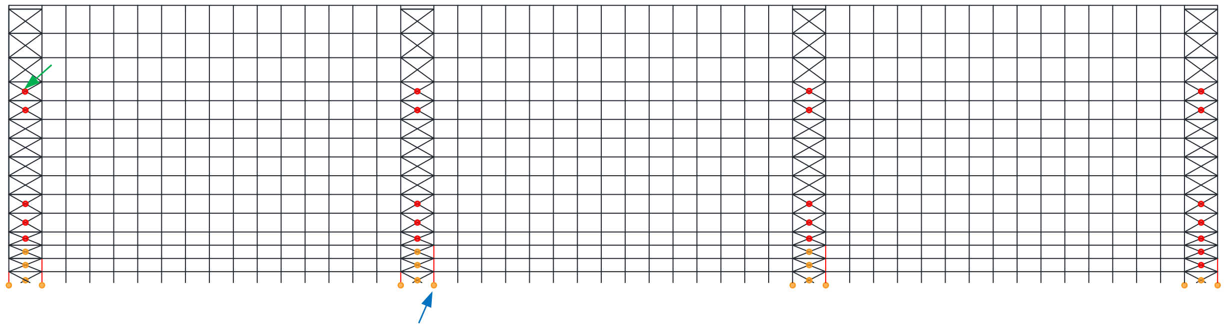
<table>
<tr><td>Key: ● failure in connection happening in the step; ● failure in connection already happened;
▬ failure in element happening in the step; ▬ failure in element already happened;
→ first mechanism happening in the step; → last mechanism happening in the step</td></tr>
<tr><th>1st STEP</th></tr>
<tr><td></td></tr>
<tr><td>3.64 – 2.36</td></tr>
<tr><td><u>First mechanisms</u>: failure of base connectors (post-installed anchors) due to tensile+shear force.

<u>Last mechanism:</u> failure of diagonal connection due to shear force (bolted connection).

<u>Note:</u> Only failures of connections are involved.</td></tr>
</table>

Key: failure in connection happening in the step; failure in connection already happened;
failure in element happening in the step; failure in element already happened;
first mechanism happening in the step; last mechanism happening in the step

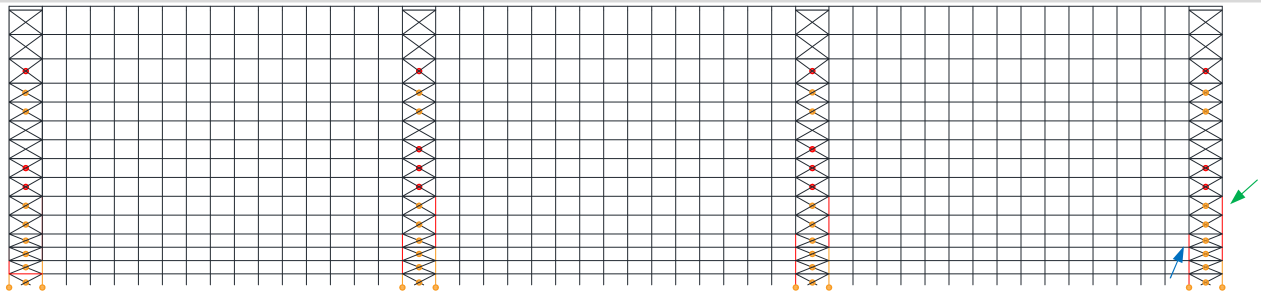
2nd STEP



2.35 – 1.82

First mechanisms: failure of upright due to axial compression (stability).
Last mechanism: failure of diagonal connection due to shear force (bolted connection).
Note: Failure of connections spread along the bracing towers. Also uprights start to bukle.

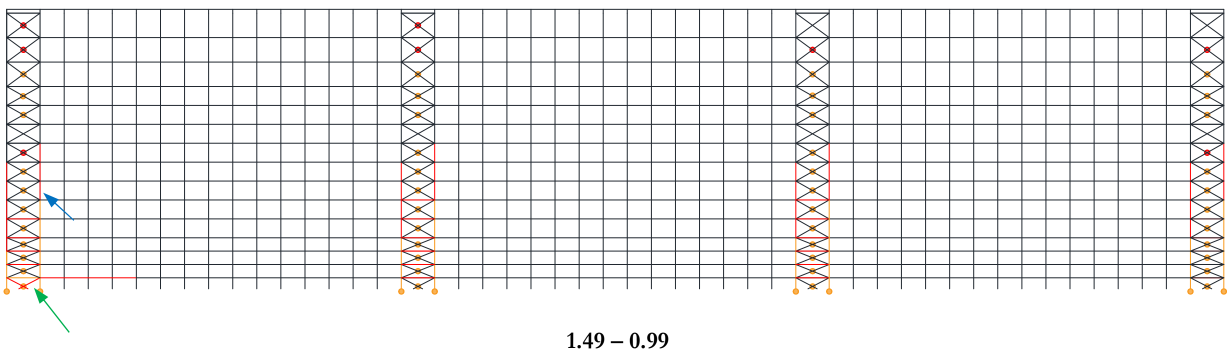
3rd STEP



1.81 – 1.50

First mechanisms: failure of upright due to axial compression (stability).
Last mechanism: failure of upright due to axial compression (stability).
Note: Failures spread in both the bracing towers, involving both connections of diagonals and uprights (diagonal elements are still not involved).

4th STEP



1.49 – 0.99

First mechanisms: failure of upright due to axial compression (stability).
Last mechanism: failure of diagonal in tension.
Note: Failure of diagonal in tension is the last one reached.

Table 7-34: Hierarchy of criticalities for CS3: seismic action along CA direction.

Key: ● failure in connection happening in the step; ● failure in connection already happened;
failure in element happening in the step; failure in element already happened;
first mechanism happening in the step; last mechanism happening in the step

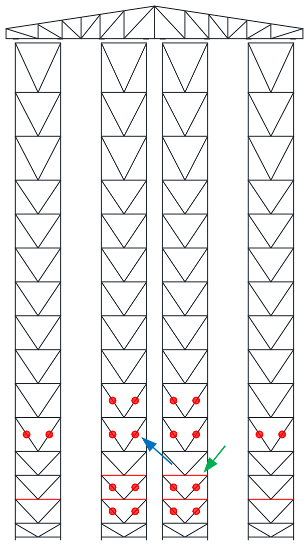
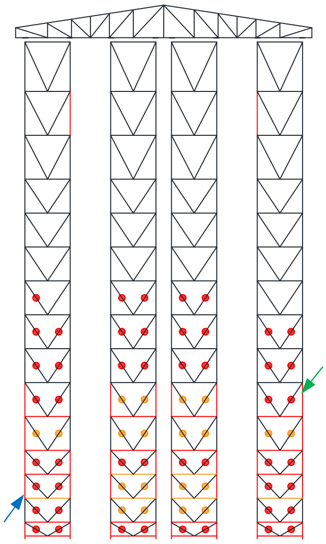
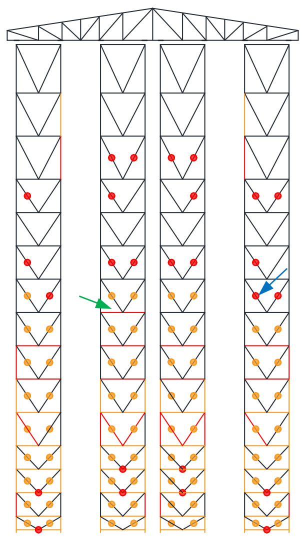
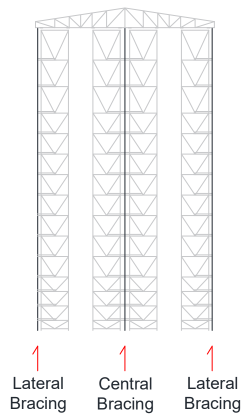
1st STEP	2nd STEP	3rd STEP
		
1.50 – 1.42	**1.41 – 1.10**	**1.09 – 1.00**
First mechanisms: failure of diagonal connection due to plastic ovalization. Last mechanism: failure of horizontal element due to axial compression (stability). Note: Failure of diagonal connections, and horizontal elements are involved.	First mechanism: failure of upright due to axial compression + bending (stability). Last mechanism: failure of diagonal connection due to plastic ovalization. Note: Upright start to buckle.	First mechanism: failure of diagonal connection due to plastic ovalization. Last mechanism: failure of horizontal element due to axial compression (stability). Note: Failure of diagonal connections, horizontal elements ad uprights spread along the height of the structure.

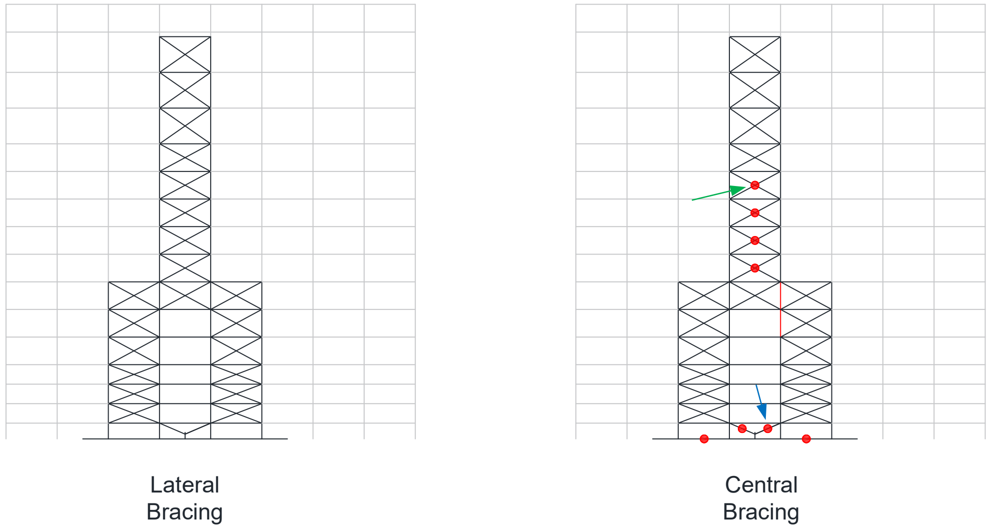
Table 7-35: Hierarchy of criticalities for CS3: seismic action along DA direction.

Key: ● failure in connection happening in the step; ● failure in connection already happened;
failure in element happening in the step; failure in element already happened;
first mechanism happening in the step; last mechanism happening in the step



Key: failure in connection happening in the step; failure in connection already happened;
failure in element happening in the step; failure in element already happened;
first mechanism happening in the step; last mechanism happening in the step

1st STEP



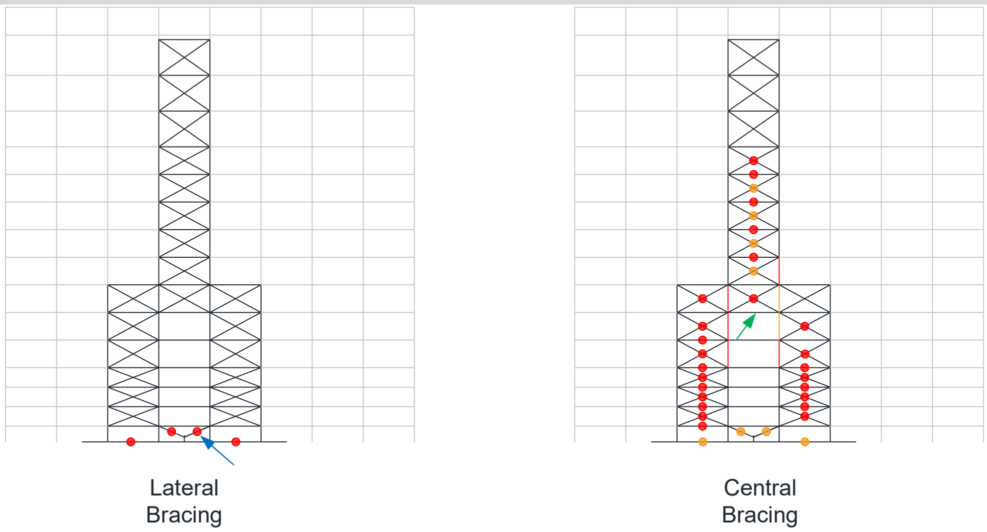
4.29 – 3.31

First mechanisms: failure of diagonal connection due to plastic ovalization (central bracing tower).

Last mechanism: failure of diagonal connection due to plastic ovalization (central bracing tower).

Note: Failures mainly concentrated in the central bracing tower.

2nd STEP



3.14 - 2.11

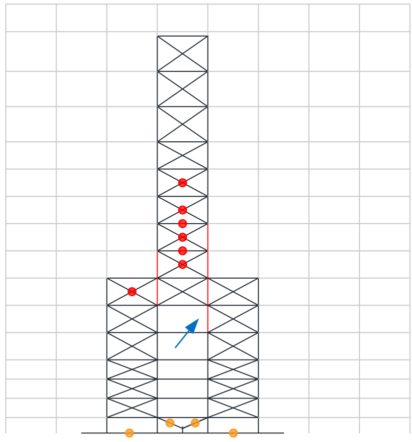
First mechanisms: failure of diagonal connection due to plastic ovalization (lateral bracing tower).

Last mechanism: failure of diagonal connection due to plastic ovalization (central bracing tower).

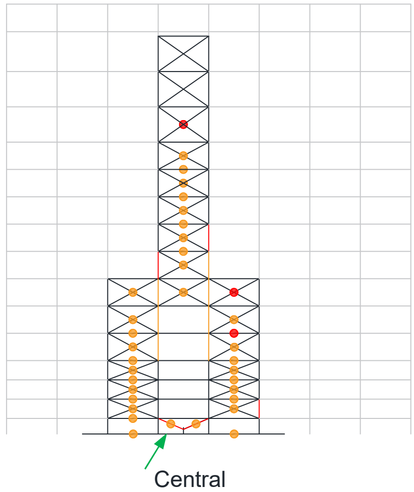
Note: Failures of diagonal connections spread in the central bracing tower, uprights start to buckle.

Key: ● failure in connection happening in the step; ● failure in connection already happened;
failure in element happening in the step; failure in element already happened;
first mechanism happening in the step; last mechanism happening in the step

3rd STEP



Lateral Bracing



Central Bracing

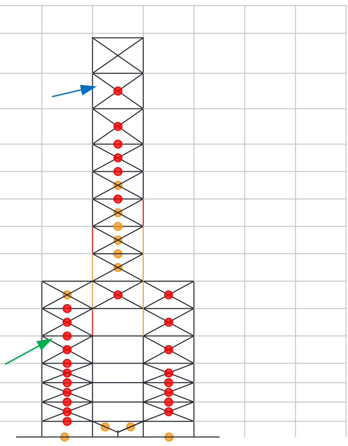
2.11 – 1.71

First mechanisms: failure of upright due to axial compression (stability).

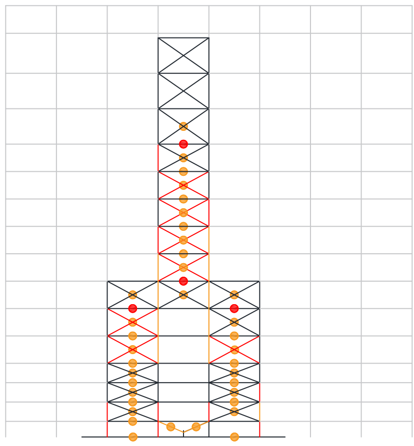
Last mechanism: failure of diagonal in tension.

Note: Failure of diagonal in tension (at the bottom of the central bracing tower) happens after most of the diagonal connections reached ultimate strength.

4th STEP



Lateral Bracing



Central Bracing

1.70 – 1.24

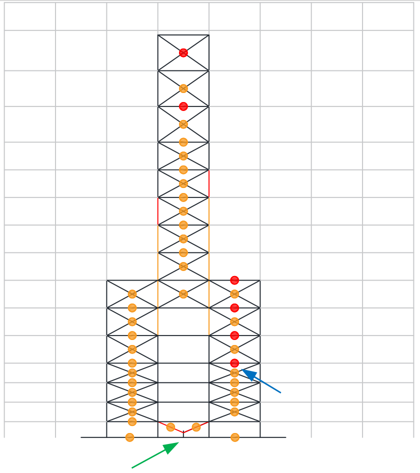
First mechanisms: failure of diagonal connection due to plastic ovalization (lateral bracing tower).

Last mechanism: failure of connection horizontal element due to plastic ovalization (lateral bracing tower).

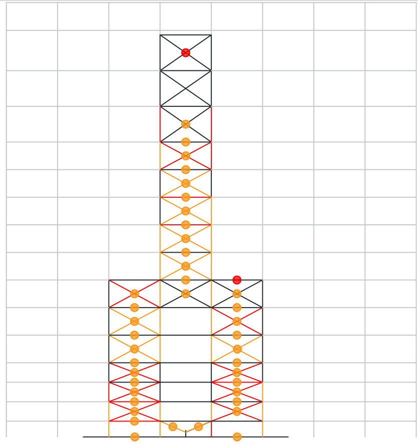
Note: Failures of diagoanl connections spread also in the lateral bracing tower.

Key: ● failure in connection happening in the step; ● failure in connection already happened;
failure in element happening in the step; failure in element already happened;
first mechanism happening in the step; last mechanism happening in the step

5th STEP



Lateral Bracing



Central Bracing

1.23 – 1.00

First mechanisms: failure of connection horizontal element due to plastic ovalization (lateral bracing tower).

Last mechanism: failure of diagonal at the bottom of the lateral bracing tower in tension.

Note: Diagonal elements of the central bracing tower reach ultimate strength in tension, but failure of connection already happened.

Table 7-36: Hierarchy of criticalities for CS4: seismic action along CA direction.

Key: ● failure in connection happening in the step; ● failure in connection already happened;
failure in element happening in the step; failure in element already happened;
first mechanism happening in the step; last mechanism happening in the step

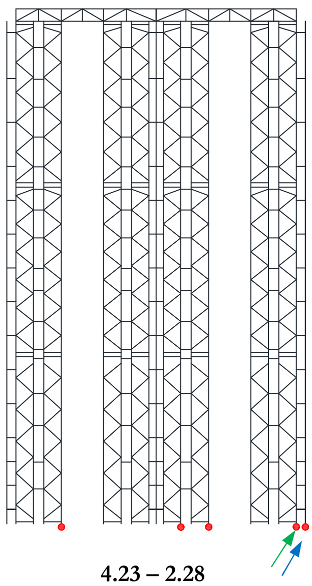
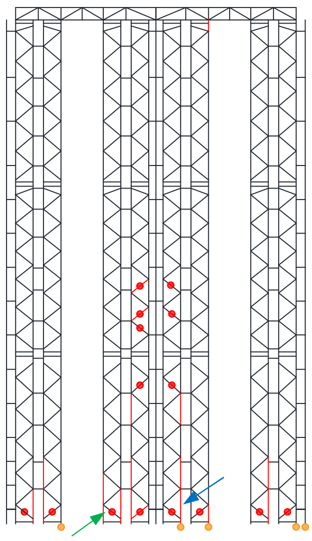
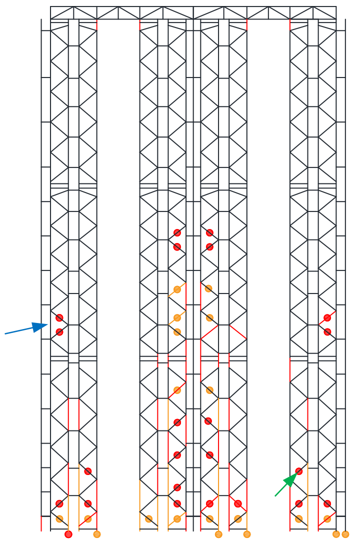
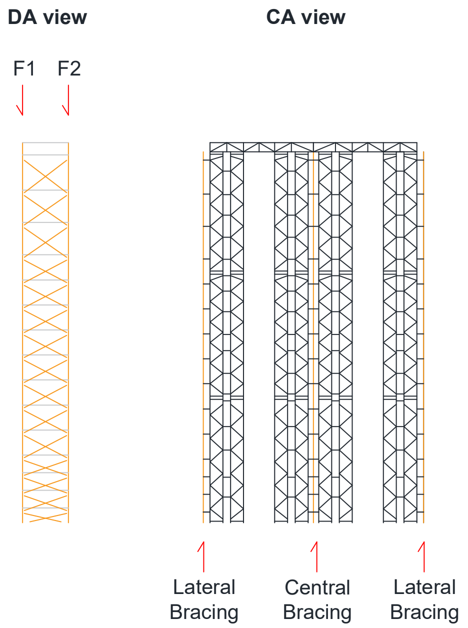
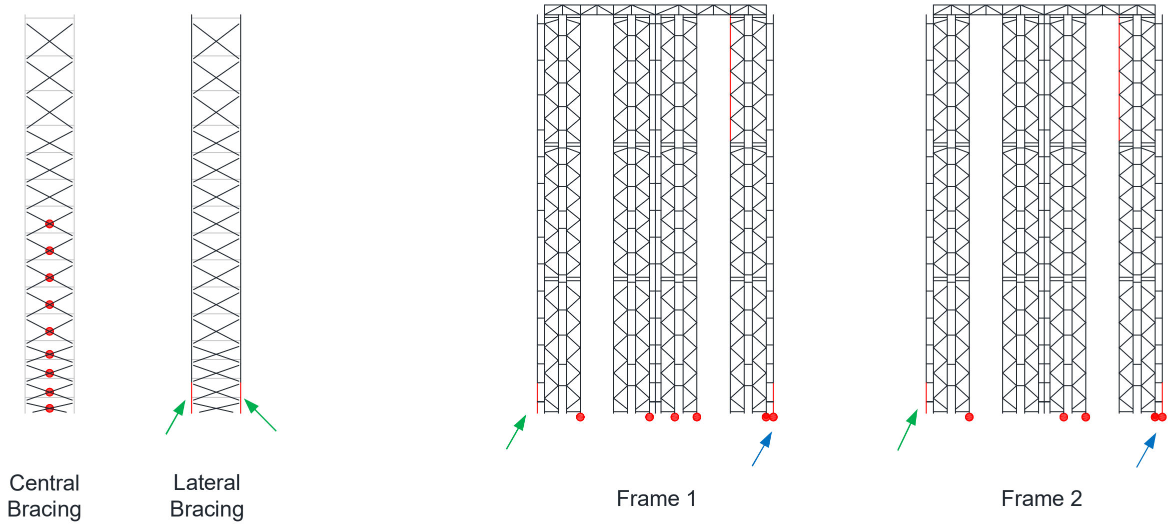
1st STEP	**2nd STEP**	**3rd STEP**
		
4.23 – 2.28	**2.25 – 1.32**	**1.32 – 1.00**
First mechanisms: failure of base connectors (post-installed anchors) due to tensile+shear force.	First mechanism: failure of upright due to axial compression + bending (stability).	First mechanism: failure of diagonal connection due to plastic ovalization.
Last mechanism: failure of base connectors (post-installed anchors) due to tensile+shear force.	Last mechanism: failure of upright due to axial compression + bending (stability).	Last mechanism: failure of diagonal connection due to plastic ovalization.
Note: Only failures of connections are involved.	Note: Diagonal elements start to buckle (failure of connection already happened), as well es uprights (at the bottom).	Note: Failure of diagonal connections, diagonal elements and uprights spread along the height of the structure.

Table 7-37: Hierarchy of criticalities for CS4: seismic action along DA direction.

Key: ● failure in connection happening in the step; ● failure in connection already happened;
failure in element happening in the step; failure in element already happened;
first mechanism happening in the step; last mechanism happening in the step



1st STEP



8.77 – 2.33

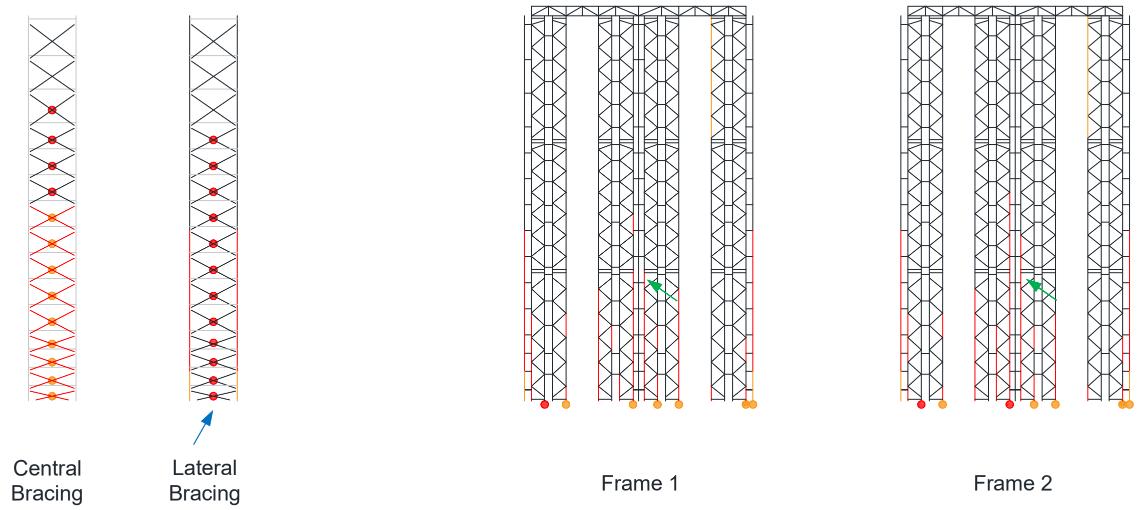
First mechanisms: failure of base connectors (post-installed anchors) due to tensile+shear force (upright belonging to the lateral bracing tower).

Last mechanism: failure of upright due to axial compression (stability, upright belonging to the lateral bracing tower).

Note: Failures mainly concentrated in the central bracing tower, where most of diagonal connections reach ultimate resistance (plastic ovalization). Many uprights base connections fail, due to tensile+shear force on anchors.

Key: failure in connection happening in the step; failure in connection already happened;
failure in element happening in the step; failure in element already happened;
first mechanism happening in the step; last mechanism happening in the step

2nd STEP



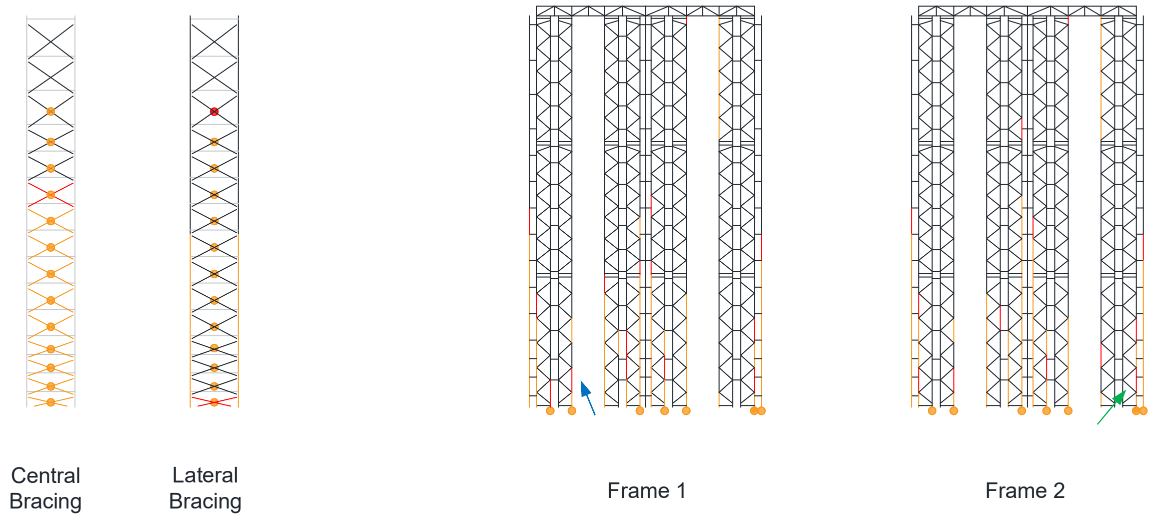
2.21 – 1.11

First mechanisms: failure of diagonal connection due to plastic ovalization (lateral bracing tower).

Last mechanism: failure of upright due to tension + bending..

Note: Failures of diagonal connections spread in the central and in the lateral bracing tower, uprights start to buckle, and diagonal reach their tensile resistance (failure of connection has already happened).

3rd STEP



1.10 – 1.00

First mechanisms: failure of upright due to axial compression and bending (stability).

Last mechanism: failure of upright due to axial compression and bending (stability).

Note: Uprights not belonging to the bracing tower are the most involved in this step.

Table 7-38: Hierarchy of criticalities for CS5: seismic action along CA direction.

Key: failure in connection happening in the step; failure in connection already happened;
failure in element happening in the step; failure in element already happened;
first mechanism happening in the step; last mechanism happening in the step

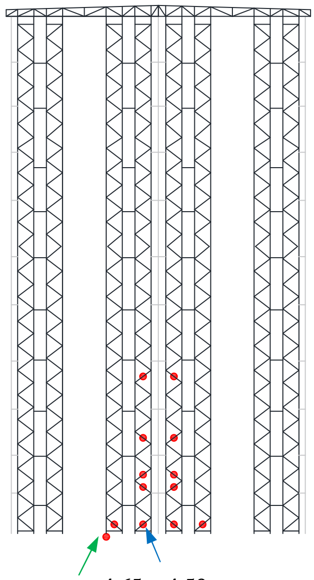
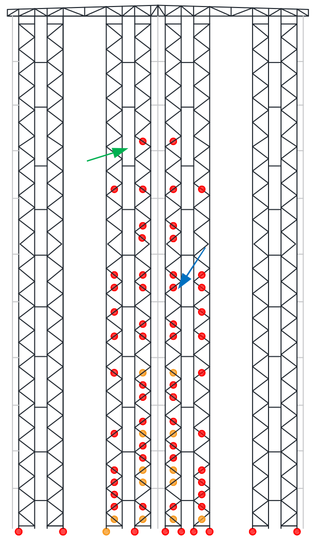
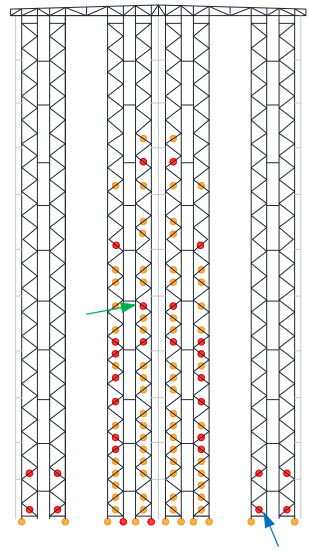
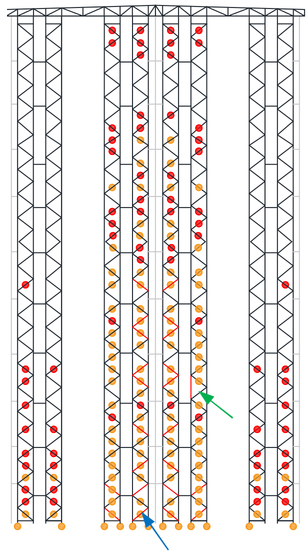
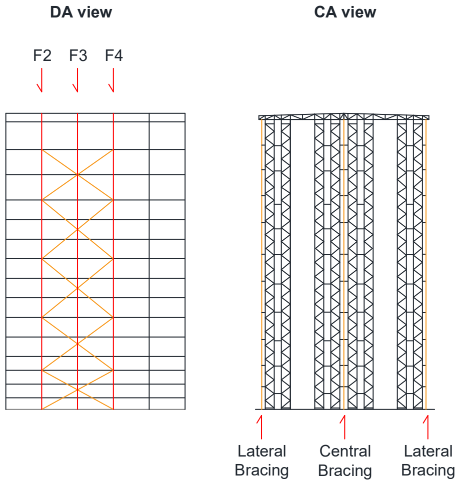
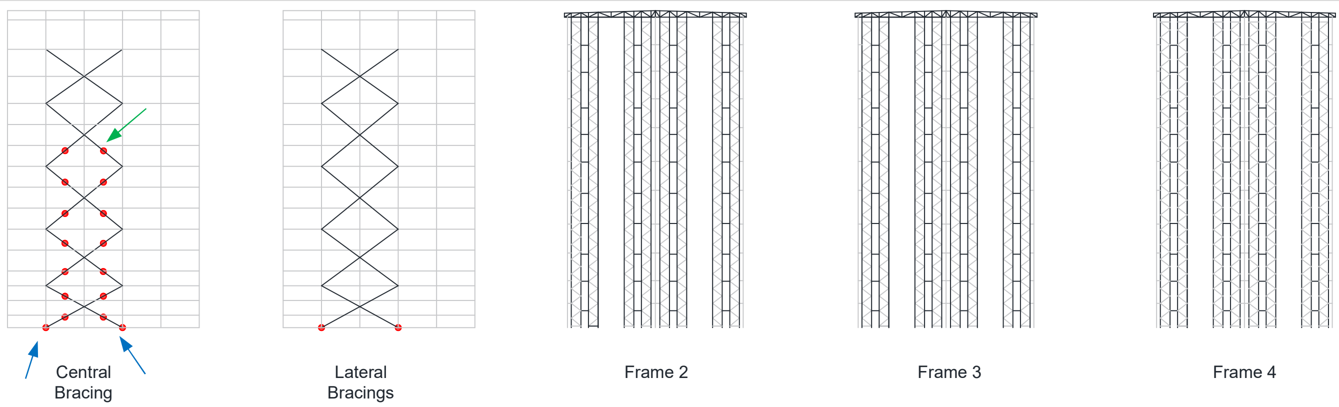
1st STEP	2nd STEP	3rd STEP	4th STEP
			
4.65 – 4.58	**4.52 – 2.25**	**2.23 – 1.72**	**1.71 – 1.00**
First mechanisms: failure of diagonal connection due to plastic ovalization.	First mechanism: failure of diagonal connection due to plastic ovalization.	First mechanism: failure of diagonal connection due to plastic ovalization.	First mechanism: failure of diagonal element due to axial compression (stability).
Last mechanism: failure of base connectors (post-installed anchors) due to tensile+shear force.	Last mechanism: failure of diagonal connection due to plastic ovalization.	Last mechanism: failure of diagonal connection due to plastic ovalization.	Last mechanism: failure of upright due to axial compression + bending (stability).
Note: Failure of connections are the only involved.	Note: Failure of connections are the only involved. The central shelve is the main involved.	Note: Failure of connections are the only involved. The central shelve is the main involved. Most of the base connections fail due to tensile+shear force on post-installed anchors.	Note: Failure of diagonal elements start to occur (ultimate resitance of connection already reached).

Table 7-39: Hierarchy of criticalities for CS5: seismic action along DA direction.

Key: ● failure in connection happening in the step; ● failure in connection already happened;
▬ failure in element happening in the step; ▬ failure in element already happened;
→ first mechanism happening in the step; → last mechanism happening in the step



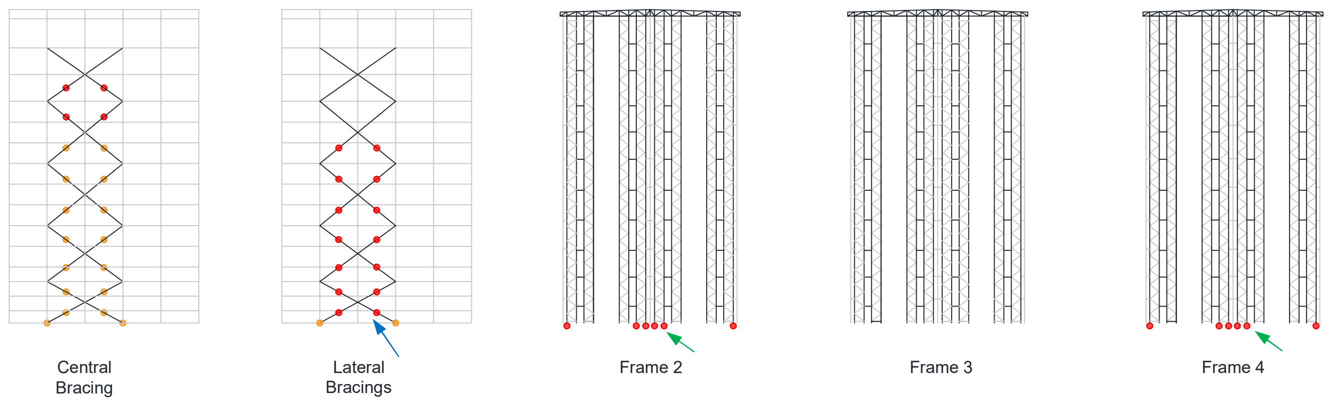
1st STEP



9.00 – 8.73

First mechanisms: failure of base connectors (post-installed anchors) due to tensile+shear force.
Last mechanism: failure of diagonal connection due to plastic ovalization.
Note: Only failures of connections are involved.

2nd STEP

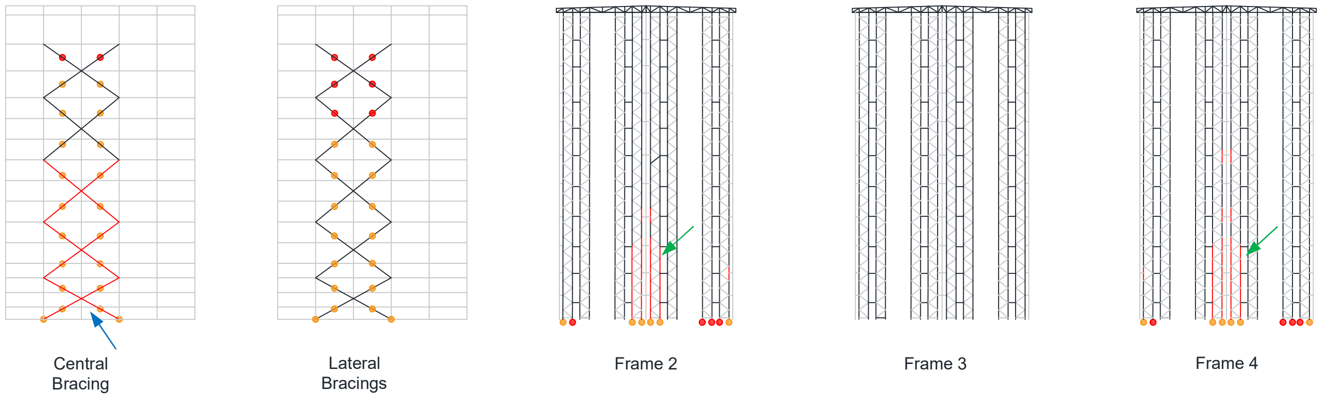


6.71 – 3.37

First mechanisms: failure of diagonal connection due to plastic ovalization.
Last mechanism: failure of base connectors (post-installed anchors) due to tensile+shear force.
Note: Failure of connections spread along the bracing towers. Base connections of the uprights not belonging to the bracing towers start to fail due to tensile + shear force on post-installed anchors.

Key: failure in connection happening in the step; failure in connection already happened;
failure in element happening in the step; failure in element already happened;
first mechanism happening in the step; last mechanism happening in the step

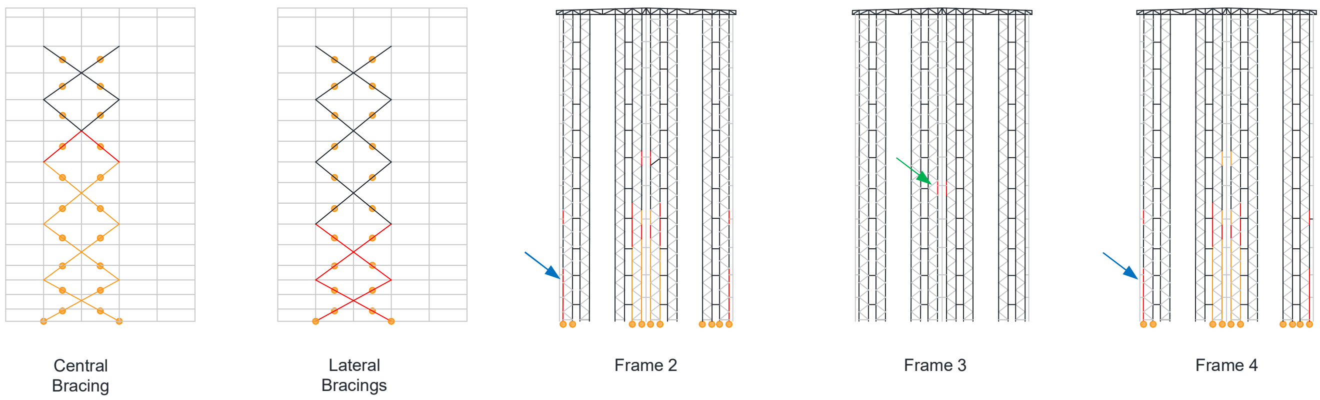
3rd STEP



2.93 – 1.32

First mechanisms: failure of diagonal element due to tensile force.
Last mechanism: failure of upright due to axial compression + bending (stability).
Note: Failures spread in both the bracing towers, involving both connections of diagonals and diagonal elements. Uprights not belonging to the bracing towers start to buckle.

4th STEP



1.31 – 1.00

First mechanisms: failure of upright due to axial compression + bending (stability).
Last mechanism: failure of upright due to axial compression + bending (stability).
Note: Falure spreads in urpights not belonging to the bracing towers.

7.5. Concluding remarks

In this chapter, the structural assessment of 5 double depth ARSWs has been carried out. These structures are designed by five big European companies that nowadays design and produce ARSWs. The final aims of this study are to comprehend the current design strategies, structural choices, technological features and resulting structural behaviour. The design of the case studies has been performed in the framework of STEELWAR research project, where the common input design parameters have been fixed (§7.1) in order to have comparable structures. These common parameters can be summarized in the following ones:

(i) input geometry parameters (i.e. global dimensions as width, length and height of the whole warehouse) and number of defining components (i.e. number of upright lines, number of shelves – coupled uprights - and number of aisles);

(ii) number of pallets per load level and relative characteristic (weight and dimensions);
(iii) the design force input, both in static and seismic conditions;
(iv) load combination factors and definition of participating mass.

The designers can freely choose the structural type for the two main directions of the structure, the dimensions of the structural model (2D or 3D), the profiles to be used for the main structural elements (cross-sections, steel grade), the technological solutions to adopt, and all the other possible design parameters that that they think that is correct to consider according to their experience.

From the analysis of the configurations and of the structural choices adopted, it seems that the main path that guides all these structural choices is structural optimization, which aims to balance the structural needs with: (i) limiting the costs connected to the necessary amount of steel and to the additional processes at workshops (i.e. welds or additional sheets for connections are very limited); (ii) for the same element (i.e. diagonals, uprights, pallet beams), limiting the number of different cross-sections needed. Similar technological solutions are used for the same structural element and connection.

Dealing with the analysis of the design parameters to be freely adopted, there are many having a relevant impact on the definition of the design response spectrum and so on the value of the seismic design force. In fact, the seismic design force can significantly vary, also for the same starting design hypothesis and for the same structural type.

As regards the definition of the design response spectrum, besides the behaviour factor, there is the K_d factor (EN16681 (2016), equation (7-III)), that takes into account the capability of pallets to dissipate energy through their movement on the pallet beams when friction is overcome. In particular, a decrease of the seismic acceleration up to 46% corresponds to the assumption of K_d equal to 0.8 and q-factor equal to 1.5, while a decrease up to 60% can be reached adopting a q-factor equal to 2.0.

Dealing with the definition of the seismic mass (§7.1, equation (7-I)), if both Ψ_2 and RF factors are assumed, in line with what indicated within EN16681 (2016), a reduction of the seismic mass up to 20% is obtained along CA direction and up to 46% is obtained along DA direction. This assumption directly affects the size of the total base shear due to seismic action, which is reduced by this reduction of mass, but is also increased due to the increment of the seismic acceleration (since, assuming the same stiffness, the mass decreases, and so also the fundamental period decreases, too). As a consequence, it is not possible to say *a priori* if these assumptions finally determine an increase or a decrease of the design seismic base share.

Finally, along CA direction, it is possible to take into consideration the reduction of the lateral shear stiffness of the frames due to the eccentricity of the diagonal-to-upright connections with respect to the centroid of the uprights. The size of the reduction of this lateral stiffness can be determined by the execution of shear experimental tests on the shelves constituting the structure. This assumption basically can make the natural period of the structure (along CA direction) increase also up to 30-40%, being the structure more flexible, and determining a reduction of the seismic acceleration.

Table 7-23 gathers the effects of these parameters on the value of the design base shear (the definition of the seismic participant mass is not considered, since it is not possible to say *a priori* if these assumptions finally determine an increase or a decrease of the design seismic base share). It can be noticed that, in some cases, quite high reductions are reached (up to 60%), and could get worsen in case that also the reduction of the participant mass would be not conservative. In general, it is necessary to evaluate if these assumptions, that are justified for traditional racks, are also suitable for ARSWs or if they actually lead to an unsafe and not conservative design. Besides, it seems that a universally accepted and shared guideline about the possibility of using these parameters is missing, and this lack implies a great variability of the value of the design seismic force, although the structural type is the same.

Dealing with the vulnerability assessment, the results of the safety check executed on the main components of the structure are organized in the so called "hierarchy of criticalities", where the weakest parts of the structure and the chain of mechanisms are individuated by putting in order (form the highest to the lowest) the demand-capacity ratios.

Referring to the hierarchy of criticalities corresponding to seismic action along CA direction, a similar behaviour can be individuated among all the case studies: the components that are characterized by the highest D/C ratios are diagonal connections and uprights base connections. The leading mechanism for diagonal connections is plastic ovalization, while the leading one for base connections is failure due to tensile and shear force on anchors, were the mechanism associated to tensile force is the concrete-cone one, and the mechanism connected to shear force is failure of anchor. Failure of base connections is relevant in all the case studies where anchors used are the post-installed ones. The only case study where base connections do not fail first is CS3, where traditional base connections with threaded bars have been chosen, allowing better performances. As regards diagonal elements, it can be noticed that the ultimate resistance of the element (both tensile and in compression) is always higher that the resistance of the connection (at least 40% higher). This happens because, according to EN16681 (2016), if behaviour factors up to 2 are used, no hierarchy design rules are mandatory to be applied. The only request is to avoid a fragile failure of connection by having the shear resistance of bolts at least 1.20 times higher than the plastic ovalization. This design strategy implies that, although fragile failure of connection is prevented, no over-resistance is provided to connection with respect to diagonal element, and connections are designed directly with the reduced design forces from numerical analysis. Considering that diagonals are directly bolted to uprights, and that these elements are characterized by very low thicknesses, the leading mechanism (the one characterized by the lower resistance) is the plastic ovalization.

Along DA direction, the highest D/C ratios are concentrated in the bracing systems, starting from the bottom, and firstly involving diagonal to upright connections and upright base connections. Dealing with diagonal connections, the leading mechanism is most of the time plastic ovalization of the diagonal, or of the plate connecting the diagonal to the upright. Also for this direction, the leading failure of base connections is due to tensile and shear force on anchors, were the mechanism associated to tensile force is the concrete-cone one, and the mechanism connected to failure of anchor due to shear force. Both these mechanisms are fragile. The immediate consequent criticality occurs in the uprights belonging to the bracing towers, due to axial compression.

What appears clear by the analysis of the effects of seismic action in both directions of double depth warehouses is that failure of connections is the one happening first. This is one of the possible consequences of not applying any hierarchy rule in the design of the structure, although a behaviour factor major than 1.5 has been used. This criticality implies that, from the global point of view, the structure has very limited post-elastic sources: if plastic ovalization of diagonal-to-upright connection happens first, these connections would become loose (with a poor dissipative behaviour associated), the lateral deflections of the structure would increase, second order effects may become relevant and cause failure of uprights due to stability issues. If crisis of an upright base connection is the first to happen, this could trigger a series of chain collapses, leading to the collapse of the whole structure (Figure 3-1). In conclusion, the analysis of the case studies highlights that the current design approach can be applied if the structure is designed to remain in the elastic field. Anyway, it is probable that, if crisis in connections are the first occurring, the whole structure could be involved in the mechanism and be irreparably damaged. If the current design strategies are applied and a dissipative behaviour is expected (a behaviour factor major than 1.5 is adopted), post-elastic sources appear to be very limited, especially if the indications from EN16681 (2016) are applied, suggesting the no need of applying hierarchy rules for the design of the structure for low-dissipative design (behaviour factor between 1.5 and 2).

8. Design optimization

In this chapter, the possible optimization of the design of double depth warehouses is proposed. In the framework of this thesis, primary attention is focused on CA direction, which looks very peculiar and atypical with respect to the configurations of DA direction.

Based on the results obtained from the analysis of the case studies (§7.5), the main criticalities for cross-aisle direction can be summarized in the followings:

- The first failures appear to be in base connections when made with post-installed anchors. In particular, the leading failure is due to tensile and shear force on anchors. The mechanism associated with tensile force is the concrete-cone one, and the one connected to shear force is anchor’s failure. Both of these mechanisms are fragile, and should be prevented, avoided and anticipated by ductile mechanisms.
- The contemporary or immediately consequent failure involves the diagonal-to-upright connection. This connection is always realized by connecting the diagonal directly to upright through one or more bolts. The lowest resistance of connection is always plastic ovalization (diagonal side), and it is mainly due to the reduced thickness of the diagonal's cross-section. This kind of mechanism is not fragile, but, in any case, it cannot be fully trusted as the main source of dissipation with the perspective of designing these structures to be dissipative.
- The consequent criticality that occurs is buckling failure of uprights at the bottom of the structure, or where the reinforcement stops, if the reinforcement provides sufficient resistance. The highest D/C ratios are obtained for combined axial compression and bending. Axial compression force acting on uprights is very high, considering that the first relevant rate is due to the gravitational loads (weight of the goods), and the second one is due to seismic action. Form capacity side, a good resistance of uprights in compression is obtained, adopting different strategies (as external reinforcement or local reinforcement) to control global and local buckling phenomena. A significant rate of D/C ratio is due to bending, since, although bending force is moderate, the section modulus of the adopted uprights is very low. So, the contribution of bending is relevant and cannot be disregarded.

In summary, connections are the first to fail, involving fragile mechanisms (in case of base connections) or mechanisms with limited and hardly reliable post-elastic sources (referring to plastic ovalization of diagonal-to-upright connections).

The new proposal concerns the possibility to design the structure as dissipative. With this purpose, the most suitable structural type is individuated by performing a design optimization from a global and a local perspective. The selected scheme is the X-shaped one, where diagonals are the dissipative elements, and all the others are designed to be over-resistant. Given that the lower part of the structure is the most exploited one, in contrast with the upper one, in this approach the lower part only is included in the plastic mechanism, while the higher remains in the elastic field. The design of the lower part is carried out by applying Eurocode 8 prescriptions for the medium ductility class (prEN 1998:2019). All the structural choices are made always thinking about the necessity to make this proposal applicable to the market. This implies that the same technical features and cross-section shapes are adopted.

The assessment of the method is performed through the execution of NLTH analyses. Still, the proposal should be validated and supported by an experimental campaign that should provide the exact characterization of the dissipative elements' structural behaviour, which is only numerically determined. This is one of the more relevant future developments of this work.

8.1. Aims of the new design approach, starting design hypothesis and applied method

The new design approach aims to develop a method that guarantees the resistance and ductility request at Ultimate Limit State (ULS). In particular, the possibility of designing the structure as dissipative is investigated, preventing fragile failures to occur first, but always considering the current technological solutions (profiles cross-sections mainly used, types of connections), and the cost-benefit ratio. Concerning the collapse mechanism, from the analysis of the hierarchy of criticalities, the more exploited part of the structure is the lower one, where higher forces occur. This happens because the sections of the structural elements are not optimized at all the levels, aiming to a standardized production of profiles (at most, they chance once at the middle height of the system, to better fit the demand in the upper part of the structure). With this philosophy, aiming to a global collapse mechanism for these structures may not be the right approach. This is why alternative yielding patterns are investigated, where only a portion of the structure is involved (the lower), while the higher remains in the elastic field (Figure 8-1).

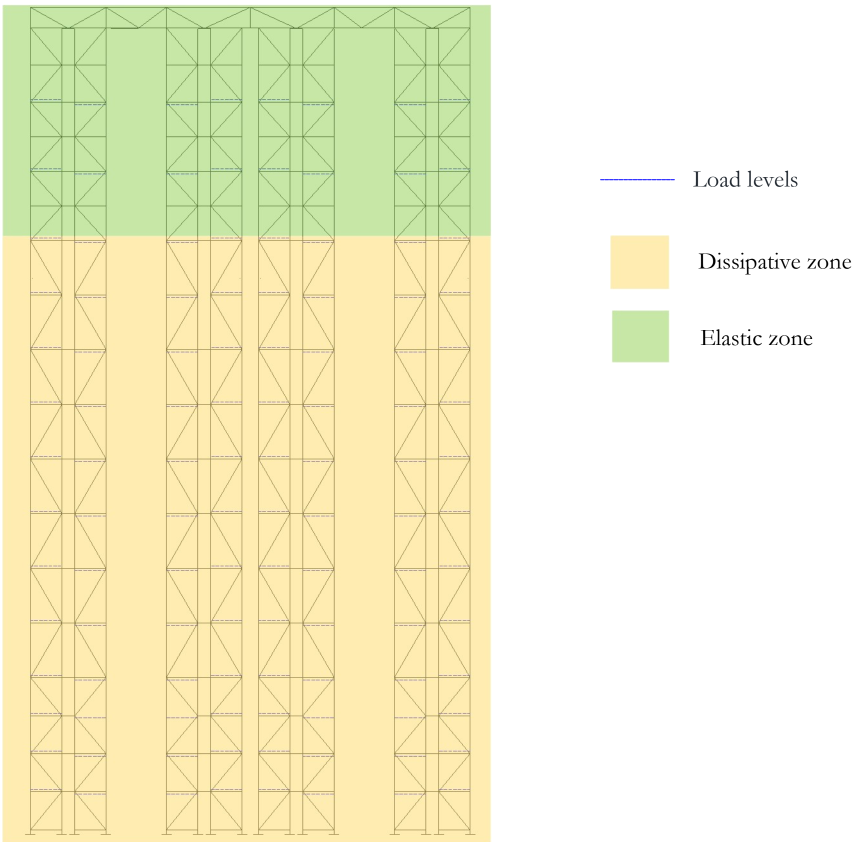


Figure 8-1: Alternative yielding pattern for CA frames of double depth warehouses.

In general, the studies around the development of the new design strategy are organized in the following steps:

- A global optimization of the structure from the geometrical point of view is carried out to reduce possible eccentricities;

- From the global point of view, an optimization of the structural scheme is performed to identify the structural type that allows reaching the desired dissipative structural behaviour;
- From the local point of view, optimization at local level is required to guarantee a sufficient over-resistance of connections of dissipative elements with respect to the dissipative ones. Eurocodes prescriptions are adopted as starting direction for the design of components.

The structural assessment of the designed structure is made through the execution of NLTH analyses, considering the cyclic behaviour of the dissipating elements.

The global optimization of the structure from the geometrical point of view is made as the first design step to reduce all the possible eccentricities that may negatively affect the distribution of forces in the main elements. The overall geometry is kept the same that has been adopted for the design of the case studies. The placement of load levels and the shelves' geometry are defined to reduce to the minimum the distance between the pallet beam-to-upright connection (where the weight of the pallets is transmitted to uprights) and the diagonal-to-upright connections.

The second design step deals with optimising the structure's design at global level, investigating different structural typologies, and finding those more suitable to reach the aimed structural behaviour. In particular, three structural schemes are investigated:

- The truss scheme, where diagonals are the dissipating elements and work both in tension and compression;
- The V-shaped braces, where diagonals both work in tension and compression.
- The X-shaped braces (split or not between the consecutive shelves), where diagonals are the dissipating elements and only work in tension;

The starting design hypotheses are the same for each structural type and listed below:

- As previously said, the overall geometry of the structure is the same that has been adopted for the design of the case studies (§7.1);
- The number of load levels is the same, and three pallets for each couple of beams have been considered. The pallets' characteristics are the same that have been adopted for the design of the case studies (dimensions, weight, type of pallet for each load level).

The seismic design of the structures is made through modal with response spectrum numerical analysis. A 2D model representing the CA frame is made. For this direction, the whole structure's behaviour is judged fully represented by a unique CA frame, where corresponding loads and participant mass have been inserted. Material is defined with an elastic behaviour, and all the elements are modelled through a mono-dimensional frame element. The elastic response spectrum used as the seismic input is the same that has been used for the design of the case studies (Figure 7-2). The response spectrum is defined for the city of Van (Turkey), which is characterized by a peak ground acceleration equal to 0.3g for a return period of 475 years. The design response spectrum is obtained by reducing the elastic one through the behaviour factor, which depends on the structural scheme adopted. The value of the behaviour factor is determined according to Eurocodes prescriptions (prEN 1998:2019). No other reductions of the response spectrum are taken into consideration: no reduction of lateral stiffness is considered, neither the adoption of coefficients to take into account the capability of pallets to dissipate seismic energy (K_d factor, equation (7-III) from §7.2). This strict "no discount" line has been adopted, preferring to find acceptable solutions in the most challenging conditions. The possibility to re-introduce more relaxed

rules may be evaluated only after, assessing firstly if their applicability is reasonable for ARSWs (these assumptions are validated only for traditional racks, and their applicability to ARSWs should be evaluated).
Load combinations in static and seismic ULS conditions are defined the same as for the case studies (equation (7-II), §7.1), while the participating mass has been defined as follows:

$$G_1 + G_2 + Q_1 \tag{8-I}$$

where Ψ_2 coefficient has not been considered, unlike in the previous design phase. This choice is in line with the strict "no discount" policy, whose characteristics are gathered within Table 8-1, where the input design parameters are compared to those assumed for the case studies' design.

Table 8-1: Comparison of starting design hypothesis for case studies and the design optimization.

	Design optimization: "no discount" policy	Case studies
Number of pallets for each couple of beams	3	3 or 2
Mass definition	$G_1 + G_2 + Q_1$ (no reduction of pallet mass)	$G_1 + G_2 + \Psi_2 \cdot Q_1$ ($\Psi_2 = 0.8$) (reduction of pallet mass up to 20%)
Definition of design response spectrum	Q factor = 2.0	Q factor = 2.0 or 1.5 (depending on the structural typology)
	No other reduction considered.	K_d =0.8 reduction factor considered.
Reduction of stiffness along CA direction	Not taken into consideration.	Considered when available from experimental tests. It implies higher fundamental periods.

The design of the structure is made adopting the prescriptions of Eurocode 8 (prEN 1998:2019). Capacity design rules are applied to the dissipative zone, while the upper elastic one is designed with the forces resulting from the numerical analysis. The dissipative zone is designed aiming at a medium ductility class, corresponding to Ductility Class 2 (DC2) as defined by prEN 1993-1-1:2019. In DC2, the local overstrength capacity, the local deformation capacity and the local energy dissipation capacity are considered, and global plastic mechanisms are controlled. According to this DC, a behaviour factor major than 1 can be assumed, and, in line with the highest behaviour factor suggested within Eurocode 8 part 2 (prEN 1998-1-2:2019) for the structural typologies selected, the structures are designed assuming a behaviour factor equal to 2.
The design strategy that has been applied agrees with the prescriptions of Eurocode 8 part 2 for structures belonging to DC2 (prEN 1998-1-2:2019):

- The dissipating elements are the diagonals, and all the others have to be over-resistant. The diagonals are designed with the forces resulting from the numerical analysis, while the other elements are designed by assuming the following increased values of N_{Ed}, M_{Ed} and V_{Ed} :

$$\begin{aligned} N_{Ed} &= N_{Ed,G} + \Omega \cdot N_{Ed,E} \\ M_{Ed} &= M_{Ed,G} + M_{Ed,E} \\ V_{Ed} &= V_{Ed,G} + V_{Ed,E} \end{aligned} \tag{8-II}$$

  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 in the seismic design situation; $N_{Ed,E}$, $M_{Ed,E}$ and $V_{Ed,E}$ are the axial force, bending moment and shear force due to seismic action; and Ω is the seismic magnification factor. Ω varies with the structural system. For DC2 structure, the values of Ω are given based on the structural type, and may change based on the non-dissipating element to design. For the structural types adopted in this study (frames with concentric bracings), the unique value of 1.5 is given to design beams and columns.

When tension-only braces are adopted (X bracings), the non-dimensional slenderness λ of the dissipating elements must be limited to 1.5< λ< 2.5. The lower limit is set to avoid the over-loading of the columns in the pre-buckling phase, and the upper level avoids having too slender profiles that may have an unsatisfactory behaviour under cyclic loading.

- The connection of the dissipative element has to be over-resistant with respect to the tensile resistance of the dissipative element (in this case, of the diagonal), according to the following equation:

$$R_d = \gamma_{rm} \cdot \gamma_{sh} \cdot R_{fy} \tag{8-III}$$

  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. Since steel qualities for cold-formed applications have been adopted, no values for the γ_{rm} are available for these kinds of steel, actually because usually cold-formed thin-walled elements are not used for dissipative design. In this framework, a medium value of 1.20 has been adopted. This value is used for a steel grade whose nominal yield strength is close to the one adopted in this application. The hardening factor has been assumed equal to 1.1, as suggested by Eurocode 8 standards. In addition to this design rule, as indicated within EN16681 (2016), the connection is designed to be characterized by a bearing resistance (plastic ovalization) at least major than 1.20 times the shear resistance of bolts to prevent fragile crises.

The design of an over-resistant connection with respect to the dissipating element requires an optimization at local level, where studies are made around the possible strategies to reach this hierarchy.

Finally, the structural assessment is made through the execution of NLTH analyses. The seismic input is the same that has been previously used for the vulnerability assessment of the case studies (§7.4.2). In particular, this set is made of 15 natural accelerograms that have been selected from NGA-West2 database (Bozorgnia et al. 2014) that match the target Conditional Spectra (CS) (J. Baker 2011; Lin, Haselton, and Baker 2013a; 2013b) at a 2475 years return period, or equivalently an exceedance probability of 2% in 50 years. Record selection has also been performed for a wider range of probabilities of exceedance. The structure is tested through 9 amplification of the seismic inputs (from now on Scale Factors (SF)), corresponding to a range of various probability of exceedance (from 60% to 0.1%). The numerical model is made including geometrical non-linearities and modelling the non-dissipative elements as elastic, while the non-linear cyclic behaviour of the dissipative elements is included. In the following paragraphs, major details about the dissipative elements' modelling strategies and the definition of the cyclic behaviour are given. The non-dissipative components are checked in the post-process by the execution of the safety checks.

8.2. Optimization at global level

The design optimization at global level deals with investigating different structural typologies and finding those more suitable for the aimed structural behaviour. Two structural types are mainly involved: the truss one (where diagonal work both in compression and in tension) and the X bracings (with tension-only diagonals). Different configurations and cross-sections for diagonals have been adopted to fit the best the design forces. Still, the problem is always in the diagonal-to-upright connection, which is very hard to be designed over-resistant with respect to the diagonal element. The driving parameter for the design of the connection is plastic-ovalization. In the following, more details are given about this phase of the optimization of the design.

8.2.1 Truss scheme

In this paragraph, the results about the adoption of truss scheme are given and analysed. In the truss structural scheme, the dissipating elements are the diagonals, working both in tension and in compression. The C-shaped cross-section is adopted for diagonals, being the most used for these elements. The methodology previously illustrated (§8.1) is followed for the design of the structure. The design input parameters are gathered within Table 8-1. Diagonals have been designed directly using the forces resulting from the numerical analyses (Figure 2-1 summarises diagonals' characteristics). Table 8-2 gathers the D/C ratios of diagonal elements (axial compression and tension are checked): diagonal thicknesses are changed five times to fit better the demand (Figure 8-2 b). The configuration of diagonal-to-upright connection is kept the same as currently used, where diagonal is directly connected to upright through one (or more) bolts. In this case, 2 M12 10.9 bolts have been used for connection (the maximum number and diameter compatible with the size of diagonal cross-section, to respect the minimum distances of holes as indicated within Eurocode3 part 8 (prEN 1993-1-8:2019)). Table 8-3 gathers the D/C ratios resulting from the execution of safety checks of connections. It can be noticed that both shear bolt resistance and bearing resistance are not sufficient, and bearing resistance is the leading design parameter. The bearing resistance of a bolted connection with thin-walled cold-formed elements involved can be calculated as follows (prEN1993-1-3:2019 table 10.5):

$$F_{b,Rd} = \frac{2.5 \cdot \alpha_b \cdot k_t \cdot f_u \cdot d \cdot t}{\gamma_{M2}} \tag{8-IV}$$

where: α_b depends on the geometry of the connection (in any case $\alpha_b \leq 1.00$); k_t depends on the thickness of the element, and if the thickness is minor than 1.25 mm, k_t is minor than 1.00 but in any case $k_t \leq 1.00$; 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 to be used for checks on steel connections. In case of having both α_b and k_t equal to 1.00, the only way 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, 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 this case, the increment of bolts' number would imply the use of bigger cross-sections, and so, the design force of the connection would be increased, too.

Finding balance is not easy. Besides, considering that the connection has to be over-resistant with respect to the diagonal element's ultimate resistance, the configuration of the connection is not helpful. In fact, the bolts are placed (and 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 plastic ovalization 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 bigger than the former, and the use of ultimate stress f_u (that is used to calculate plastic ovalization resistance, while yield strength is used for tensile resistance) may be not sufficient to make up for this difference (Figure 8-4).
Another point is that this structural scheme implies that diagonals work both in tension and compression, so basically, since buckling resistance is lower than tensile resistance, buckling resistance is the design parameter for diagonals. This implies that although the profiles fit good the compression force demand, they are over-dimensioned for tensile force demand. The design of connections gets so more complicated since it has to be over-resistant with respect to diagonals' ultimate tensile resistance. This is why the X bracings scheme type results more suitable for these applications, with diagonals only working in tension.

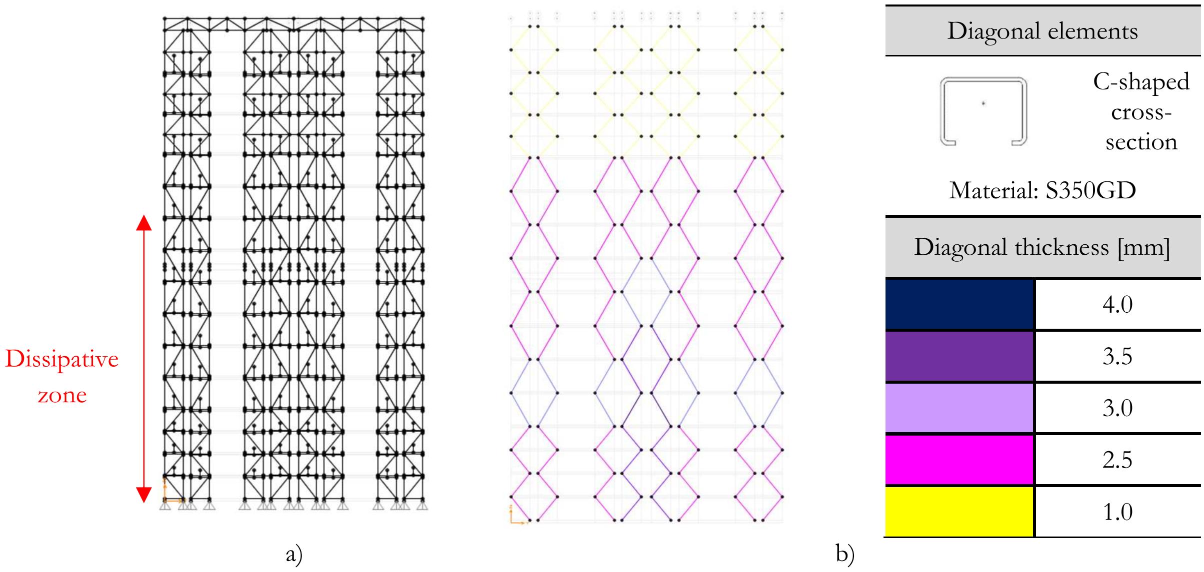


Figure 8-2: Truss scheme type: a) frame view, where the dissipative zone is indicated; b) cross-sections of diagonals.

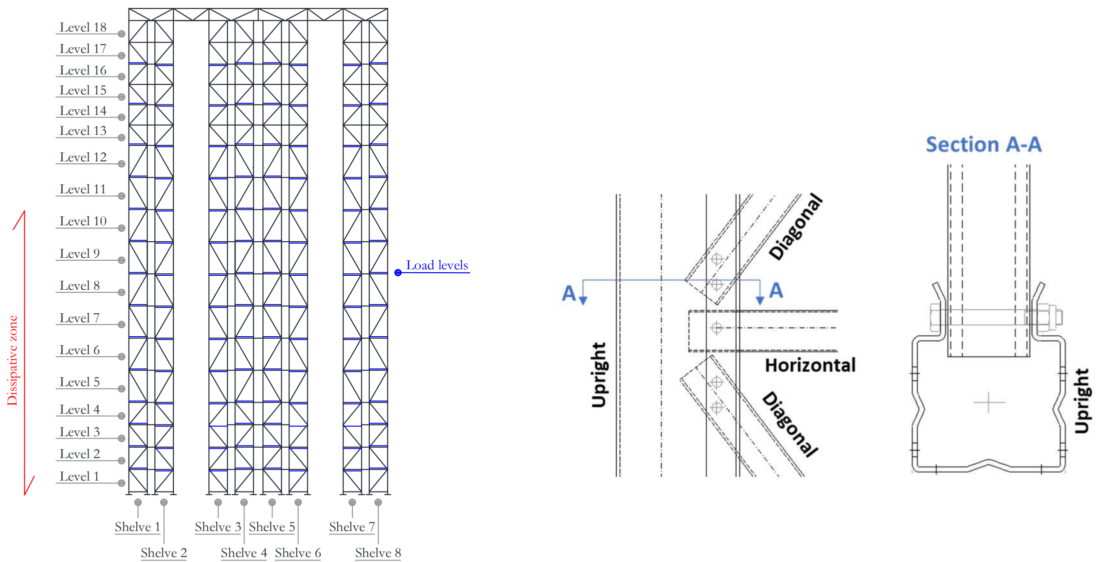


Figure 8-3: Reference scheme and view of diagonal-to-upright connection.

Table 8-2: Demand / Capacity ratios for diagonal elements (Figure 8-3 to be taken as reference scheme).

		D/C axial compression (stability)								D/C tensile force							
		Shelve								Shelve							
		1	2	3	4	5	6	7	8	1	2	3	4	5	6	7	8
Level	1	0.89	0.93	0.91	0.96	0.96	0.90	0.96	0.92	0.27	0.30	0.29	0.31	0.31	0.29	0.30	0.27
	2	0.80	0.76	0.86	0.71	0.70	0.86	0.78	0.82	0.23	0.20	0.27	0.15	0.15	0.27	0.20	0.23
	3	0.62	0.66	0.60	0.72	0.72	0.60	0.67	0.64	0.16	0.18	0.15	0.20	0.20	0.16	0.18	0.16
	4	0.69	0.65	0.73	0.60	0.60	0.74	0.67	0.71	0.20	0.18	0.24	0.13	0.14	0.24	0.18	0.21
	5	0.83	0.90	0.79	1.00	1.00	0.79	0.93	0.87	0.13	0.16	0.13	0.21	0.21	0.13	0.17	0.13
	6	0.91	0.83	1.00	0.92	0.91	1.00	0.87	0.95	0.18	0.14	0.23	0.15	0.15	0.23	0.15	0.18
	7	0.78	0.87	0.73	0.99	0.99	0.73	0.91	0.83	0.14	0.18	0.13	0.23	0.23	0.13	0.19	0.15
	8	0.84	0.75	0.89	0.88	0.88	0.90	0.79	0.88	0.19	0.15	0.22	0.18	0.18	0.22	0.16	0.19
	9	0.64	0.76	0.64	0.97	0.97	0.64	0.76	0.65	0.10	0.15	0.11	0.22	0.22	0.11	0.15	0.10
	10	0.72	0.60	0.79	0.89	0.88	0.80	0.60	0.71	0.14	0.09	0.18	0.17	0.17	0.18	0.09	0.14
	11	0.47	0.61	0.55	0.91	0.92	0.54	0.60	0.47	0.06	0.12	0.10	0.23	0.22	0.10	0.12	0.06
	12	0.44	0.38	0.53	0.69	0.68	0.53	0.37	0.42	0.10	0.07	0.15	0.18	0.18	0.15	0.07	0.10
	13	0.30	0.39	0.45	0.68	0.69	0.44	0.36	0.29	0.10	0.15	0.18	0.28	0.28	0.18	0.14	0.10
	14	0.29	0.32	0.45	0.65	0.64	0.45	0.30	0.26	0.12	0.13	0.21	0.28	0.29	0.20	0.12	0.12
	15	0.19	0.23	0.38	0.50	0.51	0.37	0.19	0.20	0.07	0.09	0.16	0.22	0.22	0.16	0.08	0.07
	16	0.19	0.15	0.32	0.43	0.42	0.32	0.13	0.20	0.09	0.06	0.15	0.19	0.20	0.15	0.05	0.10
	17	0.41	0.08	0.30	0.18	0.19	0.29	0.09	0.45	0.19	0.04	0.13	0.08	0.08	0.13	0.04	0.21
	18	0.45	0.08	0.32	0.21	0.20	0.32	0.13	0.48	0.22	0.04	0.16	0.10	0.10	0.16	0.05	0.25

Table 8-3: Demand / Capacity ratios for diagonal-to-upright connections (Figure 8-3 to be taken as reference scheme).

External shelve	Shear resistance of bolts	Bearing resistance	Failure side	Net section	Failure of end side (Diagonal side)	Failure of end side (upright side)	Max D/C	failure
Level 1	1.13	1.76	Diagonal	1.41	0.94	0.43	1.76	Bearing resistance
Level 2	1.13	1.76	Diagonal	1.41	0.94	0.43	1.76	Bearing resistance
Level 3	1.13	1.76	Diagonal	1.41	0.94	0.43	1.76	Bearing resistance
Level 4	1.13	1.76	Diagonal	1.41	0.94	0.43	1.76	Bearing resistance
Level 5	1.35	2.64	Diagonal	1.41	0.93	0.52	2.64	Bearing resistance
Level 6	1.35	2.64	Diagonal	1.41	0.93	0.52	2.64	Bearing resistance
Level 7	1.13	1.76	Diagonal	1.41	0.94	0.43	1.76	Bearing resistance
Level 8	1.13	1.76	Diagonal	1.41	0.94	0.43	1.76	Bearing resistance
Level 9	1.13	1.76	Diagonal	1.41	0.94	0.43	1.76	Bearing resistance
Level 10	1.13	2.56	Upright	1.41	0.94	0.71	2.56	Bearing resistance

Central shelve	Shear resistance of bolts	Bearing resistance	Failure side	Net section	Failure of end side (Diagonal side)	Failure of end side (upright side)	Max D/C	failure
Level 1	1.57	2.64	Diagonal	1.41	0.93	0.6	2.64	Bearing resistance
Level 2	1.57	2.64	Diagonal	1.41	0.93	0.6	2.64	Bearing resistance
Level 3	1.57	2.64	Diagonal	1.41	0.93	0.6	2.64	Bearing resistance
Level 4	1.57	2.64	Diagonal	1.41	0.93	0.6	2.64	Bearing resistance
Level 5	1.79	2.63	Diagonal	1.41	0.93	0.69	2.63	Bearing resistance
Level 6	1.57	2.64	Diagonal	1.41	0.93	0.6	2.64	Bearing resistance
Level 7	1.57	2.64	Diagonal	1.41	0.93	0.6	2.64	Bearing resistance
Level 8	1.35	2.64	Diagonal	1.41	0.93	0.52	2.64	Bearing resistance
Level 9	1.35	2.64	Diagonal	1.41	0.93	0.52	2.64	Bearing resistance
Level 10	1.13	2.56	Upright	1.41	0.94	0.71	2.56	Bearing resistance

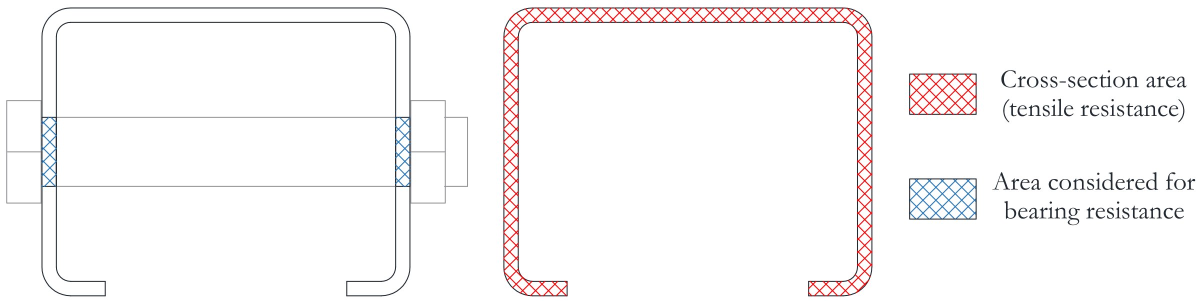


Figure 8-4: Comparison of mobilised areas for tensile resistance of diagonals and bearing resistance of connection.

8.2.2 X bracing scheme

In this paragraph, the results about the adoption of the X bracing scheme are given and analysed. In the X bracing structural scheme, the dissipating elements are the diagonals, working only in tension. For diagonals, the most used C-shaped cross-section is adopted. The methodology previously illustrated (§8.1) is followed for the design of the structure. The design input parameters are gathered within Table 8-1. Diagonals have been designed directly using the forces resulting from the numerical analyses (Figure 8-5 summarises diagonals' characteristics). Table 8-4 gathers the D/C ratios of diagonal elements (axial tensile resistance is checked): diagonal thicknesses are changed four times to fit better the demand (Figure 8-5 b). The configuration of diagonal-to-upright connection is kept the same as currently used, where diagonal is directly connected to upright through one (or more) bolts. In this case, 2 M12 10.9 bolts have been used for connection (the maximum number and diameter compatible with the size of diagonals' cross-section, to respect the minimum distances of holes as indicated within Eurocode3 part 8 (prEN 1993-1-8:2019)). It can be noticed that, although this structural typology allows a better optimization of the diagonal elements (see Table 8-4), the design of the connection is critical, and also this time, the leading design parameter for connection is plastic ovalization.

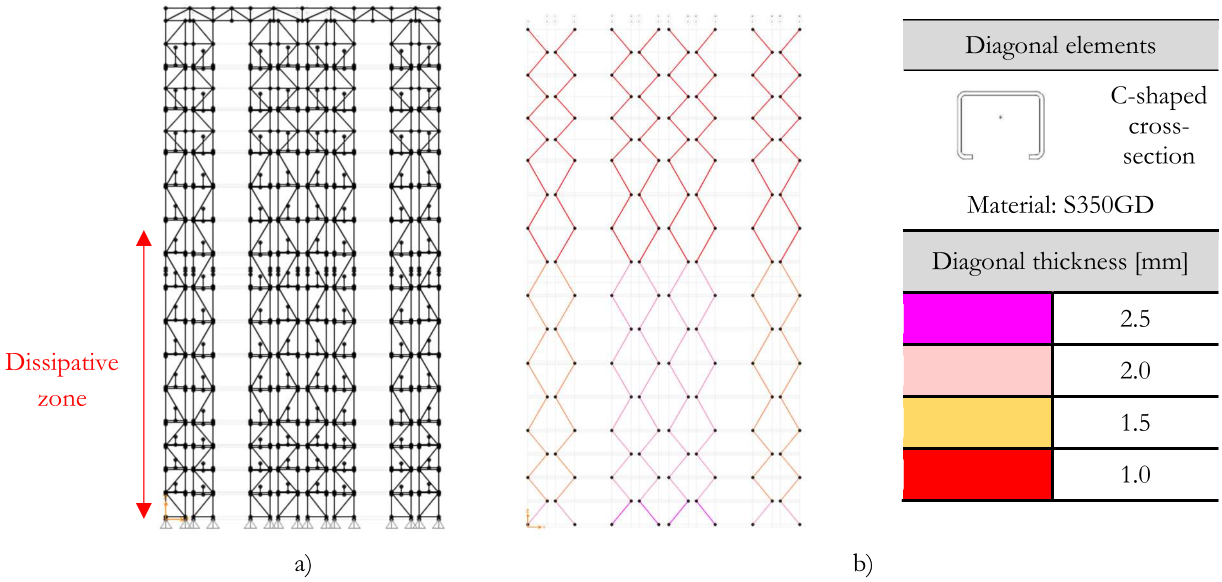


Figure 8-5: X bracings scheme type: a) frame view, where the dissipative zone is indicated; b) cross-sections of diagonals.

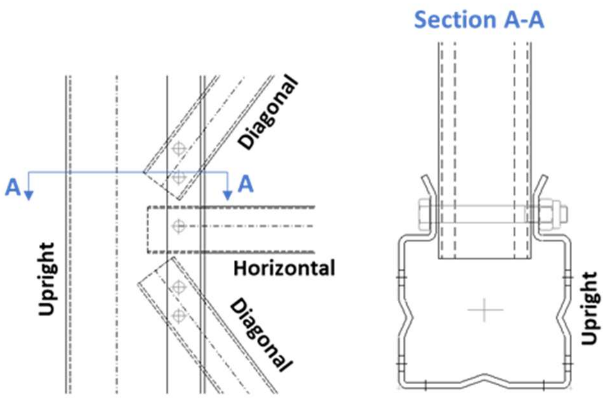


Figure 8-6: View of diagonal-to-upright connection.

Table 8-4: Demand / Capacity ratios for diagonal elements (Figure 8-3 to be taken as reference scheme).

		D/C tensile force							
		Shelve							
		1	2	3	4	5	6	7	8
Level	1	0.88	0.88	0.84	0.84	0.84	0.84	0.90	0.91
	2	0.85	0.85	0.71	0.70	0.70	0.71	0.88	0.87
	3	0.92	0.92	0.73	0.72	0.72	0.73	0.94	0.95
	4	0.79	0.79	0.65	0.64	0.64	0.65	0.82	0.81
	5	0.96	0.95	0.79	0.78	0.79	0.79	0.99	0.99
	6	0.84	0.84	0.72	0.71	0.71	0.72	0.89	0.88
	7	0.83	0.83	0.69	0.68	0.69	0.69	0.87	0.87
	8	0.69	0.70	0.61	0.61	0.60	0.61	0.73	0.73
	9	0.69	0.68	0.61	0.61	0.61	0.61	0.68	0.69

	D/C tensile force							
	Shelve							
	1	2	3	4	5	6	7	8
10	0.74	0.74	0.99	0.99	0.99	0.99	0.74	0.74
11	0.68	0.68	0.99	0.98	0.98	0.98	0.67	0.68
12	0.50	0.50	0.83	0.83	0.83	0.83	0.48	0.48
13	0.10	0.15	0.18	0.28	0.28	0.18	0.14	0.10
14	0.12	0.13	0.21	0.28	0.29	0.20	0.12	0.12
15	0.07	0.09	0.16	0.22	0.22	0.16	0.08	0.07
16	0.09	0.06	0.15	0.19	0.20	0.15	0.05	0.10
17	0.19	0.04	0.13	0.08	0.08	0.13	0.04	0.21
18	0.22	0.04	0.16	0.10	0.10	0.16	0.05	0.25

Table 8-5: Demand / Capacity ratios for diagonal-to-upright connections (Figure 8-3 to be taken as reference scheme).

External shelve	Shear resistance of bolts	Bearing resistance	Failure side	Net section	Failure of end side (Diagonal side)	Failure of end side (upright side)	Max D/C	failure
Level 1	0.90	1.76	Diagonal	1.41	0.94	0.58	1.76	Bearing resistance
Level 2	0.61	1.59	Diagonal	1.44	0.85	0.39	1.59	Bearing resistance
Level 3	0.61	1.59	Diagonal	1.44	0.85	0.39	1.59	Bearing resistance
Level 4	0.61	1.59	Diagonal	1.44	0.85	0.39	1.59	Bearing resistance
Level 5	0.61	1.59	Diagonal	1.44	0.85	0.39	1.59	Bearing resistance
Level 6	0.61	1.59	Diagonal	1.44	0.85	0.39	1.59	Bearing resistance
Level 7	0.61	1.59	Diagonal	1.44	0.85	0.39	1.59	Bearing resistance
Level 8	0.61	1.59	Diagonal	1.44	0.85	0.39	1.59	Bearing resistance
Level 9	0.61	1.59	Diagonal	1.44	0.85	0.39	1.59	Bearing resistance
Level 10	0.45	1.93	Diagonal	1.41	0.95	0.47	1.93	Bearing resistance

Central shelve	Shear resistance of bolts	Bearing resistance	Failure side	Net section	Failure of end side (Diagonal side)	Failure of end side (upright side)	Max D/C	failure
Level 1	1.13	1.76	Diagonal	1.41	0.94	0.72	1.76	Bearing resistance
Level 2	0.90	1.76	Diagonal	1.41	0.94	0.58	1.76	Bearing resistance
Level 3	0.90	1.76	Diagonal	1.41	0.94	0.58	1.76	Bearing resistance
Level 4	0.90	1.76	Diagonal	1.41	0.94	0.58	1.76	Bearing resistance
Level 5	0.90	1.76	Diagonal	1.41	0.94	0.58	1.76	Bearing resistance
Level 6	0.90	1.76	Diagonal	1.41	0.94	0.58	1.76	Bearing resistance

Level 7	0.90	1.76	Diagonal	1.41	0.94	0.58	1.76	Bearing resistance
Level 8	0.90	1.76	Diagonal	1.41	0.94	0.58	1.76	Bearing resistance
Level 9	0.61	1.59	Diagonal	1.44	0.85	0.39	1.59	Bearing resistance
Level 10	0.45	1.93	Diagonal	1.41	0.95	0.47	1.93	Bearing resistance

To overcome the problem related to design an over-resistant connection, optimization at the local level has to be performed. In general, two different strategies can be adopted: the former is the reinforcement of connection, the latter is the reduction of the dissipative element's resistance. In the first case, since the leading mechanism for the design of connection is plastic ovalization, a possible way is to locally increase the thickness of the end sections of the element in correspondence with the connection. In this way, bearing resistance is increased without increasing the ultimate resistance of the element. Anyway, this solution could determine an increment of the costs related to production of diagonal elements, which would need additional working processes (welding additional sheets at the end parts). As an alternative, the cross-section resistance can be reduced by locally weakening the cross-section, in line with the "dog bone" philosophy. Preliminary confirmation of this strategy is given by adopting a rectangular plate section for the diagonals, which is larger at the extremities, allowing to put the necessary number of bolts (Figure 8-7). The reduction of cross-section is calculated to obtain an over-resistant connection, in line with the capacity design strategy (equation (8-III)). Table 8-6 gives the D/C ratios of diagonal elements in this configuration (axial tensile resistance is checked). Table 8-7 shows the D/C ratios of diagonal connections, calculated with respect to the diagonals' ultimate tensile resistance. All the safety checks are fulfilled. This solution offers a double benefit, because the resistance of the connection is increased while, at the same time, the resistance of the element is limited thanks to the reduction of the cross-section. On the other end, the element's slenderness is quite high and the consequent cyclic performance may be poor due to the dynamic overshoot that may occur due to the meagre buckling resistance (Kazemzadeh Azad, Topkaya, and Bybordiani 2018).

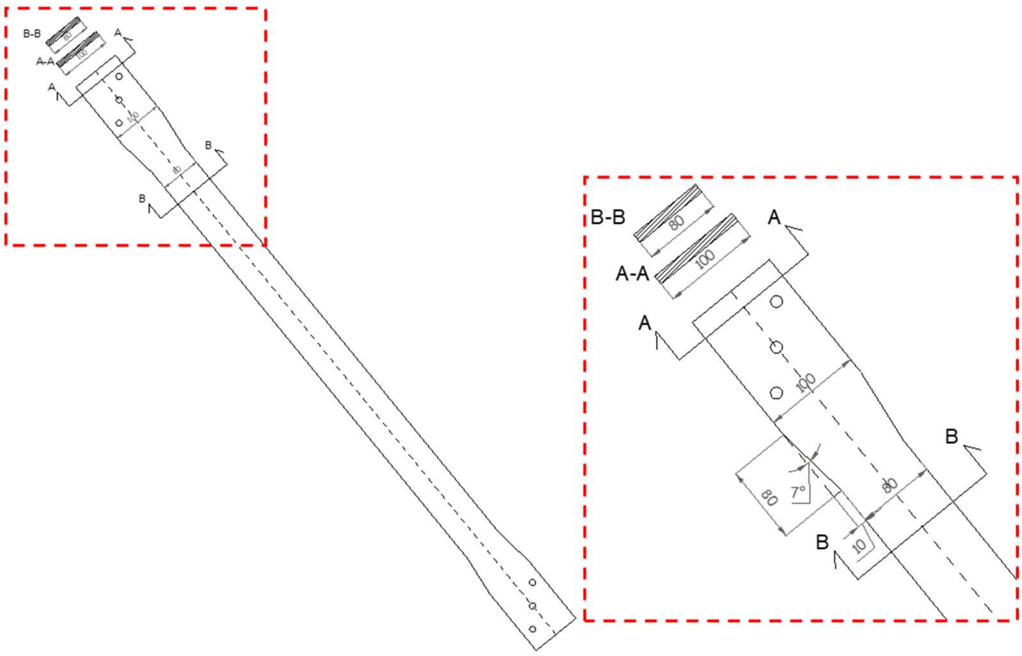


Figure 8-7: The plate section diagonal.

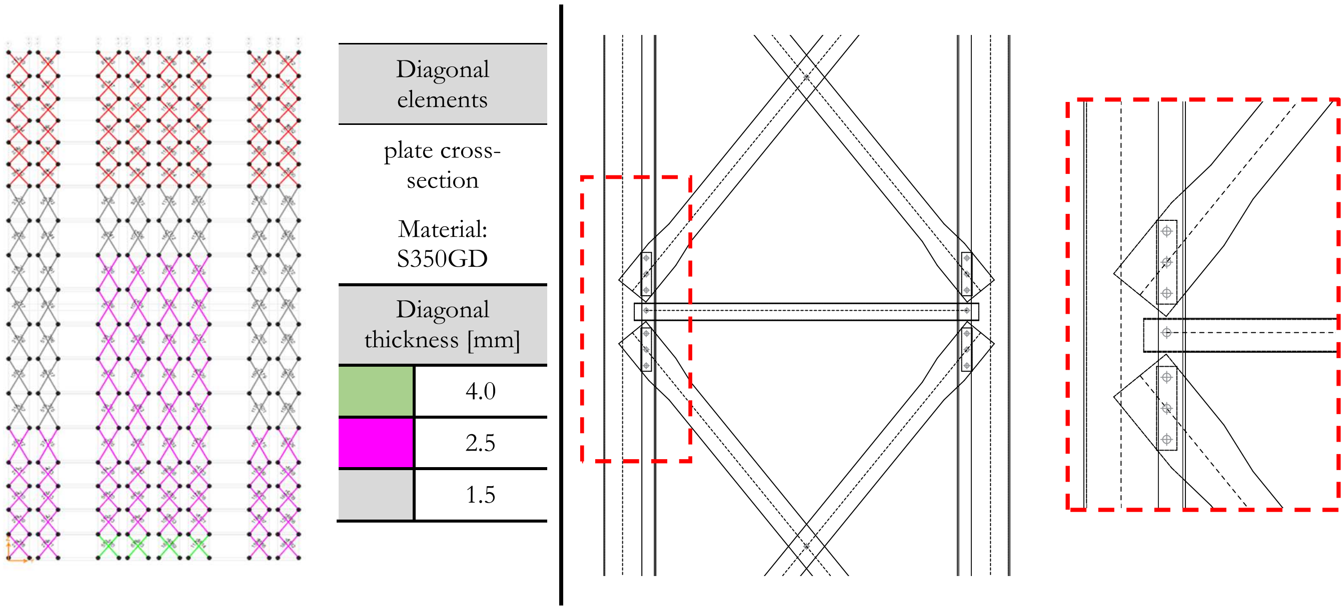


Figure 8-8: X bracings scheme type with plate cross-section for diagonals: diagonals' characteristics are indicated.

Table 8-6: Demand / Capacity ratios for diagonal elements (Figure 8-3 to be taken as reference scheme).

		D/C tensile force							
		Shelve							
		1	2	3	4	5	6	7	8
Level	1	0.88	0.86	0.92	0.82	0.82	0.91	0.88	0.90
	2	0.48	0.51	0.54	0.72	0.71	0.54	0.52	0.49
	3	0.62	0.60	0.70	0.70	0.70	0.70	0.61	0.64
	4	0.42	0.45	0.46	0.69	0.69	0.46	0.46	0.43
	5	0.70	0.67	0.76	0.79	0.79	0.76	0.69	0.72
	6	0.78	0.83	0.50	0.80	0.79	0.50	0.87	0.82
	7	0.91	0.87	0.62	0.72	0.72	0.61	0.92	0.96
	8	0.69	0.74	0.41	0.71	0.71	0.41	0.78	0.72
	9	0.74	0.70	0.50	0.61	0.61	0.50	0.69	0.74
	10	0.53	0.61	0.34	0.65	0.65	0.33	0.61	0.53
	11	0.55	0.52	0.68	0.81	0.81	0.67	0.51	0.55
	12	0.36	0.44	0.54	0.80	0.80	0.54	0.43	0.34
	13	0.25	0.26	0.39	0.56	0.57	0.39	0.23	0.23
	14	0.18	0.26	0.36	0.56	0.56	0.36	0.24	0.16
	15	0.12	0.10	0.28	0.42	0.42	0.28	0.08	0.13
	16	0.12	0.11	0.29	0.40	0.39	0.29	0.07	0.14
	17	0.31	0.13	0.16	0.23	0.23	0.16	0.17	0.35
	18	0.37	0.10	0.30	0.24	0.24	0.30	0.13	0.41

Table 8-7: Demand /Capacity ratios for diagonal-to-upright connections (Figure 8-3 to be taken as reference scheme).

External shelve	Shear resistance of bolts	Bearing resistance	Failure side	Net section	Failure of end side (Diagonal side)	Failure of end side (upright side)	Max D/C	failure
Level 1	0.33	0.97	Diagonal	0.94	0.52	0.32	0.97	Bearing resistance
Level 2	0.33	0.97	Diagonal	0.94	0.52	0.32	0.97	Bearing resistance
Level 3	0.33	0.97	Diagonal	0.94	0.52	0.32	0.97	Bearing resistance
Level 4	0.33	0.97	Diagonal	0.94	0.52	0.32	0.97	Bearing resistance
Level 5	0.33	0.97	Diagonal	0.94	0.52	0.32	0.97	Bearing resistance
Level 6	0.2	0.97	Diagonal	0.94	0.52	0.19	0.97	Bearing resistance
Level 7	0.2	0.97	Diagonal	0.94	0.52	0.19	0.97	Bearing resistance
Level 8	0.2	0.97	Diagonal	0.94	0.52	0.19	0.97	Bearing resistance
Level 9	0.2	0.97	Diagonal	0.94	0.52	0.19	0.97	Bearing resistance
Level 10	0.2	0.97	Diagonal	0.94	0.52	0.32	0.97	Bearing resistance

Central shelve	Shear resistance of bolts	Bearing resistance	Failure side	Net section	Failure of end side (Diagonal side)	Failure of end side (upright side)	Max D/C	failure
Level 1	0.39	0.97	Diagonal	0.94	0.52	0.39	0.97	Bearing resistance
Level 2	0.33	0.97	Diagonal	0.94	0.52	0.32	0.97	Bearing resistance
Level 3	0.33	0.97	Diagonal	0.94	0.52	0.32	0.97	Bearing resistance
Level 4	0.33	0.97	Diagonal	0.94	0.52	0.32	0.97	Bearing resistance
Level 5	0.33	0.97	Diagonal	0.94	0.52	0.32	0.97	Bearing resistance
Level 6	0.33	0.97	Diagonal	0.94	0.52	0.32	0.97	Bearing resistance
Level 7	0.33	0.97	Diagonal	0.94	0.52	0.32	0.97	Bearing resistance
Level 8	0.33	0.97	Diagonal	0.94	0.52	0.32	0.97	Bearing resistance
Level 9	0.33	0.97	Diagonal	0.94	0.52	0.32	0.97	Bearing resistance
Level 10	0.33	0.97	Upright	0.94	0.52	0.53	0.97	Bearing resistance

8.3. Optimization at local level: locally reduced cross-section of diagonal elements

The optimization at global level of the structure highlights that the X bracing structural type is more suitable for the design method's purposes, since it allows for better optimization of the diagonal elements. Anyway, the design of an over-resistant connection is tricky and quite difficult to be obtained, since the driving design parameter is the bearing resistance of the connection, which is quite difficult to improve without increasing the cross-section of the diagonal. The increment of the diagonal's cross-section causes an increase of the ultimate tensile resistance of the element (that is the demand for the design of the connection, in line with the capacity design strategies for steel dissipative concentric braces structure). To overcome this issue, alternative methods are investigated, such as reducing the dissipative element's resistance. In this regard, Legeron et al. (2014) studied the performances of bracings of concentrically steel braced frames under cyclic loading, with the introduction of holes in the braces. In this study, holes are introduced in the brace to weaken the cross-section and allow an easier design of the over-resistant bolted connection. Holes characteristics (transversal size and longitudinal length) are calculated aiming to respectively have an over-resistant connection and gain an acceptable ductility level. Holes are placed in different part of the bracing element (at the end sections and/or at the mid-length), and experimental tests (monotonic in tension, monotonic in compression and cyclic) are executed to investigate the influence of the different configuration of the holes in the structural behaviour of the brace under different load conditions. In the same framework, Gusella et al. (2019) carried out an experimental campaign focused on studying the influence of various pattern of holes in the structural behaviour of double C braces in compression, tension and cyclic load conditions. In this study, cold-formed profiles are used. The transversal size of the holes is kept the same, while the number, the length of each hole and position (in the middle of the element or close to connection, at the end sections) is varied. Different profiles lengths are tested to investigate the influence of holes on the slenderness of the profile. In general, many studies around the influence of holes on the performances in compression of cold-formed elements have been performed (Pu et al. 1999; Cristopher D. Moen and Schafer 2008; Cristopher D Moen and Schafer 2009a), pointing out that the hole placement and its size can significantly affect the buckling resistance, the buckling mode and the slenderness of the element. As a consequence, holes may also negatively influence the behaviour of the component in cyclic load conditions.

In this study, the optimization at local level of the element is carried out by reducing the cross-section through holes. In line with previous research findings (Cristopher D Moen and Schafer 2009b), holes are placed at the end sections of the element, close to connection, to affect the less possible the slenderness of the element.

Design steps are inverted: the first step is to design the connection of diagonals, using as demand the forces resulting from the modal response spectrum analysis, amplified according to capacity design rules (equation (8-III)). The connection's design determines the dimensions of the cross-section of the diagonals in terms of thickness and size of the sides of the C section. Then, the holes' transversal size is determined, calculating the sufficient area of the diagonal so that the ultimate tensile resistance of the element is major than the demand forces resulting from the analysis. Basically, the following equations are used for the design of connections (equation (8-V)) and diagonal elements (equation (8-VI)):

$$R_d \geq \gamma_{rm} \cdot \gamma_{sh} \cdot F_{Ed} \rightarrow \textit{the cross-section of diagonal } A \textit{ is determined} \quad (8\text{-V})$$

$$R_{fy,Red} = F_{Ed} \rightarrow A_{Red} = {F_{Ed}}\Big/{f_y} \cdot \gamma_{M0} \rightarrow A_{hole} = A - A_{Red} \quad (8\text{-VI})$$

where: R_d is the resistance of connection; F_{Ed} is the demand corresponding to the force acting on diagonals resulting from the modal with response spectrum analysis; A is the area of the cross-section of the diagonal that results from the design of connection; $R_{fy,Red}$ is the minimum tensile resistance of the diagonal element (that has to be guaranteed at the reduced section); A_{Red} is the value of the minimum area in correspondence with the reduced section, and A_{hole} is the transversal size of the hole. In this way, the over-resistance of the connection is guaranteed, and the diagonal has a sufficient resistance with respect to the forces acting on the structure. Table 8-8 gathers diagonal elements' main characteristics, and

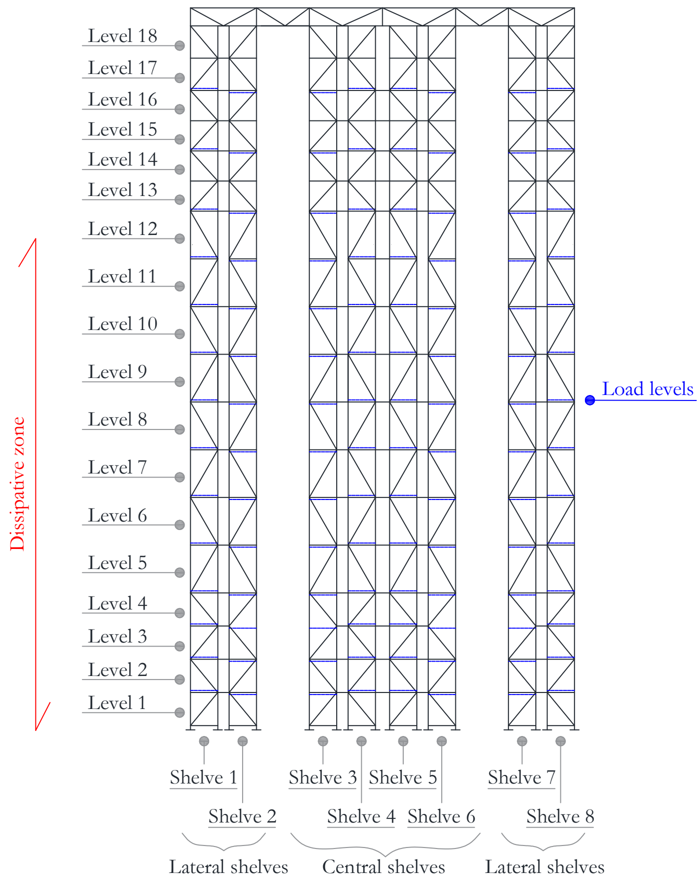
Table 8-9 shows the D/C ratios of diagonal connections. It can be noticed that three different sections are used for the dissipative area: the C 60x85x20x3 section for the 1st level, the C 60x85x20x2.5 for the 2nd to the 4th levels, and the C 60x77x20x2.5 for the 5th to the 12th level. The steel grade is S420MC (EN10149-2:2013).

Table 8-10 shows the calculation of the maximum reduction of section for each element (A_{hole}) and the final transversal size of the reduction assumed for each load level (A_{hole} assumed). In fact, with the aim of optimizing the production of the elements, and so reducing the variability of the size of the holes, the size of the holes is kept the same for different levels of diagonal, but keeping a maximum variability of 25% between the assumed area reduction and the calculated one (this variability is indicated in

Table 8-10 through the coefficient Ω*), to have the maximum involvement of the most of diagonal levels in the collapse mechanism.

Table 8-10 also shows that the over-resistance of connection is guaranteed in any case.

Table 8-8: Main characteristics of diagonal elements.



Level	lo [mm]	External Shelves						Central Shelves					
		Profile	N_{Rd} [kN]	λ	t [mm]	A [mm2]	A_{eff} [mm2]	Profile	N_{Rd} [kN]	λ	t [mm]	A [mm2]	Aeff [mm2]
1	1556	C 60x85x20x3	300.4	1.86	3	751	721	C 60x85x20x3	300.4	1.86	3	751	721
2	1556	C 60x85x20x2.5	253.6	1.66	2.5	634	470	C 60x85x20x2.5	253.6	1.66	2.5	634	470
3	1556	C 60x85x20x2.5	253.6	1.66	2.5	634	470	C 60x85x20x2.5	253.6	1.66	2.5	634	470
4	1556	C 60x85x20x2.5	253.6	1.66	2.5	634	470	C 60x85x20x2.5	253.6	1.66	2.5	634	470
5	1989	C 60x77x20x2.5	253.6	2.11	2.5	594	481	C 60x77x20x2.5	253.6	2.11	2.5	594	481
6	1989	C 60x77x20x2.5	253.6	2.11	2.5	594	481	C 60x77x20x2.5	253.6	2.11	2.5	594	481
7	1989	C 60x77x20x2.5	253.6	2.11	2.5	594	481	C 60x77x20x2.5	253.6	2.11	2.5	594	481
8	1989	C 60x77x20x2.5	253.6	2.11	2.5	594	481	C 60x77x20x2.5	253.6	2.11	2.5	594	481
9	1989	C 60x77x20x2.5	253.6	2.11	2.5	594	481	C 60x77x20x2.5	253.6	2.11	2.5	594	481
10	1989	C 60x77x20x2.5	253.6	2.11	2.5	594	481	C 60x77x20x2.5	253.6	2.11	2.5	594	481
11	1989	C 60x77x20x2.5	253.6	2.11	2.5	594	481	C 60x77x20x2.5	253.6	2.11	2.5	594	481
12	1989	C 60x77x20x2.5	253.6	2.11	2.5	594	481	C 60x77x20x2.5	253.6	2.11	2.5	594	481
13	1473	C 60x55x20x1.5	119.6	1.66	1.5	299	257	C 60x55x20x1.5	119.6	1.66	1.5	299	257
14	1473	C 60x55x20x1.5	119.6	1.66	1.5	299	257	C 60x55x20x1.5	119.6	1.66	1.5	299	257
15	1473	C 60x55x20x1.5	119.6	1.66	1.5	299	257	C 60x55x20x1.5	119.6	1.66	1.5	299	257
16	1473	C 60x55x20x1.5	119.6	1.66	1.5	299	257	C 60x55x20x1.5	119.6	1.66	1.5	299	257
17	1533	C 60x55x20x1.5	119.6	1.73	1.5	299	257	C 60x55x20x1.5	119.6	1.73	1.5	299	257
18	1533	C 60x55x20x1.5	119.6	1.73	1.5	299	257	C 60x55x20x1.5	119.6	1.73	1.5	299	257

Table 8-9: D/C ratios of diagonal connections. (*) ND=Non-Dissipative element.

Level	Shelve							
	1	2	3	4	5	6	7	8
1	0.79	0.80	1.00	1.00	1.00	1.00	0.82	0.81
2	0.74	0.74	0.94	0.89	0.88	0.94	0.76	0.76
3	0.64	0.64	0.81	0.79	0.79	0.81	0.66	0.66
4	0.67	0.65	0.87	0.78	0.77	0.87	0.67	0.69
5	0.72	0.73	0.89	0.93	0.93	0.89	0.75	0.74
6	0.67	0.66	0.89	0.84	0.84	0.89	0.69	0.70
7	0.64	0.65	0.81	0.81	0.81	0.80	0.68	0.67
8	0.55	0.54	0.74	0.71	0.71	0.74	0.57	0.57
9	0.50	0.51	0.67	0.66	0.67	0.66	0.51	0.50
10	0.42	0.41	0.60	0.58	0.58	0.60	0.41	0.41
11	0.93	0.95	0.55	0.54	0.54	0.55	0.93	0.91
12	0.75	0.73	0.49	0.49	0.48	0.49	0.71	0.72
13 (ND*)	0.39	0.41	0.67	0.66	0.67	0.66	0.38	0.36
14 (ND*)	0.39	0.38	0.69	0.69	0.68	0.69	0.35	0.36
15 (ND*)	0.20	0.22	0.49	0.50	0.50	0.49	0.19	0.18
16 (ND*)	0.23	0.22	0.49	0.49	0.48	0.49	0.20	0.21
17 (ND*)	0.26	0.28	0.22	0.22	0.22	0.22	0.33	0.32
18 (ND*)	0.36	0.34	0.33	0.33	0.32	0.33	0.37	0.38

Table 8-10: D/C ratios of diagonal connections. (*) ND=Non-Dissipative element.

	A_{hole} requested [mm^2]								A_{hole} assumed [mm^2]	Ω*	Over-resistance of connection with respect to the reduced area of diagonal (minimum value 1.32)							
	Shelve										Shelve							
	1	2	3	4	5	6	7	8			1	2	3	4	5	6	7	8
1	475	472	403	390	389	403	465	468	389	1.22	1.36	1.35	1.32	1.32	1.32	1.32	1.34	1.35
2	418	419	361	376	377	361	413	413	361	1.24	1.52	1.51	1.40	1.42	1.44	1.40	1.50	1.50
3	447	448	397	404	404	398	443	442	361	1.24	1.60	1.60	1.48	1.48	1.48	1.47	1.57	1.58
4	440	445	381	408	409	380	438	434	361	1.24	1.56	1.59	1.44	1.48	1.50	1.44	1.58	1.54
5	385	382	335	325	324	336	376	379	324	1.25	1.54	1.54	1.44	1.42	1.42	1.44	1.52	1.53
6	399	402	336	348	349	336	392	392	324	1.25	1.58	1.60	1.44	1.47	1.47	1.44	1.57	1.55
7	408	405	360	358	357	360	396	399	324	1.25	1.61	1.60	1.48	1.49	1.49	1.49	1.58	1.58
8	435	437	380	387	387	380	429	428	380	1.25	2.05	2.08	1.78	1.83	1.82	1.79	2.02	2.03
9	448	445	400	401	400	401	446	448	380	1.25	2.17	2.15	1.87	1.89	1.87	1.89	2.15	2.17
10	473	475	419	424	425	419	475	473	380	1.25	2.37	2.42	1.97	2.01	2.01	1.98	2.42	2.44
11			435	437	437	435			380	1.25			2.06	2.08	2.09	2.05		
12			450	452	453	450			380	1.25			2.20	2.18	2.23	2.20		

After the determination of the transversal size of the holes, the longitudinal length of the hole, the number of the holes and the shape of the holes is determined.
The longitudinal length of the hole depends on the level of ductility requested. In particular, the necessary length of the hole L_{red} is determined assuming that the diagonal's plastic elongation is concentrated in the reduced-section part of the profile. The following equation is used to calculate the length of the reduced-section portion:

$$L_{red} \cdot (\varepsilon_f - \varepsilon_y) = \varepsilon_y \cdot L_{diagonal} \cdot (\mu - 1) \qquad \text{(8-VII)}$$

where ε_y is the yield strain, that for the steel grade S40GD and assuming the young modulus E equal to 210000 N/mm2, is equal to 0.002; ε_f is the ultimate strain, that, according to EN10149-2 (2013), is equal to 16%; $L_{diagonal}$ is the diagonal element's total length, and μ is the local expected ductility of the element. The determination of the μ factor is made iteratively: as starting hypothesis, it is assumed that the deformation of the structure is linear and that the request of ductility is the same for all the load levels. Given that the global request of ductility is 2 (corresponding to the behaviour factor that is assumed for the design of the structure), and assuming, for the sake of simplicity, to be in the field of small deformations, the minimum length of the reduced-section part corresponding to these hypotheses is around 40 mm. This value is re-arranged in further attempts, when the effective ductility demand can be assessed.
The optimization of the shape and number of the holes (meaning that if the length of the reduced-section part of the element should be divided into more holes or not) is made looking at the behaviour in compression of the element. The optimisation aim is double: on one hand, the holes' pattern should have the more similar buckling resistance of the same element without holes to control and limit the increment of the element's slenderness. On the other hand, the element's behaviour in compression should be such as deformations are not concentrated in the reduced-section part. This would imply that the buckling mode is global and more similar as possible to the one of the intact element. The behaviour in

compression is a relevant aspect to consider, since it may affect the element's cyclic behaviour, causing, if poor, a rapid decrease of stiffness and strength when re-loading in tension. The behaviour in compression of the diagonals without any weakening is obtained by performing numerical simulations through Abaqus® FEM software. The elements are modelled using quadrangular linear SHELL elements (Figure 8-9). The material's constitutive equation is taken from EN10149-2 (2013), assuming an elasto-plastic behaviour with isotropic hardening. Assuming that x and y are the axes in-plane with the cross-section of the element and z is the axis along longitudinal axis of the element, for one end section (the one where the load is not applicated), no translational displacements and rotation around the z axis are allowed. For the other end section (where the incremental compression load is applied) only translational displacement along the z axis and rotations around the x and the y axis are allowed (translational displacement along longitudinal axis is from now on indicated as u_z). The global imperfection given to the element is calibrated to obtain the same elastic buckling load that is calculated by applying Eurocodes formulas (prEN 1993-1-3:2019).
Given that the diagonals are characterized by having a mono-symmetric cross-section, three possible elastic buckling modes are considered: the flexural, the torsional and the torso-flexural one. The lowest elastic buckling load corresponds to the torso-flexural mode for all the diagonals, and numerical simulations confirm this buckling mode. This value of eccentricity is then applied when simulating the diagonal with the reduced-section part. This element is modelled with the same approach used for the one with no weakening.
Starting from diagonal D1, which is used in the first level for all the shelves, Figure 8-9 shows a global view of the element and the mesh defined. Figure 8-10 shows the buckling mode resulting from the numerical simulation, consisting of an increasing compression load applied at one of the profile's end sections. The value of the imperfection that match the buckling load as calculated using Eurocodes formulas is equal to 5 mm, corresponding to around 1/311 times the length of the profile (1556 mm). This value is lower than the maximum imperfection to be considered for member analysis (1/250 times the length of the element, according to prEN 1993-1-1 (2019)).

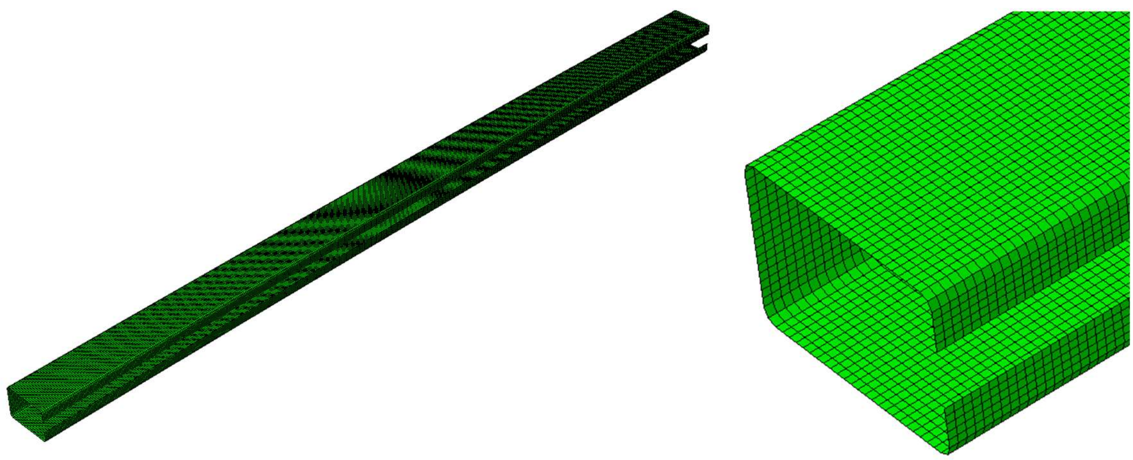


Figure 8-9: Diagonal D1: global view of the element and the mesh (Abaqus® FEM software).

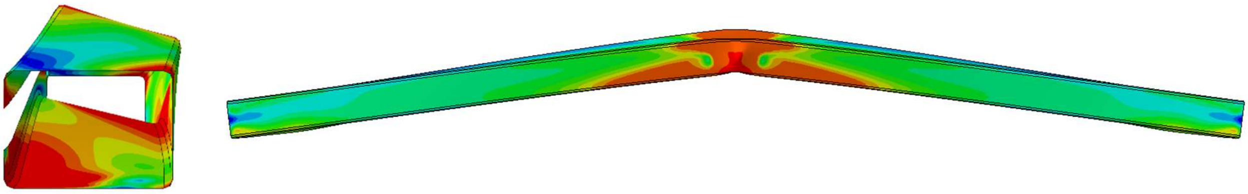


Figure 8-10: Diagonal D1: buckling mode resulting from the numerical simulation (an increasing compression load is applied).

The same numerical simulations are carried out for the D1 diagonal with the reduced-section parts. The element is modelled adopting the same strategies used for the one with no weakening, and the same imperfection has been applied. Different configurations of holes are studied, changing the number of holes and their shape (in any case, holes are placed at the extremities of the diagonals, close to connections) (Figure 8-11): in the first configuration, the total length of the reduced-section zone is 100 mm, and, for each side of the diagonal, one single hole is modelled, where the reduced zone is long 50 mm; in the second configuration, the total length of the reduced-section zone is 100 mm, and, for each side of the diagonal, two holes are modelled, where the reduced zone is long 25 mm for each hole; in the third configuration, the total length of the reduced-section zone is 60 mm, and, for each side of the diagonal, two holes are modelled, where the reduced zone is long 15 mm for each hole; in the fourth configuration, the total length of the reduced-section zone is 40 mm, and, for each side of the diagonal, two holes are modelled, where the reduced zone is long 10 mm for each hole; in the fifth configuration, the total length of the reduced-section zone is 40 mm, and, for each side of the diagonal, two holes are modelled, where the reduced zone is long 10 mm for each hole. In this last configuration, the lateral holes are "rectangular" shaped, while in all the other configurations, the rounded zone is circular. In the first four configuration, the reduced-section zone's total length is larger than the minimum requested according to the ductility demand.

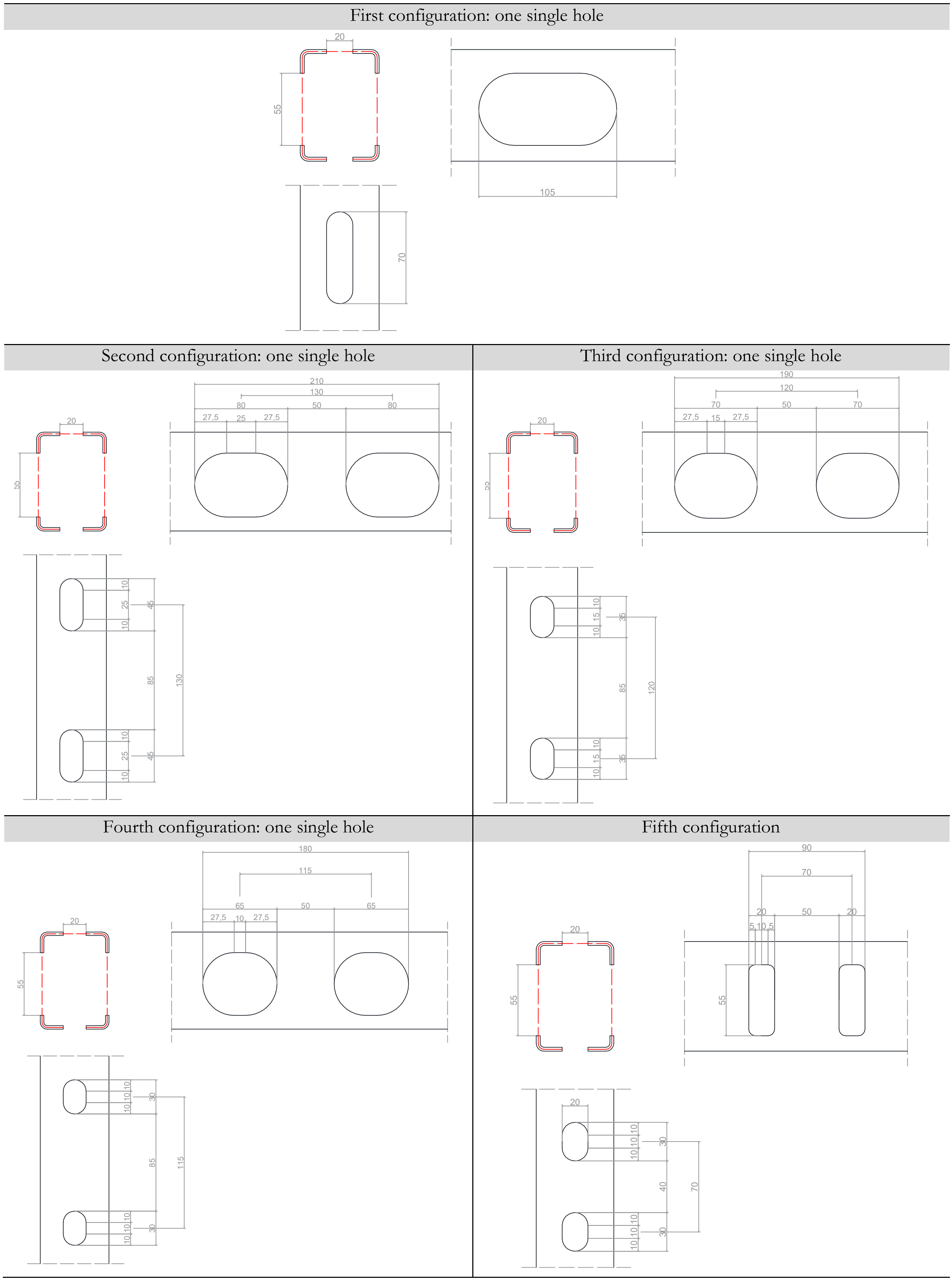


Figure 8-11: D1 diagonal: different patterns for holes.

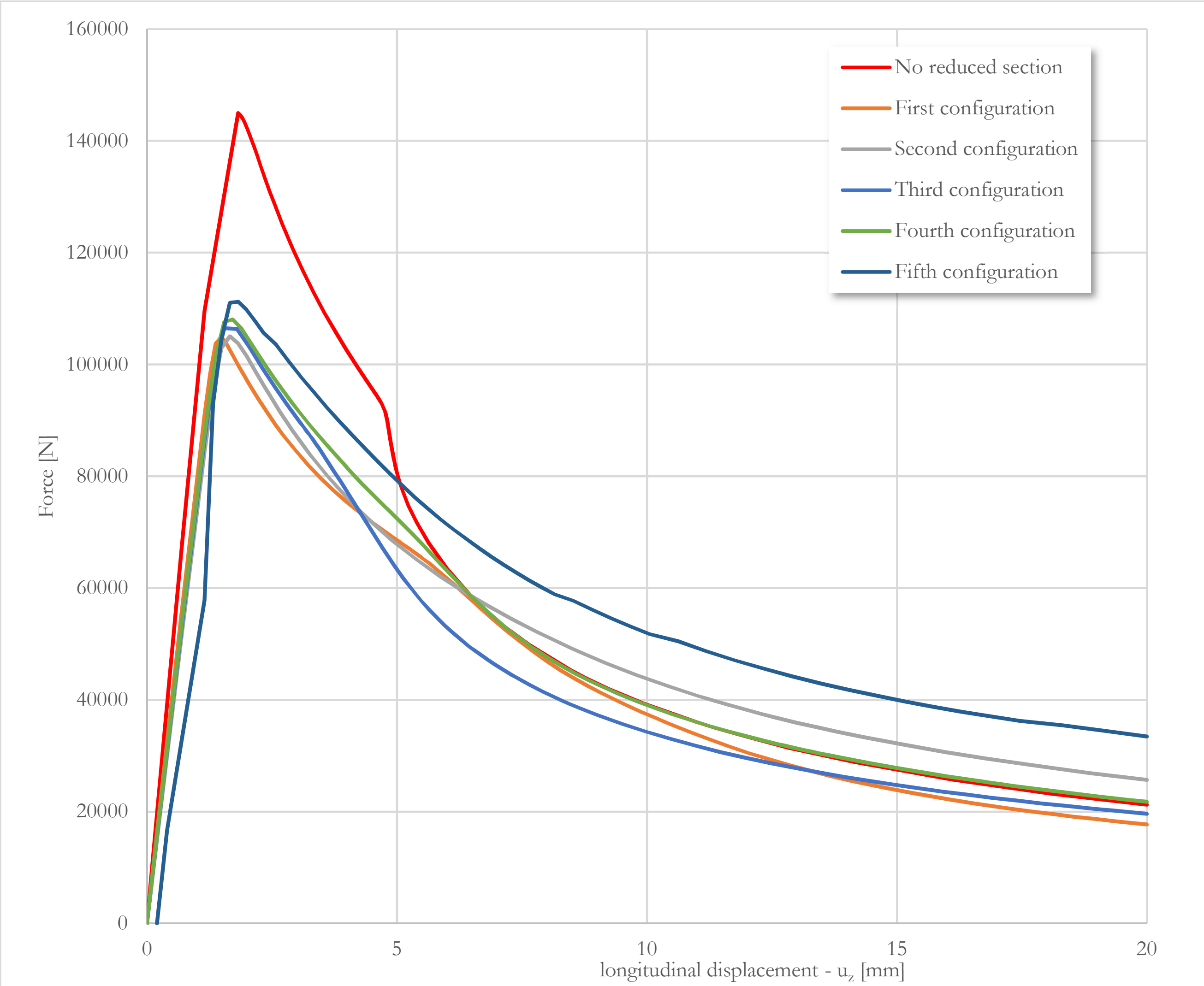
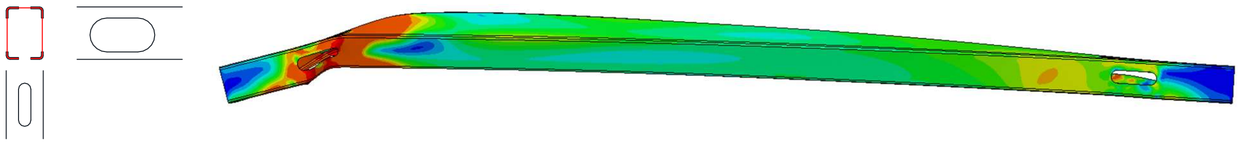
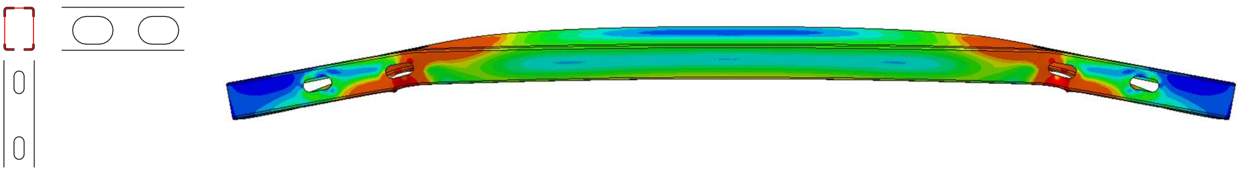
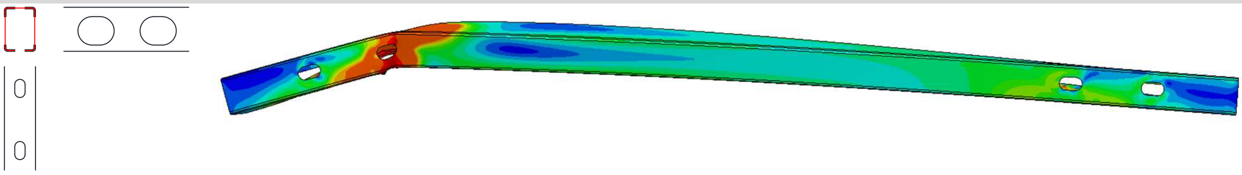


Figure 8-12: Force vs displacement of D1 diagonal, corresponding to increasing compression load and resulting from numerical simulations.

First configuration

Second confguration

Third configuration


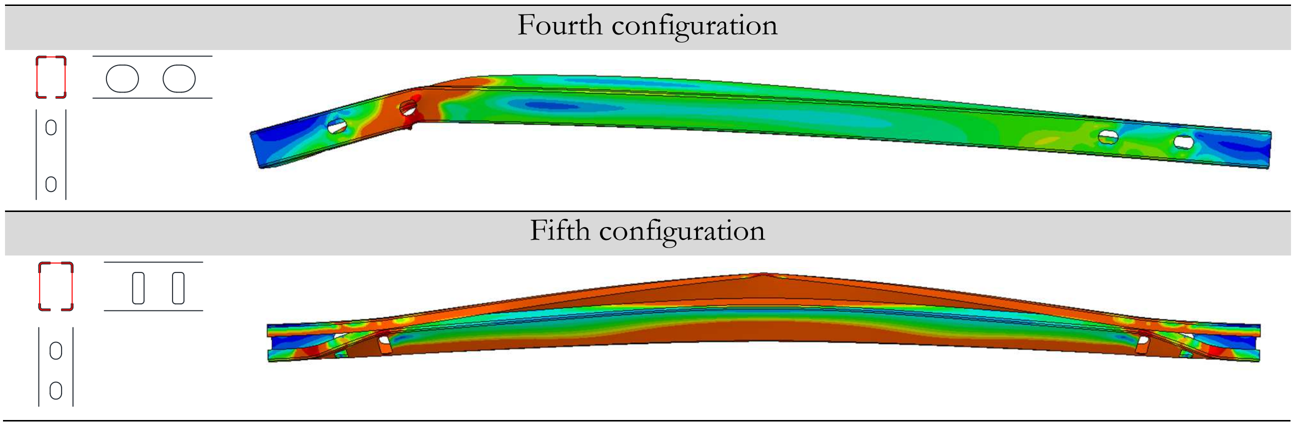
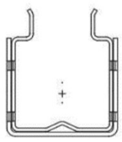
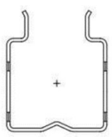
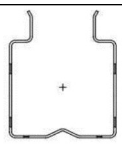


Figure 8-13: Buckling modes of D1 diagonal with various configuration of reduced-section zones.

Figure 8-12 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 holes change, the buckling resistance of the elements with the reduced-section parts is lower than the one of the element with no weakening, and the buckling resistance of all these weakened configurations are pretty similar. Looking at the buckling modes of the weakened elements (Figure 8-13), those of the first four configurations are quite different from the one of the complete diagonal (Figure 8-10), since deformations in compression are all concentrated in the reduced-section part of the elements. 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. Given that the configuration of the holes does not allow to reach the same buckling resistance of the complete element, but a global buckling mode is obtained with the fifth configuration (with the rectangular-shaped holes), guaranteeing a better behaviour in compression and cyclic load conditions, this last pattern is used and applied to all the other diagonals. The design of the diagonals is so completed. The other elements (uprights, transversal beams and spacers[8]) are designed using equations (8-II), in line with the capacity design rules within Eurocode 8 (prEN 1998-1-2:2019) for frames with concentric bracings. In this configuration, uprights are hinged at the base, and spacers are fixed to uprights.

Location	Section profile shape		Steel grade
Lower part: Reinforced		120x145x4.0 Cold-formed **Continuously holed**	S420MC
Lower part: Not reinforced		120x145x4.0 Cold-formed **Continuously holed**	S420MC
Upper part		Cold-formed 120x145x2.5	S420MC

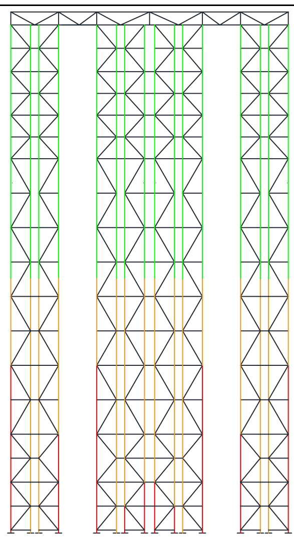


Figure 8-14: Upright cross-sections and characteristics.

[8] "Spacer" is the word from the technical jargon for the elements that connect consecutive and adjacent shelves.

8.4. Numerical validation of the design method

The numerical validation of the designed structure is made through the execution of NLTH analyses. The seismic input is the same that has been previously used for the vulnerability assessment of the case studies (§7.4.2). In particular, it consists of a set of 15 natural accelerograms that have been selected from NGA-West2 database (Bozorgnia et al. 2014) that match the target Conditional Spectra (CS) (J. Baker 2011; Lin, Haselton, and Baker 2013a; 2013b) at a 2475 years return period, or equivalently an exceedance probability of 2% in 50 years. Record selection has also been performed for a wider range of probabilities of exceedance. The structure is so tested through 9 amplifications of the seismic inputs, corresponding to a range of various probability of exceedance (from 60% to 0.1%). The numerical analyses are performed using Opensees® FEM software (Mazzoni et al. 2017). The 2D numerical model is made including geometrical non-linearities (P-Delta effect on columns) and modelling the non-dissipative elements as elastic through the *"elastic beam column"* element, to which an elastic material is associated. The dissipative elements are modelled through a *"truss"* element (an element working only in tension and in compression) to which a *pinching4 material* is associated. The *pinching4 material* is used to represent a load-deformation response that exhibits degradation under cyclic loading. The parameters for this material's definition are calibrated on the load-deformation diagrams obtained by the numerical simulations of the diagonal elements under cyclic loading, that have been previously performed using Abaqus® FEM software. All the other elements, that have been modelled as elastic, are checked in the post-process by the execution of the safety checks. In the following, major details about the calibration of pinching4 material are given, and the results of the Incremental Dynamic Analyses are showed.

8.4.1 Calibration of *pinching4* material to simulate cyclic behaviour of diagonals

The cyclic behaviour of the diagonals is obtained through the execution of numerical simulation using Abaqus® FEM software. The diagonals, whose reduced-section parts have been defined as indicated in the previous paragraph (§8.3), are modelled using quadrangular linear SHELL elements. The material's constitutive equation is taken from EN10149-2 (2013), assuming an elasto-plastic behaviour with isotropic hardening. Assuming that x and y are the axes in-plane with the cross-section of the element and z is the axis along longitudinal axis of the element, for one end section (the one where the load is not applicated), no translational displacements and rotation around the z axis are allowed. For the other end section (where the load is applied) only translational displacement along the z axis and rotations around the x and the y axis are allowed (translational displacement along longitudinal axis is from now on indicated as u_z). The global imperfection given to each weakened diagonal corresponds to the one that allows the intact profile to have a buckling load corresponding to the one calculated applying Eurocodes formulas for the elastic buckling load, according to the possible buckling modes that may characterize the profile (prEN 1993-1-3:2019). The cyclic load is applied at one of the two end sections of the element (the one that is allowed to move along the same direction of the element's longitudinal axis), and the displacement history consists of a series of stepwise increasing deformation cycles. Cycles are symmetric in peak displacements, and the peak amplitudes, in tension and in compression, are kept constant for three cycles, and then increase until a maximum displacement of 20 mm for the 1556 mm long diagonals (those from the 1st to the 4th level) and 21 mm for the 1989 mm long diagonal (those from the 5th to the 12th level). For both the 1556 mm long and the 1989 mm long diagonals, the first 15 cycles have a peak displacement given as a multiple of 1 mm (1 mm, 2 mm, 3 mm, 4 mm and 5 mm); then, the peaks are multiple of the characteristic yield displacement (around 3 mm for the 1556 mm long diagonals, and around 4 mm for the 1989 mm long diagonals) (Figure 8-15). Figure 8-17, Figure 8-18 and Figure 8-19 represent, respectively for diagonals D1 (level 1), D2 (levels 2 to 4), D5 (levels 5 to 7) and D8 (levels 8 to 12), the load-displacement diagram obtained by numerical simulations with the application of monotonic tension load (dark green line), monotonic compression load (red line), cyclic load (blue line). Besides these curves, the light green line and the orange line represent the backbone curves that control the pinching shape are used as a reference for the definition of the *pinching4* material. The material's characterisation parameters are calibrated through Opensees® FEM software (Mazzoni et al. 2017) by modelling each diagonal forced by a cyclic load that share the same displacement history used in the previous numerical simulations (Figure 8-15). The diagonal is modelled through a *truss* element, to which the *pinching4* material is associated. The boundary conditions are the same assumed in the previous numerical simulations. Table 8-11 gathers the material's characterisation parameters, whose definition is given in Mazzolani et al. (2017). Figure 8-20, Figure 8-21, Figure 8-22, and Figure 8-23 show that the calibrated pinching model can capture the cyclic hysteretic behaviour that is obtained in the previous numerical simulations, including strength degradation.

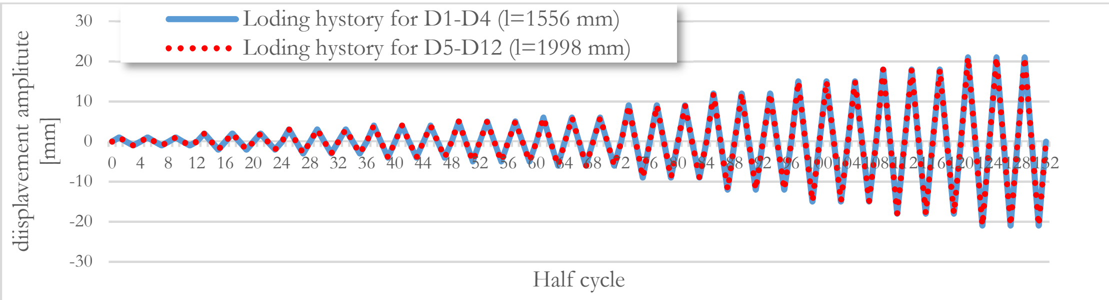


Figure 8-15: Displacement history for cyclic loading adopted for numerical simulations on diagonals.

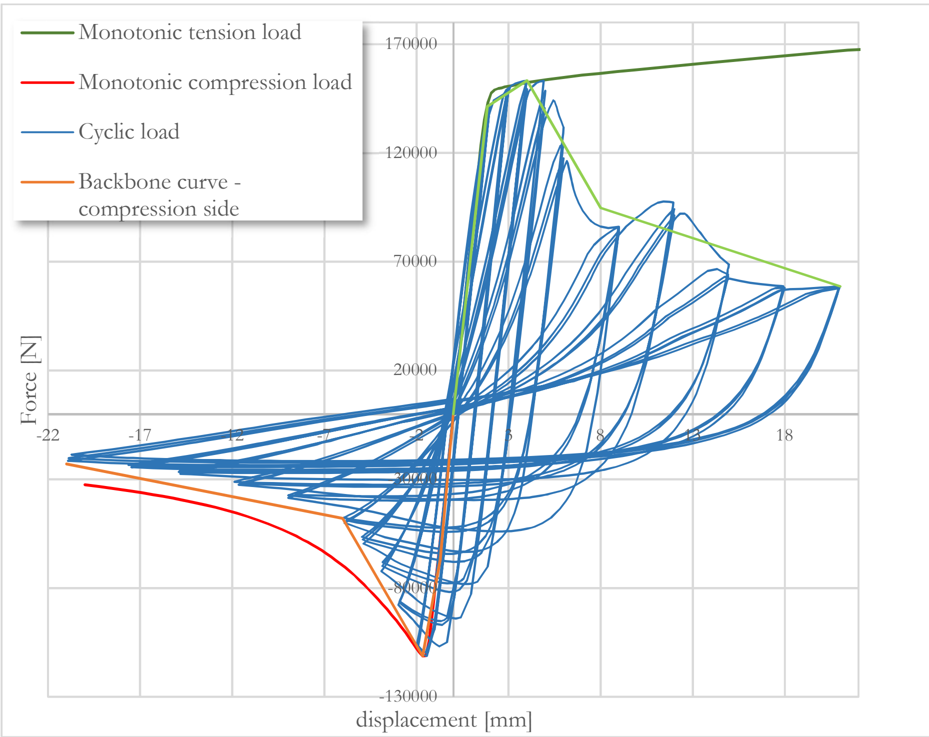


Figure 8-16: Load-displacement diagram for D1, corresponding to monotonic tension and compression load, and cyclic load, and resulting from numerical simulations.

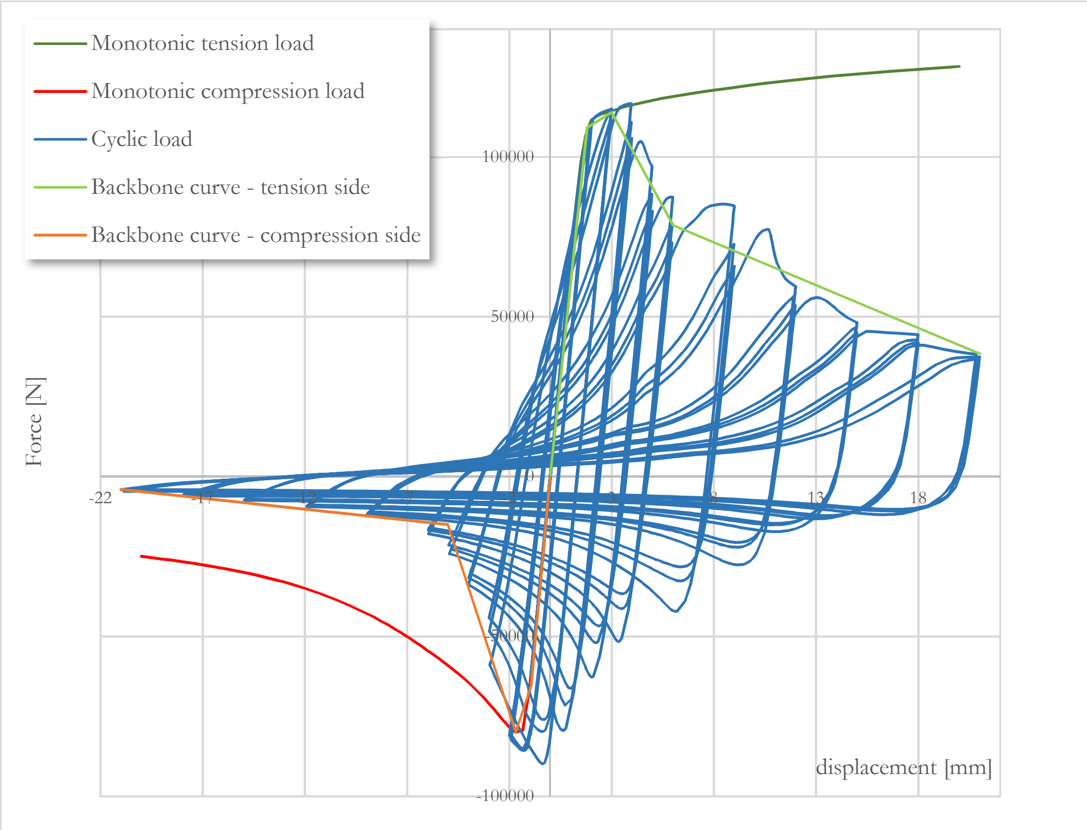


Figure 8-17: Load-displacement diagram for D2-D4, corresponding to monotonic tension and compression load, and cyclic load, and resulting from numerical simulations.

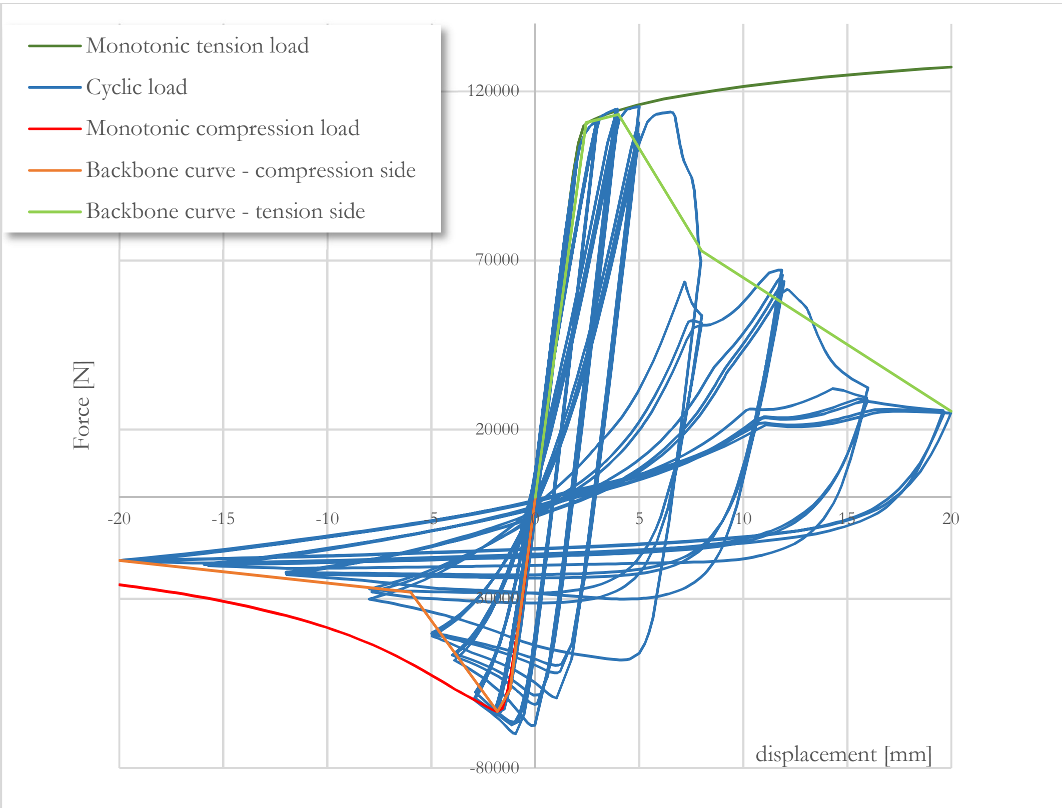


Figure 8-18: Load-displacement diagram for D5-D7, corresponding to monotonic tension and compression load, and cyclic load, and resulting from numerical simulations.

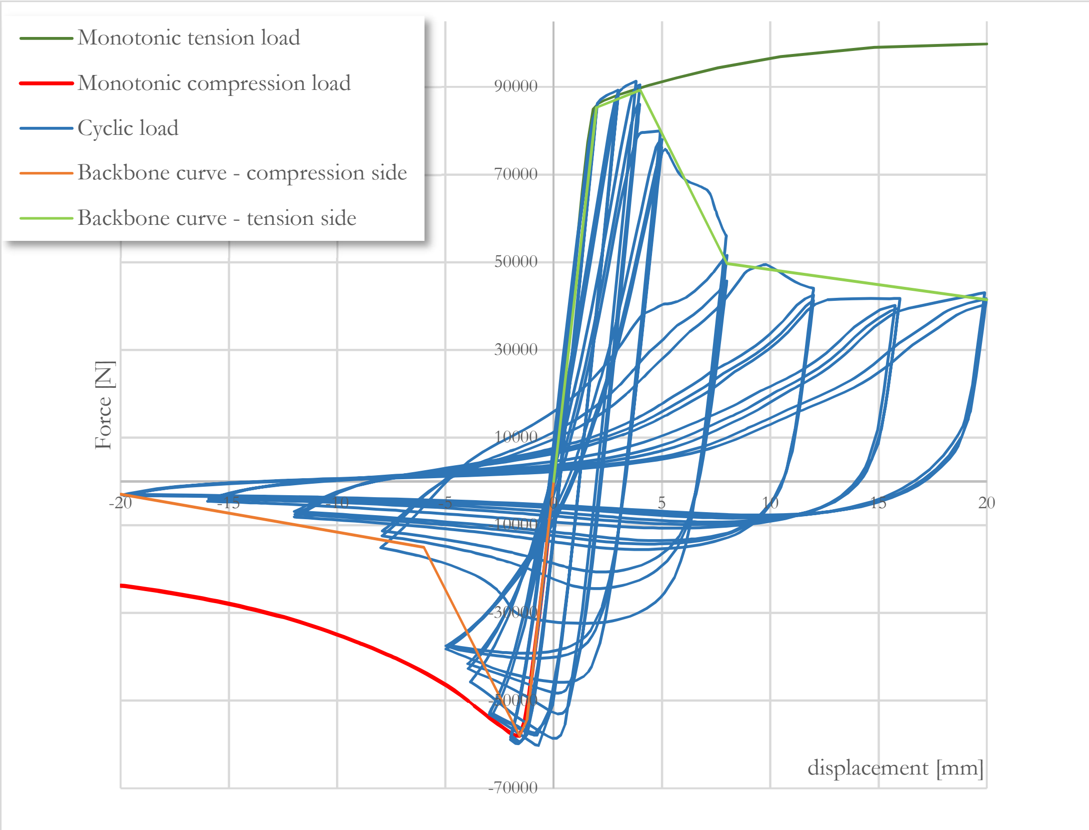


Figure 8-19: Load-displacement diagram for D8-D12, corresponding to monotonic tension and compression load, and cyclic load, and resulting from numerical simulations.

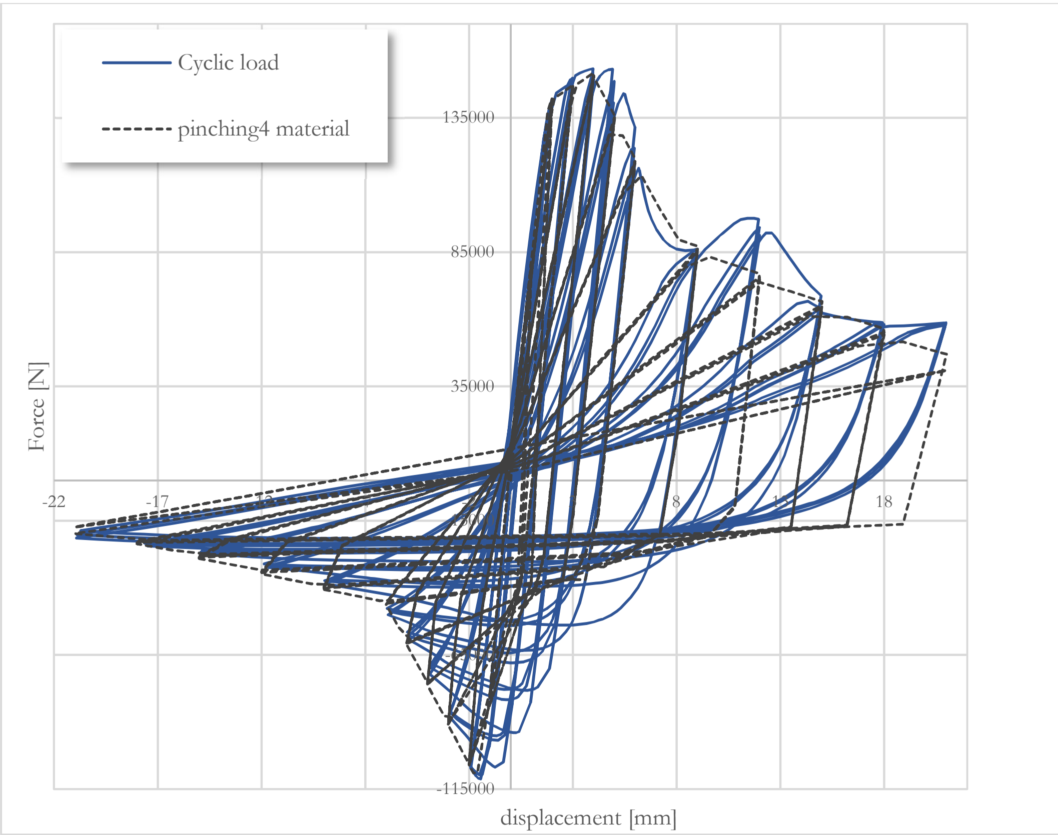


Figure 8-20: Simulated hysteretic response of axial behaviour of D1 diagonal (level 1).

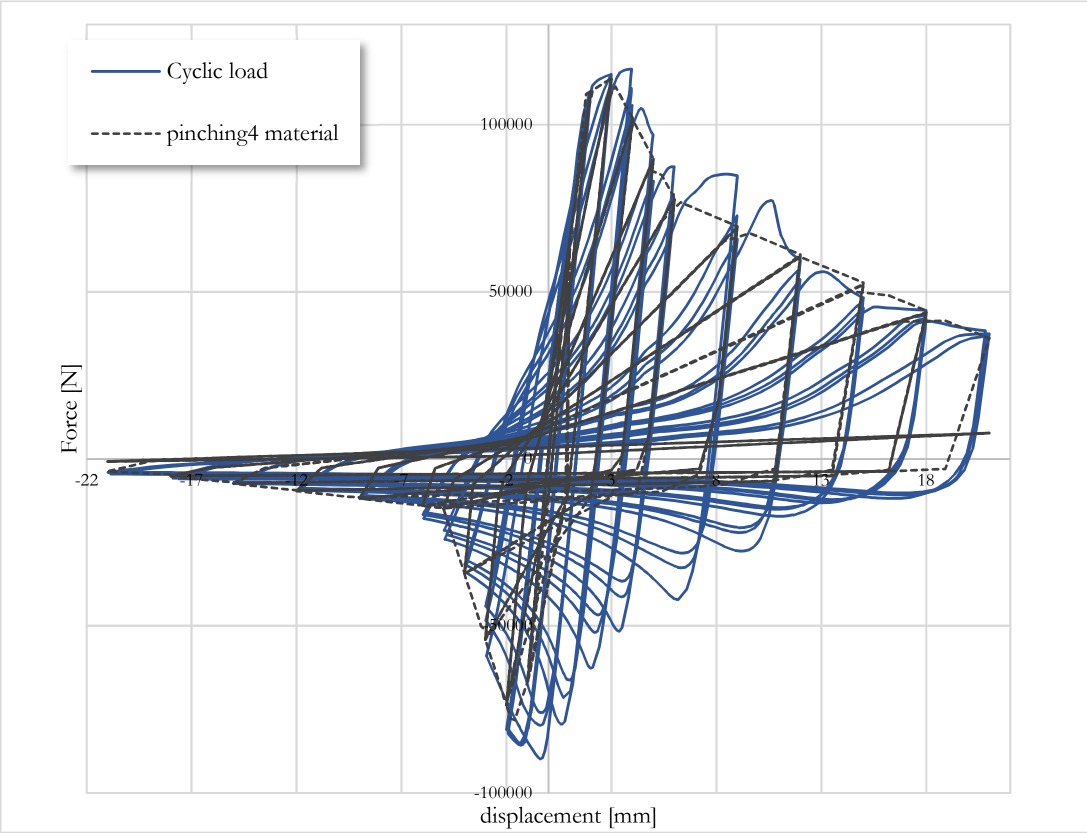


Figure 8-21: Simulated hysteretic response of axial behaviour of D2 diagonal (levels 2 to 4).

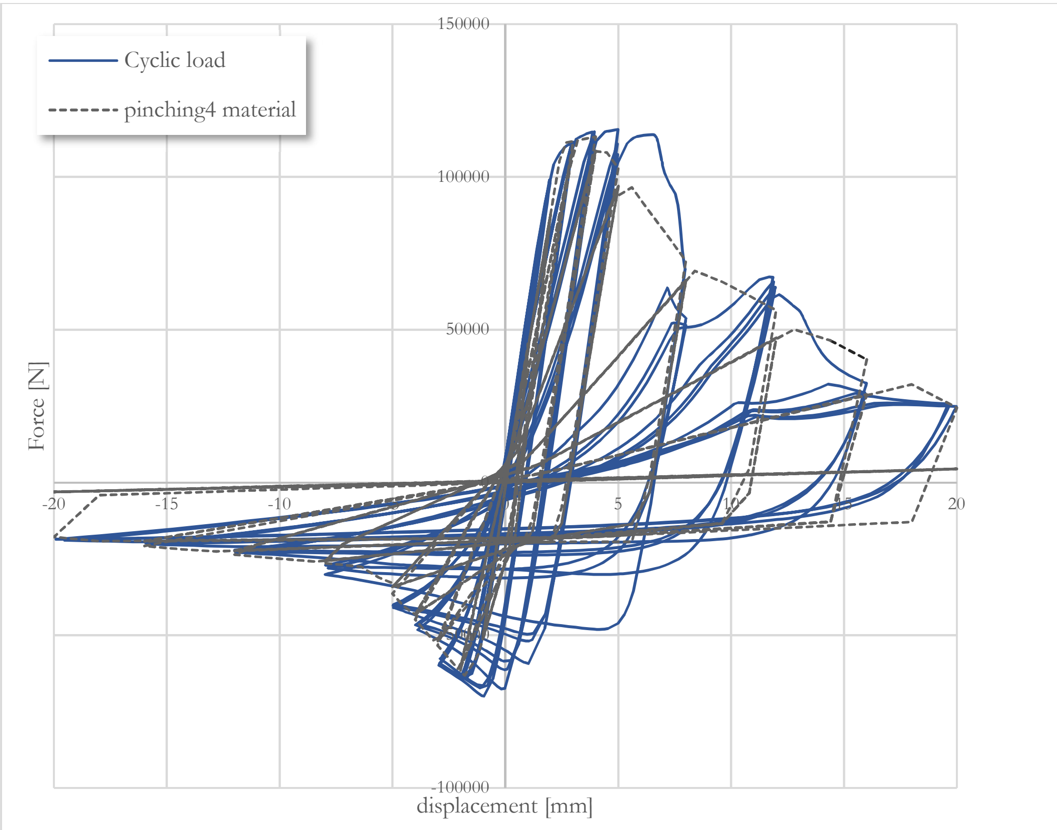


Figure 8-22: Simulated hysteretic response of axial behaviour of D5 diagonal (levels 5 to 7).

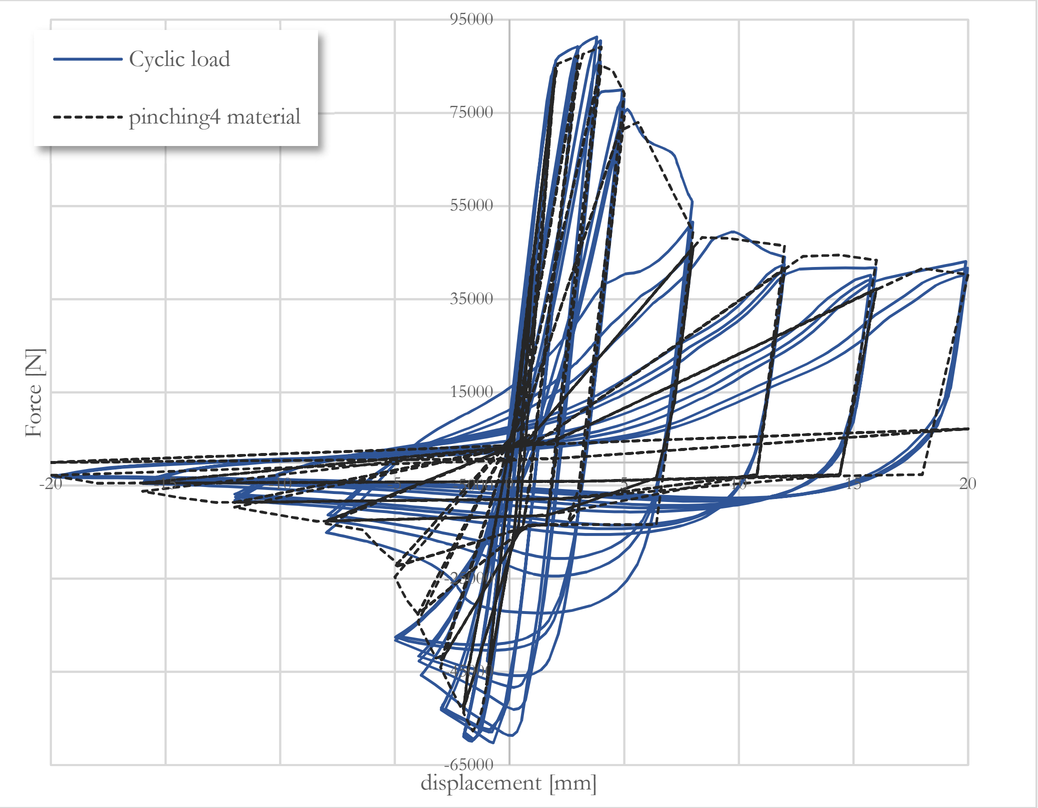


Figure 8-23: Simulated hysteretic response of axial behaviour of D8 diagonal (levels 8 to 12).

Table 8-11: Parameters for the definition of pinching4 material for all the diagonals.

	D1	D2	D5	D8
ePf1 [N]	141412	109180	110820	85287
ePd1 [mm]	1.85	1.79	2.44	1.96
ePf2 [N]	153262	113932	113241	89275
ePd2 [mm]	4.00	3.00	4.00	4.00
ePf3 [N]	94780	78650	72882	49722
ePd3 [mm]	8.00	6.00	8.00	8.00
ePf4 [N]	58698	38434	25439	41475
ePd4 [mm]	21.00	21.00	20.00	20
eNf1 [N]	-66742	-65415	-56801	-54336
eNd1 [mm]	-0.7	-0.93	-1.21	-1.23
eNf2 [N]	-111238	-79955	-63368	-58023
eNd2 [mm]	-1.65	-1.69	-1.82	-1.57
eNf3 [N]	-48000	-15000	-28000	-15000
eNd3 [mm]	-6.00	-5.00	-6.00	-6.00
eNf4 [N]	-25000	-4007	-18617	-2912
eNd4 [mm]	-20.00	-20.00	-20.00	-20.00
rDispP	0.03	-0.1	0.1	0.1
rForceP	0.10	0.10	0.10	0.10
uForceP	-0.5	-0.1	-0.9	-0.9
rDispN	-0.5	-0.9	-0.2	-0.2
rForceN	-2	-1	-1	-1
uForceN	0.8	0.7	0.7	0.9
gK1 gK2 gK3 gK4 gKLim	0.1 1.0 1.0 1.0 0.9	0.1 1.0 1.0 1.0 0.9	0.1 1.0 1.0 1.0 0.9	0.1 1.0 1.0 1.0 0.9
gD1 gD2 gD3 gD4 gDLim	0.0 0.1 1.0 1.0 0.8	0.0 0.1 1.0 1.0 0.3	0.15 0.3 1.0 1.0 0.3	0.15 0.3 1.0 1.0 0.3
gF1 gF2 gF3 gF4 gFLim	1 20 50 1.3 0.3	1 20 50 1.5 0.8	1 20 50 1.5 0.8	1 20 50 1.5 0.8
gE	350	400	400	400

It is necessary to consider that the hysteretic behaviour that is calibrated and inserted in the model results from a displacement history where the amplitude of cycles grows gradually, and many cycles are performed before getting to the ultimate deformation of the diagonal (Figure 8-15). This indeed affects the behaviour of the element, which experience a rapid decrease of resistance, especially in tension, that can be appreciated looking at the difference between the monotonic curve in tension and the backbone curve in tension, that replicate the size of the hysteretic behaviour on the tension side (Figure 8-16, Figure 8-17, Figure 8-18, and Figure 8-20, the monotonic curve is the dark green one, while the backbone curve in tension is the light green one). If the displacement history were different, the hysteretic behaviour would change, too. As an example, if the displacement demand, instead of growing gradually, would consists in an immediate big amplitude cycle in tension, the behaviour of the element would be elasto-plastic, following the path of the monotonic curve in tension. If the displacement demand would be this one, but the element's behaviour would be modelled for the gradually increasing displacement history here used, the behaviour of the element would follow the corresponding backbone curve, and this would not be representative of the actual behaviour of the element. Besides the dropping of resistance, the dissipated energy would be largely underestimated.
In awareness of this, and still not knowing the displacement demand for the structure, the hysteretic behaviour of the diagonals, that is adopted for the execution of the NLTH analyses, has been defined how previously illustrated, adopting the gradually increasing displacement history (Figure 8-15), that is in any case much safer than the other opposed possibility. These numerical analyses are also helpful to evaluate the effective displacement demand and, in case, re-adjust the definition of the hysteretic behaviour of the diagonals.

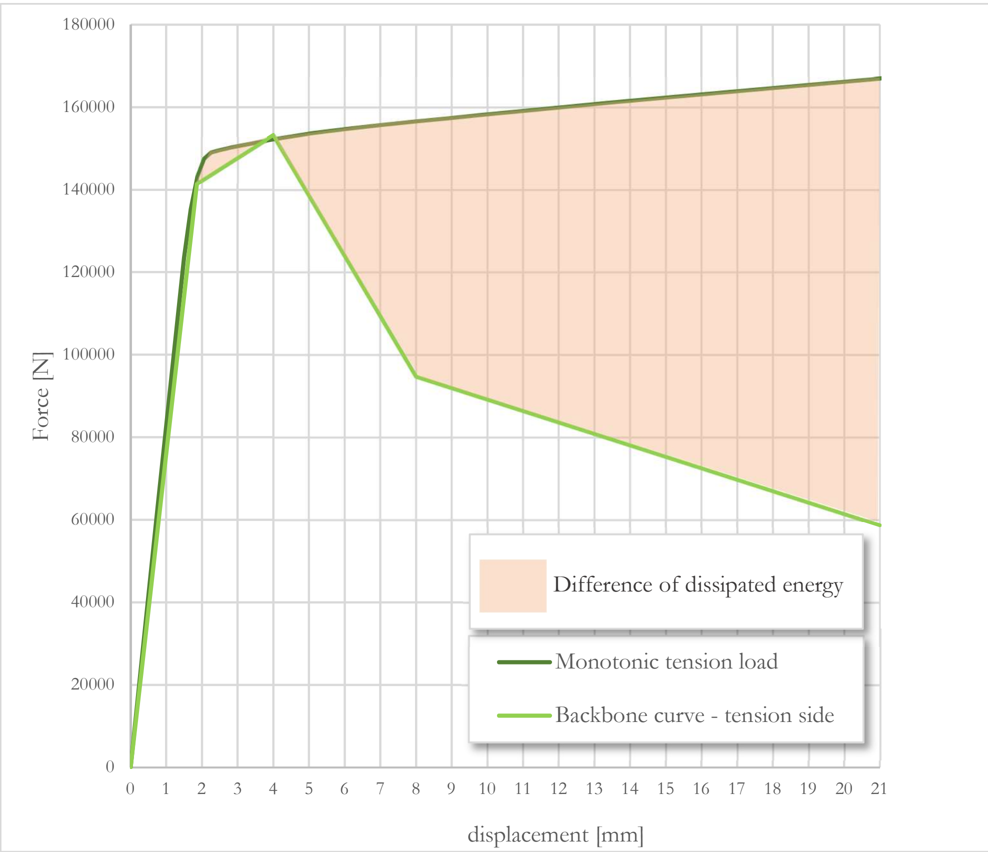


Figure 8-24: Comparison of the element's behaviour using different displacement history to represent the hysteretic behaviour.

8.4.2 **Non-Linear Time History Analyses**

NLTH analyses are performed to validate the designed structure (Vamvatsikos and Cornell 2002). The seismic input consists of a set of 15 natural accelerograms that have been selected from NGA-West2 database (Bozorgnia et al. 2014) that match the target Conditional Spectra (CS) (J. Baker 2011; Lin, Haselton, and Baker 2013a; 2013b) at a 2475 years return period, or equivalently an exceedance probability of 2% in 50 years. Record selection has also been performed for a wider range of probabilities of exceedance (9 in total), varying from 60% to 0.1%. The structure is so tested through 9 amplification of the seismic inputs (Scale Factors SF) for a total of 135 accelerograms. The numerical analyses are performed using Opensees® FEM software (Mazzoni et al. 2017). The 2D numerical model is made including geometrical non-linearities (P-Delta effect on columns) and modelling the non-dissipative elements as elastic through the *"elastic beam column"* element, to which an elastic material is associated. The dissipative elements are modelled through a *"truss"* element (an element working only in tension and in compression) to which a *pinching4 material* is associated, that is calibrated on previous numerical simulations performed (§8.3). Paragraph 8.4.1 shows the calibration of the parameters for the definition of the material. The other non-dissipative elements, which are modelled as elastic, are controlled in the post-process by executing the safety checks.

Figure 8-25 shows the IDA curves obtained from the NLTH analysis, plotting the maximum base shear versus the displacement at the top of the structure: the grey lines indicate the IDA curves for all the 15 accelerograms, while the red line represents, for each SF, the medium value of the maximum base shear and the maximum displacement. The red circles represent each scale factor. In the figure, the part of the curves with displacements major than 1 meter has been excluded since they are not consistent with the structure's acceptable deformations.

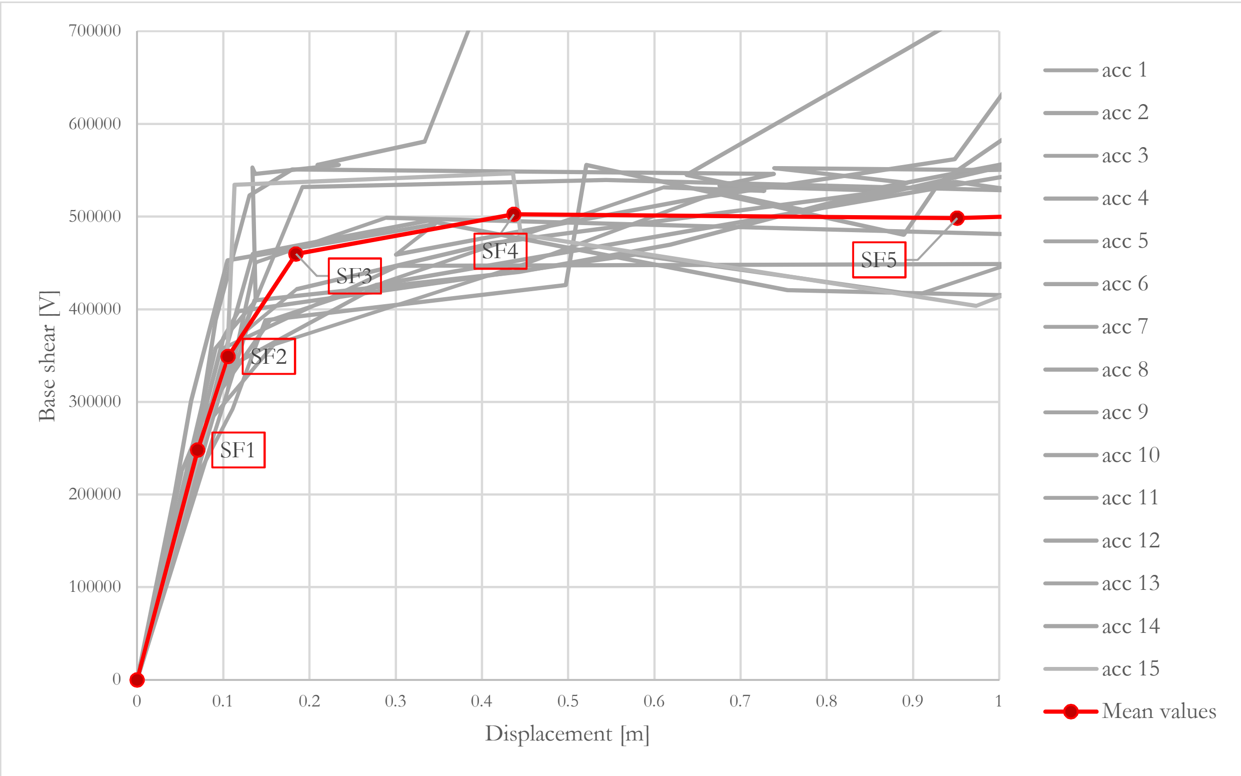
Looking at what happens at each step (Figure 8-26):

- at SF1 (60%), all the elements are in the elastic field;
- at SF2 (30%), some diagonals between 6 and 8 level reach yielding deformation;
- at SF3 (10%), which is beyond the design probability of exceedance (that is 20%), between the 1^{st} and the 4^{th} levels, all the central shelves' and some of the lateral shelves' diagonals have reached the yielding deformation. At the 5^{th} level, all the diagonals reach the yielding deformation. Between the 6^{th} and 9^{th} levels, all the diagonals reach the ultimate deformation. This also happens to the diagonal of the central shelve belonging to the 10^{th} and 11^{th} levels.

  Regarding the uprights, between the 5^{th} and the 7^{th} level, the elements indicated with the green triangle fail due to axial compression and bending force.
- at SF4 (5%), which is beyond the design probability of exceedance (that is 20%), most diagonals reach the ultimate deformation. Most uprights fail due to axial compression and bending force.

Focusing on SF3, it is obvious that the diagonals between the 6^{th} and the 11^{th} level request a higher ductility level. Looking at the relative displacements of the structure (Figure 8-27), it can be noticed that in those levels they increase significantly, probably because it is the zone of the structure that is the furthest from the reciprocally constrained areas (the base and the top, where the roof truss connects all the shelves). It is also confirmed by the main modal shapes of the structure (Figure 8-28). From Figure 8-27, it can also be noticed that the displacement demand for the central shelves is mayor than the one of the lateral shelves. This implies that the initial hypothesis of equal ductility request for all the load levels is not applicable for these structures. Regarding the uprights, their premature failure is concentrated in the levels where diagonals reach their ultimate failure and where there is an increment of relative displacement. Table 8-12 shows the mean D/C ratios of uprights at SF3 for axial compression (C), bending moment (B) and their combination (C+B) (please note that maximum axial compression and bending may not occur at the same time). Looking at those D/C ratios, it can be noticed that in the levels where the relative displacement demand increase, a relevant contribution to D/C ratios is given by bending. The increment of bending moment at those levels is because diagonals take very low forces when they reach those deformations (see Figure 8-20 to Figure 8-23). Still, uprights continue to take load because the consecutive shelves are here coupled through the spacers, which are connected to uprights through bending resistant connections. Based on this consideration, and assuming that a limited value of section modulus characterizes uprights, the approach used for the design of these elements (where only the axial tension/compression component due to seismic action $N_{Ed,E}$ is amplified by the Ω factor, see equation (8-II)) may be not appropriate for these structures. So, it may be correct and precautionary to amplify also the value of the bending force component due to seismic action $M_{Ed,E}$. The more appropriate design formulas for uprights may be the following:

$$N_{Ed} = N_{Ed,G} + \Omega \cdot N_{Ed,E}$$
$$M_{Ed} = M_{Ed,G} + \Omega \cdot M_{Ed,E} \qquad \text{(8-VIII)}$$
$$V_{Ed} = V_{Ed,G} + V_{Ed,E}$$



	Probability of exceedance [%]	Base shear [N]	Displacement [m]
SF 1	60	248147	0.07
SF 2	30	349163	0.10
SF 3	10	459740	0.18
SF 4	5	502450	0.43
SF 5	2	498420	0.95

Figure 8-25: IDA curves: Total base shear versus displacement at the top of the structure.

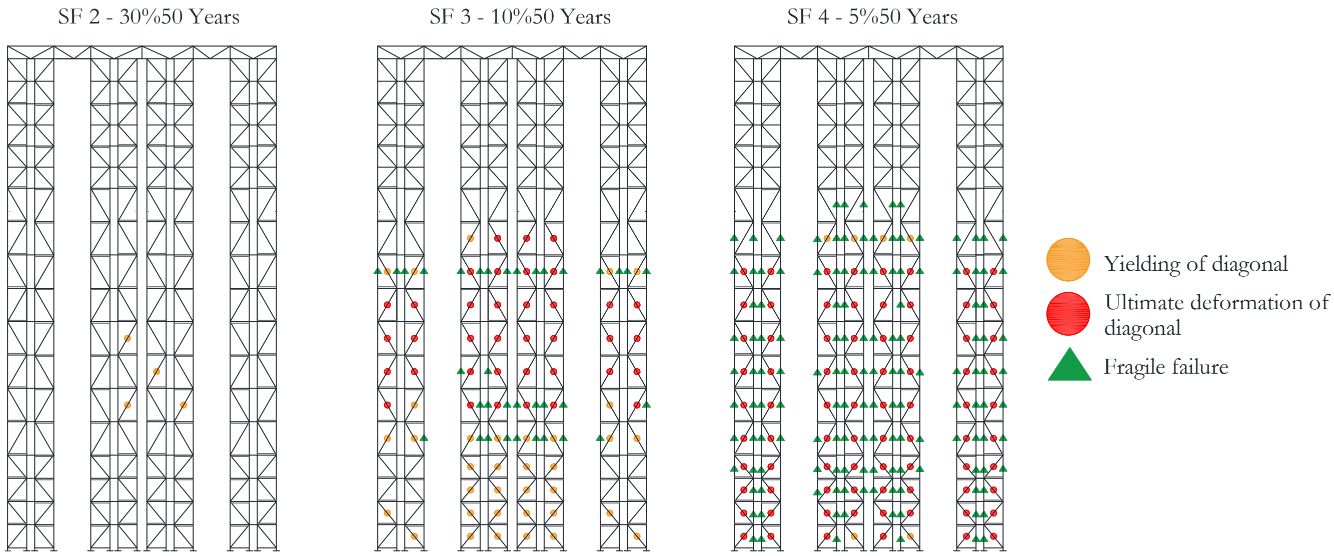


Figure 8-26: Graphic representation of ductile and fragile failures for SF2, SF3 and SF4 (structure with fixed spacer).

Table 8-12: D/C ratios of uprights belonging to central shelves, SF3: C is 'axial compression', B is "bending moment'.

	Upright 5			Upright 6			Upright 7			Upright 8			Upright 9			Upright 10			Upright 11			Upright 12		
	C	B	C+B	C	B	C+B	C	B	C+B	C	B	C+B	C	B	C+B	C	B	C+B	C	B	C+B	C	B	C+B
1	0.73	0.32	0.98	0.66	0.18	0.79	0.41	0.11	0.50	0.44	0.33	0.74	0.45	0.34	0.74	0.40	0.11	0.49	0.66	0.17	0.77	0.70	0.30	0.95
2	0.65	0.36	0.91	0.69	0.24	0.82	0.42	0.22	0.52	0.37	0.31	0.60	0.38	0.30	0.59	0.41	0.22	0.52	0.68	0.25	0.81	0.64	0.35	0.88
3	0.63	0.27	0.79	0.65	0.26	0.79	0.65	0.43	0.97	0.37	0.36	0.61	0.38	0.36	0.64	0.62	0.42	0.91	0.64	0.26	0.77	0.62	0.28	0.77
4	0.61	0.33	0.78	0.47	0.45	0.81	0.62	0.38	0.89	0.35	0.39	0.63	0.36	0.40	0.63	0.59	0.37	0.83	0.50	0.43	0.81	0.60	0.32	0.77
5	0.60	0.28	0.72	0.44	0.37	0.71	0.66	0.36	0.95	0.57	0.37	0.83	0.59	0.38	0.89	0.62	0.36	0.90	0.47	0.35	0.72	0.59	0.29	0.72
6	0.50	0.44	0.75	0.48	0.44	0.84	0.63	0.50	0.99	0.54	0.44	0.88	0.56	0.44	0.91	0.59	0.51	0.97	0.49	0.46	0.85	0.51	0.42	0.73
7	0.49	0.42	0.73	0.45	0.49	0.81	0.54	0.46	0.91	0.57	0.44	0.91	0.59	0.44	0.93	0.52	0.46	0.89	0.46	0.49	0.83	0.49	0.42	0.72
8	0.49	0.56	0.85	0.45	0.55	0.95	0.51	0.52	0.92	0.54	0.53	0.94	0.55	0.53	0.92	0.49	0.52	0.91	0.46	0.55	0.92	0.49	0.57	0.84
9	0.48	0.58	0.88	0.43	0.46	0.79	0.63	0.72	1.20	0.54	0.73	1.19	0.56	0.74	1.16	0.61	0.71	1.23	0.44	0.47	0.81	0.48	0.59	0.89
10	0.47	0.66	0.90	0.48	0.69	1.13	0.60	0.70	1.17	0.51	0.67	1.09	0.53	0.66	1.09	0.58	0.70	1.19	0.49	0.70	1.10	0.47	0.66	0.92
11	0.45	0.49	0.72	0.45	0.52	0.85	0.52	0.68	1.09	0.53	0.65	1.01	0.54	0.65	1.04	0.49	0.68	1.01	0.46	0.52	0.88	0.46	0.48	0.71
12	0.45	0.53	0.74	0.46	0.72	1.03	0.49	0.71	1.12	0.50	0.72	1.06	0.51	0.72	1.09	0.46	0.71	1.03	0.47	0.72	1.11	0.46	0.52	0.75
13	0.44	0.41	0.66	0.43	0.64	0.93	0.49	0.65	1.04	0.39	0.65	0.90	0.40	0.65	0.97	0.47	0.64	0.96	0.44	0.64	0.99	0.44	0.42	0.68
14	0.68	0.60	1.03	0.36	0.67	0.90	0.46	0.68	1.04	0.37	0.67	0.90	0.38	0.67	0.99	0.44	0.68	0.94	0.37	0.67	0.96	0.69	0.60	1.00
15	0.66	0.38	0.82	0.33	0.58	0.81	0.37	0.66	0.94	0.37	0.62	0.88	0.38	0.62	0.92	0.35	0.65	0.86	0.34	0.58	0.81	0.66	0.38	0.81
16	0.65	0.45	0.84	0.38	0.72	1.00	0.34	0.71	0.97	0.34	0.72	0.95	0.35	0.72	0.97	0.33	0.71	0.90	0.36	0.72	1.00	0.66	0.45	0.83
17	0.62	0.39	0.79	0.35	0.63	0.89	0.40	0.52	0.80	0.34	0.53	0.78	0.34	0.52	0.75	0.39	0.53	0.84	0.34	0.63	0.88	0.63	0.38	0.80
18	0.44	0.39	0.69	0.34	0.49	0.75	0.37	0.49	0.75	0.31	0.48	0.73	0.32	0.48	0.72	0.36	0.49	0.77	0.32	0.49	0.70	0.45	0.38	0.67
19	0.77	0.63	1.19	0.59	0.95	1.41	0.65	1.05	1.50	0.55	1.02	1.45	0.55	1.02	1.45	0.64	1.04	1.55	0.56	0.95	1.32	0.78	0.63	1.16
20	0.73	0.63	1.15	0.54	1.06	1.42	0.53	0.73	1.14	0.59	0.70	1.15	0.57	0.69	1.14	0.56	0.75	1.11	0.51	1.07	1.36	0.74	0.63	1.12
21	0.73	0.60	1.12	0.56	0.80	1.25	0.48	0.78	1.14	0.55	0.79	1.23	0.52	0.79	1.17	0.51	0.77	1.15	0.53	0.78	1.18	0.74	0.60	1.09
22	0.68	0.39	0.93	0.50	0.64	1.05	0.50	0.56	0.86	0.49	0.59	1.01	0.48	0.58	0.98	0.53	0.56	0.99	0.49	0.63	1.01	0.68	0.39	0.86
23	0.55	0.39	0.84	0.45	0.55	0.92	0.45	0.59	0.88	0.45	0.59	0.96	0.44	0.59	0.99	0.48	0.60	0.96	0.40	0.55	0.84	0.51	0.39	0.76
24	0.52	0.21	0.65	0.39	0.54	0.79	0.34	0.38	0.65	0.38	0.39	0.68	0.38	0.40	0.67	0.34	0.38	0.65	0.35	0.54	0.78	0.47	0.22	0.60
25	0.52	0.21	0.66	0.41	0.40	0.78	0.30	0.40	0.66	0.34	0.43	0.69	0.34	0.43	0.69	0.30	0.39	0.64	0.40	0.40	0.80	0.47	0.21	0.62
26	0.50	0.15	0.60	0.38	0.37	0.71	0.20	0.32	0.45	0.31	0.44	0.71	0.31	0.44	0.72	0.21	0.33	0.46	0.37	0.36	0.72	0.45	0.16	0.55
27	0.37	0.15	0.48	0.23	0.32	0.52	0.18	0.33	0.44	0.29	0.38	0.63	0.29	0.38	0.64	0.19	0.34	0.44	0.23	0.32	0.52	0.33	0.15	0.44
28	0.36	0.08	0.42	0.23	0.22	0.44	0.16	0.25	0.37	0.22	0.31	0.47	0.22	0.30	0.48	0.15	0.25	0.37	0.22	0.23	0.44	0.33	0.08	0.38
29	0.35	0.08	0.40	0.21	0.24	0.44	0.12	0.17	0.24	0.23	0.25	0.46	0.24	0.25	0.47	0.13	0.17	0.25	0.20	0.25	0.45	0.32	0.08	0.36
30	0.34	0.03	0.36	0.14	0.20	0.32	0.10	0.20	0.24	0.22	0.26	0.45	0.23	0.26	0.47	0.11	0.20	0.26	0.13	0.20	0.32	0.31	0.03	0.33
31	0.33	0.09	0.41	0.16	0.18	0.32	0.10	0.19	0.27	0.16	0.23	0.36	0.17	0.23	0.38	0.10	0.19	0.26	0.15	0.18	0.33	0.31	0.09	0.36
32	0.34	0.09	0.42	0.14	0.18	0.31	0.08	0.13	0.18	0.18	0.18	0.34	0.18	0.18	0.35	0.07	0.13	0.18	0.15	0.19	0.33	0.30	0.09	0.36
33	0.37	0.03	0.40	0.10	0.15	0.22	0.08	0.15	0.19	0.17	0.20	0.35	0.18	0.20	0.36	0.07	0.15	0.18	0.10	0.15	0.23	0.34	0.03	0.36
34	0.37	0.03	0.39	0.15	0.10	0.23	0.09	0.09	0.16	0.13	0.10	0.21	0.14	0.10	0.21	0.09	0.09	0.17	0.15	0.10	0.20	0.34	0.03	0.36

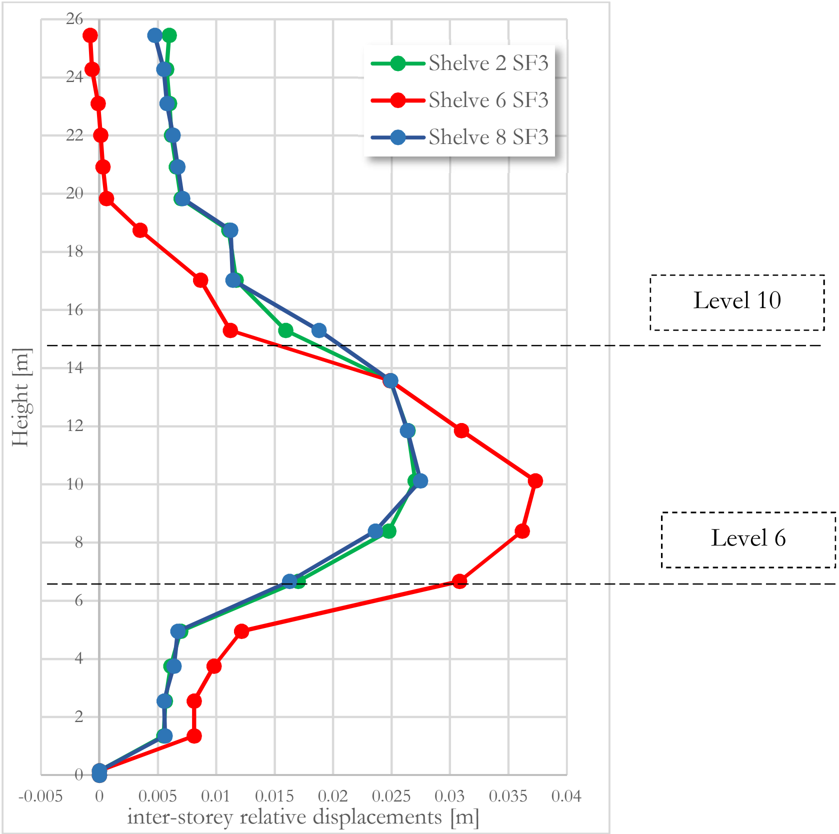


Figure 8-27: Inter-storey relative displacement relative to SF3.

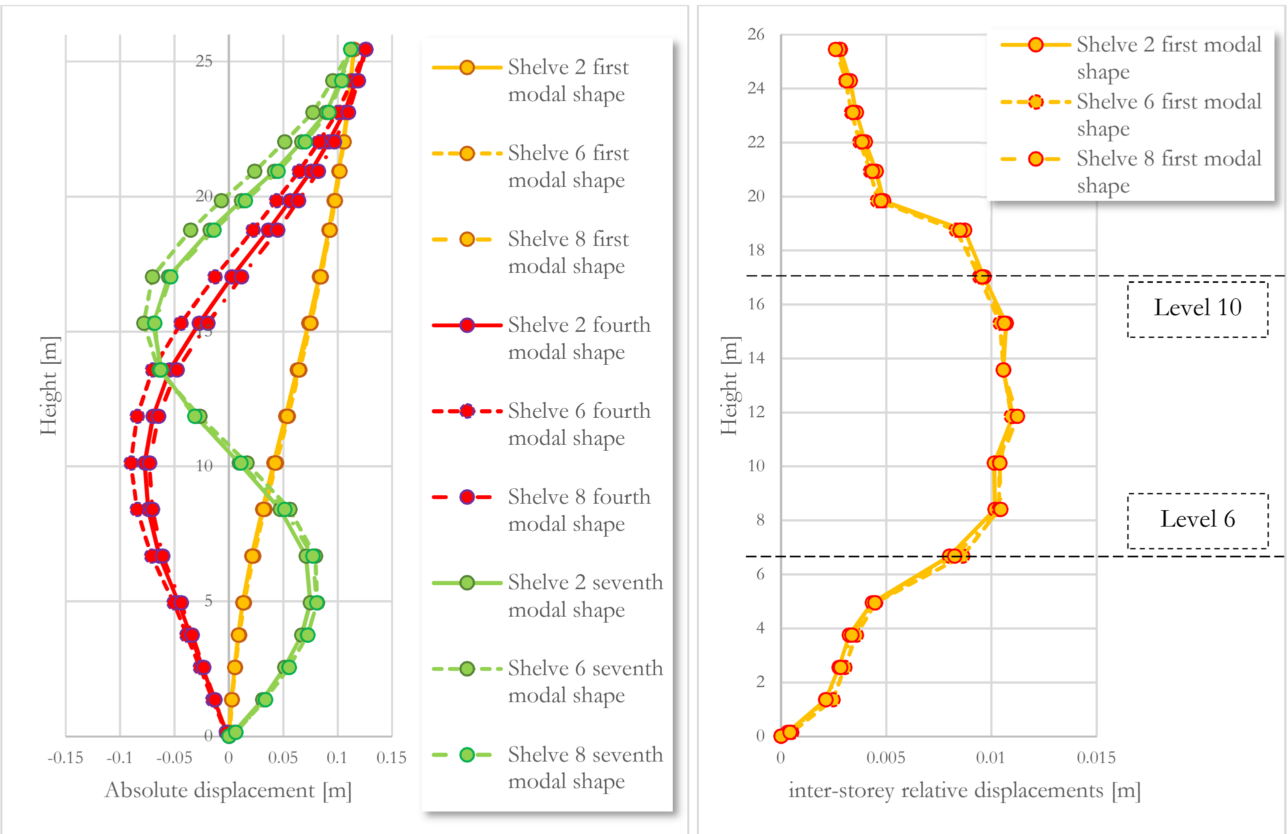


Figure 8-28: Absolute displacement and inter-storey relative displacement relative corresponding to the main modal shapes.

Given these findings, an improved design of the structure is carried out, where the main changes are listed in the following:

- increment of the ductility of the diagonals between level 6 and level 12;
- spacers are hinged to uprights;
- uprights are designed using the equations (8-VIII), resulting in the new distribution illustrated within Figure 8-29.

Changing the spacers' boundary conditions makes the structure more flexible (the fundamental period goes from 1.44s to 1.90s), and the distribution of forces in the uprights change since the consecutive shelves are not coupled anymore. Figure 8-30 gives a graphical representation of the mechanisms happening at SF2 and SF3 (at SF1, all the elements remain in the elastic field):

- at SF2 (30%), some diagonals between 8 and 9 level reach yielding deformation;
- at SF3 (10%), one diagonal at the 1st level of the central shelve and the diagonal of the central shelve at the 5th level reach yielding deformation. Between the 6th and the 10th level, diagonals reach ultimate deformation. This is happening because, being the structure more flexible, the ductility request increases with respect to the previous configuration. Besides, looking at the load-deformation diagrams of the dissipative elements, in most cases it happens that the diagonal, after little amplitude cycles in the elastic field, has a big amplitude cycle in tension (Table 8-13). According to how *pinghing4* material is modelled, the load-deformation path follows the backbone curve in tension. The backbone curve represents the envelope of the cyclic behaviour of the diagonal due to a different displacement history (Figure 8-15), where the cycles gradually increase in amplitude. The issue is that this behaviour does not correspond to the actual deformation demand of diagonals, which should not cause any degradation of force (the diagram should follow the path given by the application of the monotonic load in tension). This implies that the *pinching4* material representing the cyclic behaviour needs to be re-calibrated by applying the effective displacement history that results from these analyses.

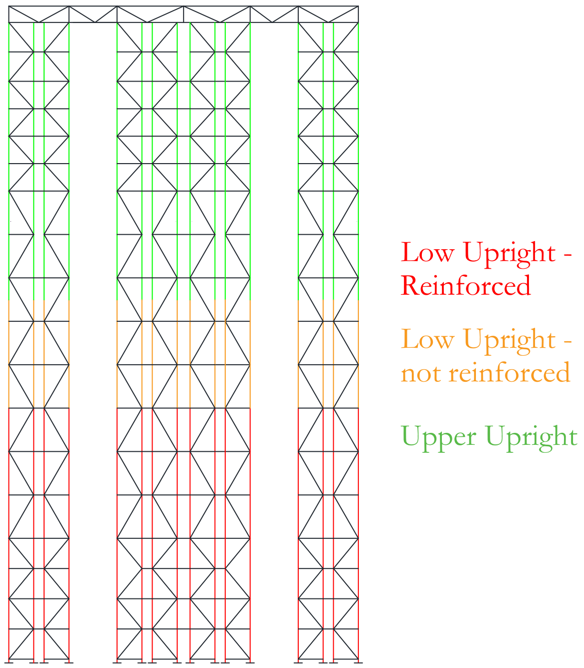


Figure 8-29: Uprights distribution in the 'hinged-spacer' structure.

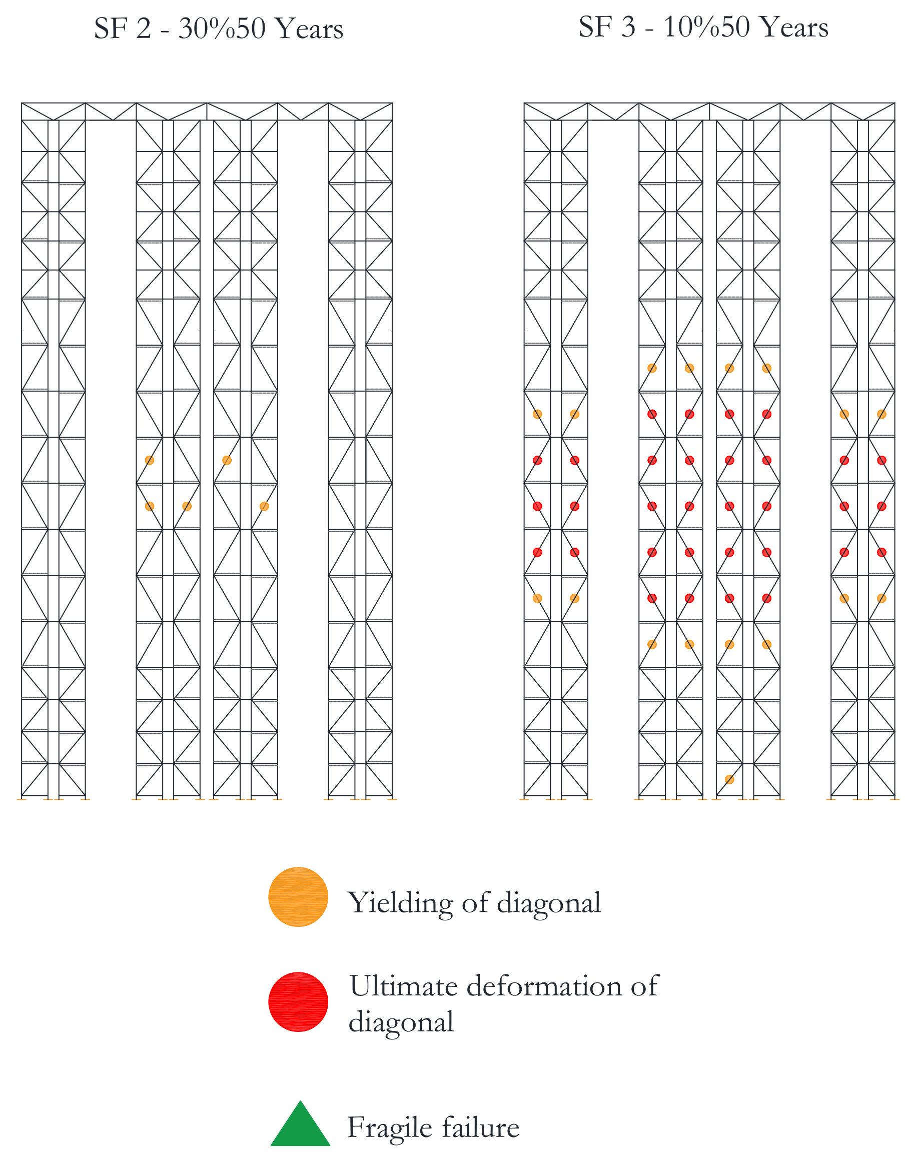
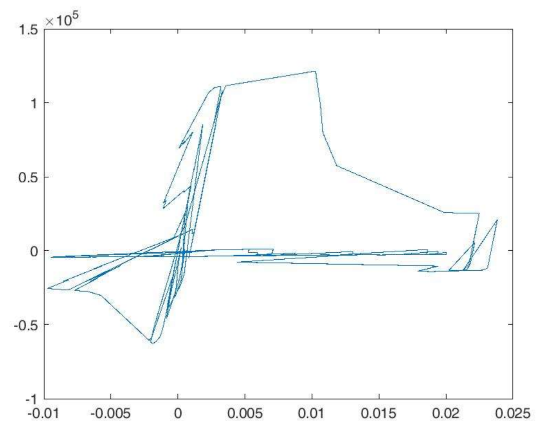
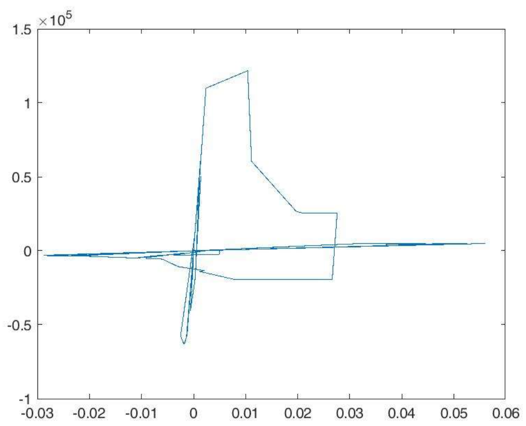
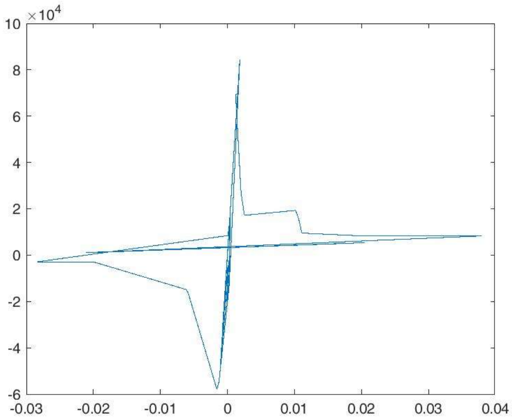
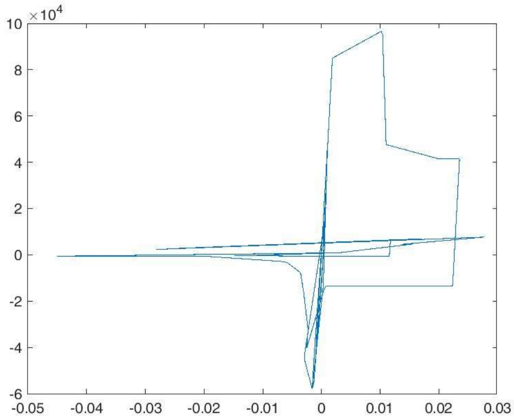
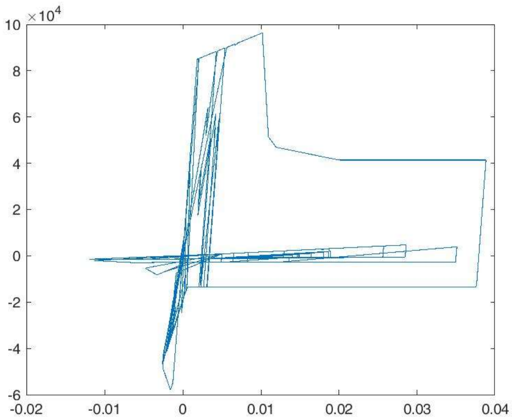
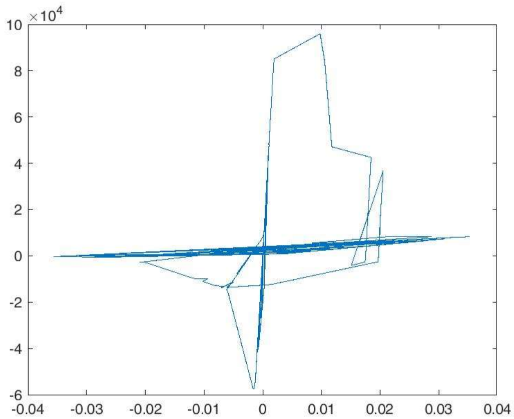
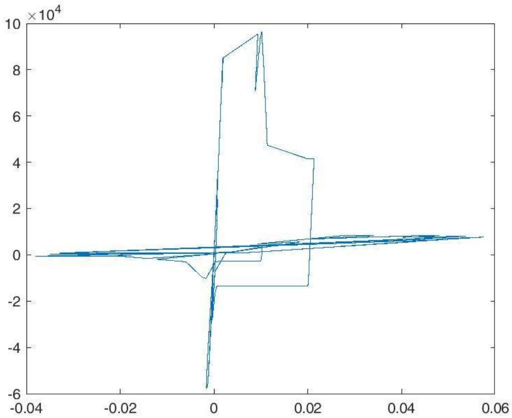
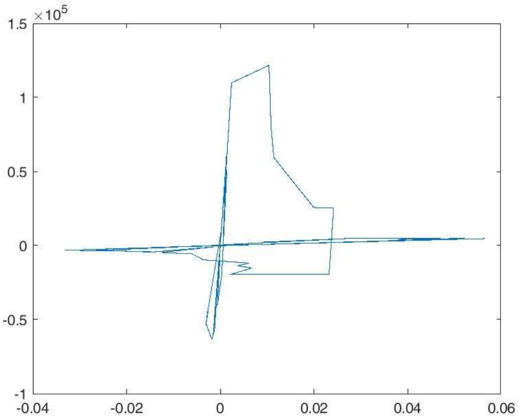


Figure 8-30: Graphic representation of ductile and fragile failures for SF2, SF3 (second project).

Table 8-13: Representative load [N] vs displacement [m] diagrams of diagonals belonging to levels 6 to 11 (SF3).

Diagonal level 6, shelve 1, accelerogram 4	Diagonal level 7, shelve 2, accelerogram 3
	

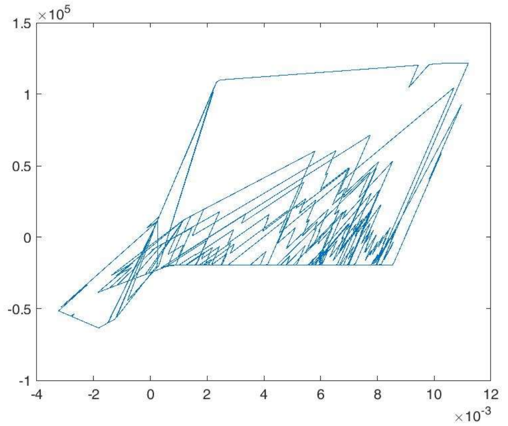
Diagonal level 8, shelve 3, accelerogram 12	Diagonal level 9, shelve 4, accelerogram 2
	
Diagonal level 9, shelve 5, accelerogram 10	**Diagonal level 11, shelve 6, accelerogram 5**
	
Diagonal level 8, shelve 7, accelerogram 3	**Diagonal level 7, shelve 8, accelerogram 3**
	

Based on the previous considerations, the pinching4 material of diagonals from level 5 to 12 has been re-defined to be more representative, considering the effective displacement demand. The NLTH analyses are carried out again for SF3. Table 8-14 shows some of the load vs displacement curves obtained for the diagonals from level 6 to 11, confirming the displacement demand. Till SF3, all the elements satisfy the requested safety level.

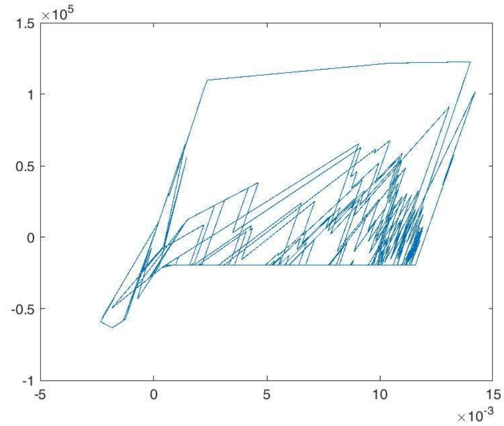
Figure 8-31 compares the IDA curves for the first structure, the “fixed spacer” one, and the second structure, the “hinged spacers” one. The curve of the “hinged spacers” structure confirms that this structure is less rigid than the first one, and for the same level of base shear, the displacement demand is higher, confirming the higher request of ductility to the diagonals. This curve is represented till SF3, corresponding to a probability of exceedance equal to 10% (that is beyond the design one). Further analyses should be done to investigate the structural behaviour of this system for higher levels of acceleration (dotted line).

Table 8-14: Load [N] vs displacement [m] diagrams of diagonals belonging to levels 6 to 11 (SF3) resulted from a more representative calibration of pinching4 material, based on the effective displacement demand.

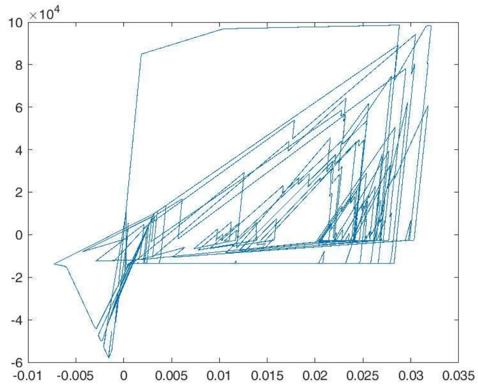
Diagonal level 6, shelve 1, accelerogram 3



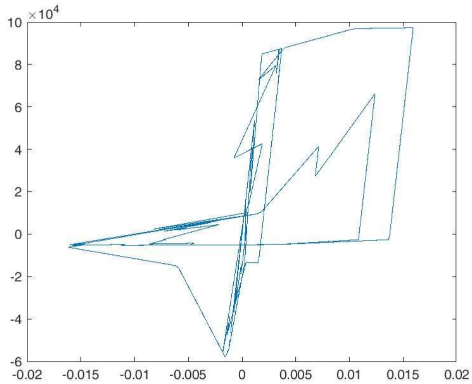
Diagonal level 7, shelve 2, accelerogram 3



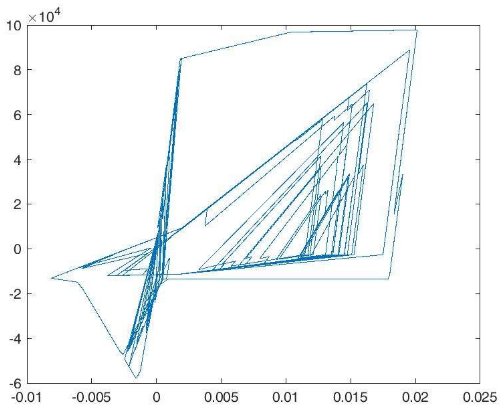
Diagonal level 8, shelve 3, accelerogram 15



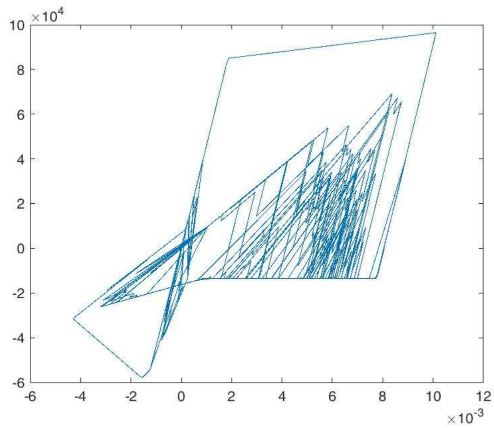
Diagonal level 9, shelve 4, accelerogram 2

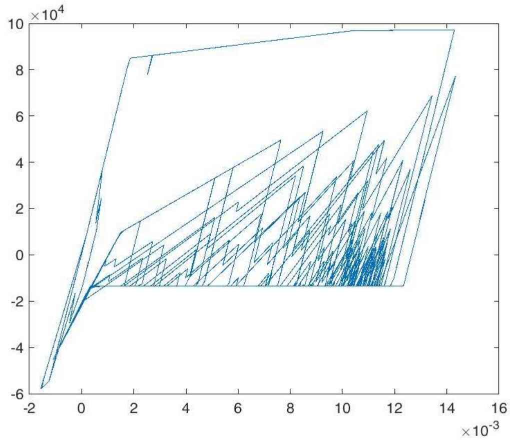
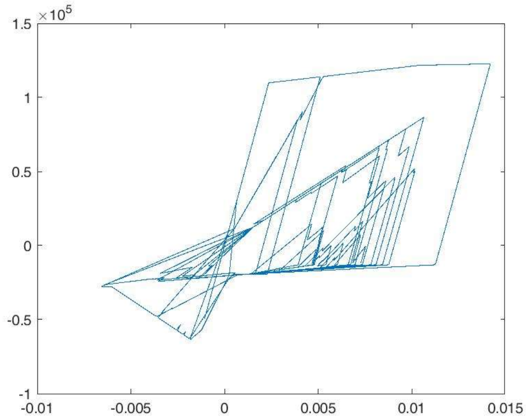


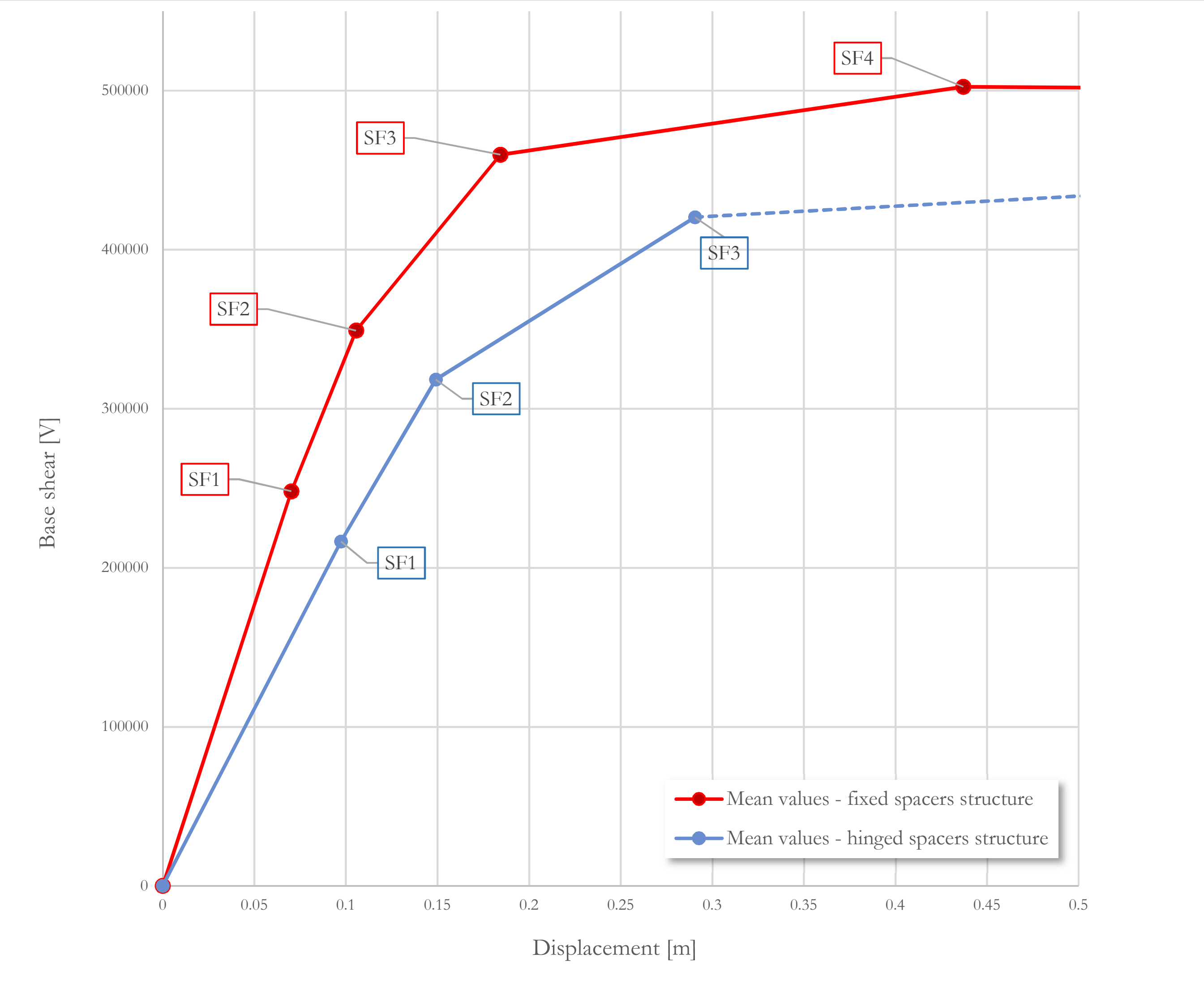
Diagonal level 9, shelve 5, accelerogram 10



Diagonal level 11, shelve 6, accelerogram 5



Diagonal level 8, shelve 7, accelerogram 3	Diagonal level 7, shelve 8, accelerogram 3
	



‘Hinged spacers’ structure	Probability of exceedance [%]	Base shear [N]	Displacement [m]
SF 1	60	216576	0.10
SF 2	30	318330	0.15
SF 3	10	420482	0.29

Figure 8-31: IDA curves for the ‘fixed spacers’ and the ‘hinged spacers’ structure: Base shear vs displacement (mean values of all the accelerograms).

If the behaviour of the two solutions is compared (the 'fixed spacer' and the 'hinged spacer' structure), in the first one, the design approach does not allow to satisfy the safety levels at SF3 (10% probability of exceedance in 50 years). The other solution (the 'hinged spacer' one), where the design approach is adjusted, and the diagonals' hysteretic behaviour is defined assuming the effective displacement demand, allows satisfying the safety levels at SF3. Further analyses should be carried out for higher seismic demand and to determine the actual global ductility of the structure according to this design approach.

8.5. Concluding remarks

Within this chapter, the possible optimization of the design of double depth warehouses is investigated. The study is performed from a global and a local point of view, exploring different structural schemes and at the same time trying to respect current and usual solutions in terms of technical features, cross-sections for the main elements, connections and structural optimization. In the framework of this thesis, the main attention is focused on CA direction, that looks very peculiar and atypical with respect to structural schemes adopted for DA direction.

The new proposal concerns the possibility to design the structure as dissipative. Given that the lower part of the structure is the most exploited one, in contrast with the upper one, in this approach the lower part only is included in the plastic mechanism, while the higher remains in the elastic field (Figure 8-1).

In general, the studies around the development of the new design strategy are organized in the following steps:

- A global optimization of the structure from the geometrical point of view is carried out to reduce possible eccentricities;
- From the global point of view, an optimization of the structural scheme is performed to identify the structural type that allows reaching the desired dissipative structural behaviour;
- From the local point of view, optimization at local level is required to guarantee a sufficient over-resistance of connections of the dissipating elements and non-dissipative elements with respect to the dissipative ones. Eurocodes prescriptions are adopted as starting direction to design the components.

The structural assessment of the designed structure is made through the execution of NLTH analyses, considering the cyclic behaviour of the dissipating elements.

A "no discount" policy has been adopted for the definition of the input design parameters, preferring to find (if possible) acceptable solutions in the most challenging conditions. The possibility of re-introducing "softer" assumptions is subsequently evaluated by assessing their applicability to ARSWs (these assumptions are validated only for traditional racks). Table 8-1 gathers all the input design parameters, highlighting that: no other reductions of the design spectrum (but the behaviour factor) are adopted, no reduction of the participating seismic mass is considered, and the maximum number of pallets for each load level is assumed.

At global level, different structural typologies are investigated to find those more suitable for the aimed structural behaviour. Two structural types are mainly involved: the truss one (where diagonal work both in compression and in tension), and the X bracings (with tension-only diagonals). From the analyses executed, the X bracing structural type is the one more suitable for the purposes of the design method, since it allows to gain a better optimization of the diagonal elements.

Anyway, the issue about the diagonal-to-upright connection remains, being very difficult for such profiles to design an over-resistant connection with respect to the diagonals' ultimate tensile resistance. The leading mechanism for the design of connection is plastic ovalization. Looking at equation (8-IV), the bearing resistance of a bolted connection with thin-walled cold-formed elements involved is directly proportional to the thickness of the element, the ultimate resistance of the steel, and the diameter of the hole made for the connection. The only way to increase the connection's bearing resistance 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 design force of the connection is increased, so the circle keeps turning. If allowed by the profile's geometry 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. Finding balance is not easy. Besides, also the configuration of the connection is not helpful: in fact, the bolts can be placed only along the vertical sides of the diagonal, without having the possibility to use the upper side. If the area collaborating for plastic ovalization 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 bigger than the former, and the use of ultimate stress f_u (that is used to calculate plastic ovalization resistance, while yield strength is used for tensile resistance) may be not sufficient to make up for this difference (Figure 8-4). This local issue can be solved only by performing a local optimization of the element since it is not possible to work only on connection.

Based on the issues about the design of an over-resistant connection, the optimization at the local level consists of reducing the resistance of dissipating element by weakening the cross-section through holes placed very close to the connection (to affect the less the slenderness of the profile). Numerical analyses are performed to find the holes' better configuration, both for transversal and longitudinal direction. Besides aiming to reach the requested level of ductility, the optimisation aims are also the following: on the one hand, the holes' pattern should allow having the closest possible buckling resistance of the intact element to control and limit the increment of the slenderness. On the other hand, the element's behaviour in compression should be such as deformations are not concentrated in the reduced-section part. A global buckling mode should be obtained, as for the intact element. The behaviour in compression is a relevant aspect to consider, since it may affect the element's cyclic behaviour, causing, if poor, a rapid decrease of stiffness and strength when re-loading in tension. All the other non-dissipative elements are designed over-resistant with respect to the dissipative ones by applying the capacity design rules indicated within Eurocode8 (prEN 1998-1-2:2019).

Two configurations are designed and numerically assessed through NLTH analyses (preformed through Opensees® FEM software (Mazzoni et al. 2017)): the one with the fixed spacers, and the one with the hinged spacers (where the spacers are the elements that connect the consecutive adjacent shelves). The first configuration allows the successive shelves to behave as coupled, affecting the structure's stiffness and the distribution of forces due to seismic action. All the non-dissipative elements are modelled as elastic, while the *pinching4* material is associated with diagonals. This material for the dissipative elements is calibrated on the cyclic behaviour obtained by applying a gradually increasing displacement history (Figure 8-15). In the 'fixed spacers' structure, looking at the IDA curves (representing base shear vs displacement at the top of the system, Figure 8-25), starting from SF2 (30%), some diagonals between 6 and 8 level reach yielding deformation, while at SF3 (10%, beyond of the design probability of exceedance), most of the diagonals reach ultimate deformation, especially between 6th and 9th levels. The uprights between the 5th and the 7th level fail due to axial compression and bending force. At SF4 (5%), most diagonals reach the ultimate deformation, and most uprights fail due to axial compression and bending force.

Focusing on SF3, it is evident that the diagonals between the 6th and the 11th level request a higher ductility level. This is confirmed by the relative inter-storey displacements of the structure (Figure 8-27). Regarding the uprights, the premature failure of these elements is concentrated in the levels where diagonal reach their ultimate failure and where there is an increment of relative displacement. Table 8-12 shows the mean D/C ratios of uprights at SF3 for axial compression (C), bending moment (B) and their combination (C+B) (please note that maximum axial compression and bending may not occur at the same time). Looking at those D/C ratios, it can be noticed that in the levels where the relative displacement demand increases, a relevant contribution to D/C ratios is given by bending. Based on this consideration, and assuming that a limited value of section modulus characterizes uprights, the approach used for the design of these elements (where only the axial tension/compression component due to seismic action $N_{Ed,E}$ is amplified by the Ω factor, see equation (8-II)) may be not appropriate for these structures. Consequently, it is precautionary to amplify also the value of the bending force component due to seismic action $M_{Ed,E}$.

This last approach is applied to design the uprights of the 'hinges spacers' structure, where the ductility of the diagonals has also been increased, too. From the execution of NLTH analyses, at SF3 (10%), one diagonal at the 1st level of the central shelve and the diagonal of the central shelve at the 5th level reach yielding deformation. Between the 6th and 10th level, diagonals reach ultimate deformation (Figure 8-30). This is happening because, being the structure more flexible, the ductility request is increased with respect to the one of the 'fixed spacers' structure. Besides, looking at the load-deformation diagrams of the dissipative elements, in most cases it happens that the diagonal, after little amplitude cycles in the elastic field, has a big amplitude cycle in tension (Table 8-13). According to how *pinghing4* material is modelled, the load-deformation path follows the backbone curve in tension. The backbone curve represents the envelope of the cyclic behaviour of the diagonal due to a different displacement history (Figure 8-15), where the cycles gradually increase in amplitude. The issue is that this behaviour does not correspond to the actual deformation demand of diagonals, which should not cause any degradation of force (the diagram should follow the path given by the application of the monotonic load in tension). If *pinching4* material is re-calibrated on the effective displacement demand, at SF3, that corresponds to a probability of exceedance of 10%, all the elements satisfy the safety levels.

If the behaviour of the two solutions is compared (the 'fixed spacer' structure and the 'hinged spacer' structure), in the first one, the design approach does not allow to satisfy the safety levels at SF3 (10% probability of exceedance in 50 years). The other solution (the 'hinged spacer' one), where the design approach is adjusted, and the effective displacement demand is taken into account for the definition of the diagonal's hysteretic behaviour, allows to satisfy the safety levels at SF3. Further analyses should be carried out for higher seismic demand and to determine the actual global ductility of the structure according to this design approach.

9. General conclusions and future developments

ARSWs are the latest in the storage solutions field. When the number of pallets is relevant and the daily handling operations are numerous, they offer the best options for goods flow management and space optimization. They constitute the direct upgrade of traditional pallet racks, and, to follow the fast-evolving market request, they acquired most of the structural features of racks without being supported by a specific regulatory framework. The lack of a specific regulatory framework brought with time to relevant catastrophes that highlight the lack of knowledge that concerns these structures. The absence of specific prescriptions for designing of ARSWs results, in most cases, in the adoption of the same guidelines defined for traditional steel racks (UNI EN15512 (2009) and UNI EN16681 (2016)) and to the adoption of the same structural choices and technical solutions (starting, from the global point of view, with structural schemes, and, from the local point of view, with the material, cross-sections for the structural elements and structural details as connections).
In this framework, the aim of this PhD thesis is to develop a new approach for the design of ARSWs. In particular, based on an accurate evaluation of safety levels and the design strategies now adopted in current practice, the main objective is to propose a design approach in seismic conditions, focusing on the Double-Depth structural typology. This approach is defined assuming a dissipative behaviour for these structures (depending on the structural schemes), and evaluating different and possible yielding patterns, as an alternative to the global collapse mechanism, where the whole system is involved. In any case, the optimization of cost-benefit ratio is always taken into consideration as one of the design goals (cost-benefit ratio consists of costs connected to a wider variety of construction details that may be implied by capacity design; benefits related to dissipative behaviour that allow obtaining controlled yielding pattern and lighter structures).

In the first step, the structural assessment of 5 double depth ARSWs is carried out with the final aims of comprehending the current design strategies, structural choices, technological features and structural behaviour. The case studies are designed by five big European companies that nowadays design and produce ARSWs. Common input parameters are fixed to have comparable solutions, while others, as the structural type, the profiles for the main structural elements (cross-sections, steel grade), the technological solutions, are set free to be chosen. From the analysis of the case studies, it seems that the main path that guides all the structural choices is optimization, which aims to balance the structural needs with limiting the costs connected to the amount of steel, the additional processes at workshops, and the variety of cross-sections. In fact, similar technological solutions are used in terms of cross-section shapes for the same structural element and connection.
Dealing with the analysis of the design parameters to be freely adopted, there are many having a relevant impact on the definition of the design response spectrum and finally on the seismic design force's value. In fact, the seismic design force can significantly vary, also for the same structural type. As an example, the following crucial parameters are individuated:

- As regards the definition of the design response spectrum, besides using the behaviour factor, there is the K_d factor (EN16681 (2016), equation (7-III)), that takes into account the capability of pallets to dissipate energy through their movement on the pallet beams when friction is overcome. In particular, a decrease of seismic acceleration up to 46% corresponds to the assumption of K_d equal to 0.8 and q-factor equal to 1.5, while a decrease up to 60% can be reached by adopting a q-factor equal to 2.0;

- Dealing with the definition of the seismic mass (§7.1, equation (7-I)), if both Ψ_2 and RF factors are assumed, in line with EN16681 (2016), a reduction of the seismic mass up to 20% is obtained along CA direction and up to 46% along DA direction. This assumption directly affects the size of the total base shear due to seismic action, which is reduced by this reduction of mass, but is also increased due to the increment of the seismic acceleration (since, assuming the same stiffness, the mass decreases, and so the fundamental natural period of the structure decreases, too). Consequently, it is not possible to say *a priori* if these assumptions finally determine an increment or a reduction of the design seismic base share.
- Along CA direction, it is possible to consider the reduction of the lateral shear stiffness of the frames due to the eccentricity of the diagonal-to-upright connections with respect to the centroid of the uprights. The size of the reduction of this lateral stiffness can be determined by the execution of experimental shear tests on the shelves constituting the structure. This assumption basically can make the natural period of the structure (along CA direction) increase also up to 30-40%, being the structure more flexible, and determining a reduction of the seismic acceleration.

Table 8-15 gathers the effects of these parameters on the value of the design base shear (the definition of the seismic participant mass is not considered, since it is not possible to say *a priori* if these assumptions finally determine an increment or a reduction of the design seismic base share). It can be noticed that, in some cases, relatively high reductions are reached (up to 60%), and could get worsen in case that also the reduction of the participant mass would be not conservative.

In general, it is necessary to evaluate if these assumptions that are justified for traditional racks are also suitable for ARSWs, or if they lead to an unsafe and not conservative design. Besides, it seems that a universally accepted and shared guideline discussing the adoption of all these parameters is missing, and this lack implies a significant variability of the value of the design seismic force, although the structural type is the same.

Table 8-15: Influence of the design assumptions in the reduction of the seismic design base shear.

Case Study	Direction	Seismic acceleration			Mass		Total Seismic base shear reduction [%]
		q	Kd	Lateral stiffness	Ψ_2	RF	
1	CA	1,5	0,8	YES	1,00	1,00	**52**
	DA	1,5	0,8	NO reduction	1,00	0,80	**46**
2	CA	1,5	0,8	YES	1,00	1,00	**52**
	DA	2,0	0,8	NO reduction	1,00	0,80	**60**
3	CA	1,5	0,8	NO reduction	0,80	1,00	**46**
	DA	2,0	0,8	NO reduction	0,80	0,80	**60**
4	CA	1,5	0,8	NO reduction	0,80	1,00	**46**
	DA	2,0	0,8	NO reduction	0,80	0,80	**60**
5	CA	1,5	1,0	NO reduction	0,80	1,00	**33**

	DA	1,5	1,0	NO reduction	0,80	0,80	**33**

Dealing with the vulnerability assessment, the results of the safety check of the main components are organized in the so called "hierarchy of criticalities", where the weakest parts of the structure and the chain of mechanisms are individuated by putting in order (form the highest to the lowest) the demand-capacity ratios. What appears clear is that failure of connections is the one happening first. The components that are characterized by the highest D/C ratios are diagonal connections and uprights base connections. The leading mechanism for diagonal connections is plastic ovalization, while the leading one for base connections is failure due to tensile and shear force on anchors (where the mechanism associated to tensile force is the concrete-cone one, and the mechanism connected to shear force is failure of anchor). Failure of base connections occurs in all the case studies where post-installed anchors are used.
This is one of the possible consequences of not applying any hierarchy rule in the structure's design. This design strategy implies that, from the global point of view, the structure has very limited post-elastic sources: if plastic ovalization of diagonal-to-upright connection happens first, these connections will become loose (with a poor dissipative behaviour associated), the lateral deflections of the structure will increase, second order effects may become relevant and cause failure of uprights due to stability issues. If the crisis of an upright base connection is the first to happen, this could trigger a series of chain collapses leading to the collapse of the whole structure (Figure 3-1). In conclusion, the analysis of the case studies highlights that the current design approach can be applied if the structure is designed to remain in the elastic field. In any case, it is probable that, if crisis in connections is the first that occurs, the whole structure could be involved in the mechanism and be irreparably damaged. If the current design strategies are applied and a dissipative behaviour is expected (a behaviour factor major than 1.5 is adopted), post-elastic sources appear to be very limited, especially if the indications from EN16681 (2016) are applied. In fact, these guidelines suggest the no need of applying hierarchy rules for the design of the structure for low-dissipative design (behaviour factor between 1.5 and 2).

The outcomes of the vulnerability assessment are used as a starting point for the optimization of the seismic design approach for double depth warehouses. The new proposal concerns the possibility to design the structure as dissipative. With this purpose, the most suitable structural type is individuated by performing a design optimization from a global and a local perspective. The selected scheme is the X-shaped one, where diagonals are the dissipative elements, and all the others are designed to be over-resistant. Given that the lower part of the structure is the most exploited one, in contrast with the upper one, in this approach the lower part only is included in the plastic mechanism, while the higher remains in the elastic field (Figure 8-1). The design of the lower part is carried out by applying Eurocode 8 prescriptions for the medium ductility class (prEN 1998:2019). All the structural choices are made always thinking about the necessity to make this proposal applicable to the market. This implies that the same technical features and cross-section shapes are adopted.
The main steps for the development of the proposed design approach are the followings:

- A global optimization of the structure from the geometrical point of view is carried out to reduce possible eccentricities;
- From the global point of view, an optimization of the structural scheme is performed, in order to identify the structural type that allows to reach the desired dissipative structural behaviour;

- From the local point of view, adopting Eurocodes prescriptions as starting direction for the design of components, optimization at local level is required and studied, in order to guarantee a sufficient over-resistance of connections of the dissipating elements and non-dissipative elements with respect to the dissipative ones.

The structural assessment of structure is made through the execution of NLTH analyses, taking into consideration the cyclic behaviour of the dissipating elements.

A "no discount" policy has been adopted for the definition of the input design parameters, preferring to find (if possible) acceptable solutions in the most challenging conditions. Only after, the possibility to re-introduce them is discussed by evaluating their applicability to ARSWs (these assumptions are in fact validated only for traditional racks). Table 8-16 gathers all the input design parameters, highlighting that no other reductions of the design spectrum but the behaviour factor are adopted, no reduction of the participating seismic mass is considered, and the maximum number of pallets for each load level is assumed.

Table 8-16: Comparison of starting deign hypothesis for case studies and for the design optimization.

	Design optimization: "no discount" policy	Case studies
Number of pallets for each couple of beams	3	3 or 2
Mass definition	$G_1 + G_2 + Q_1$ (no reduction of pallet mass)	$G_1 + G_2 + \Psi_2 \cdot Q_1$ ($\Psi_2 = 0.8$) (reduction of pallet mass up to 20%)
Definition of design response spectrum	Q factor = 2.0	Q factor = 2.0 or 1.5 (depending on the structural typology)
	No other reduction considered.	K_d =0.8 reduction factor considered.
Reduction of stiffness along CA direction	Not taken into consideration.	Taken into consideration when available from experimental tests. It implies higher fundamental periods.

From the global point of view, two structural types are the most promising: the truss one (where diagonal work both in compression and in tension), and the X-bracings one (with tension-only diagonals). Between these, the X-bracing structural type is the one more suitable for the purposes of the design method, since it allows to gain a better optimization of the diagonal elements. Anyway, the issue of designing an over-resistant connection for the dissipative element always remains. The problem is that the leading mechanism for the design of connection is plastic ovalization, and is not easy to increase this resistance without increasing also the ultimate resistance of the element (that is the demand for the design of the connection). The optimization at local level is performed by investigating alternative strategies to overcome this problem, consisting in reducing the resistance of dissipative element. The cross-section of diagonals is indeed weakened through holes placed very closely to connection to affect the less the slenderness of the profile. Numerical analyses are performed in order to find the better configuration of the holes, both for transversal and longitudinal direction. Besides aiming to reach the requested level of local ductility, the optimization is also performed considering the behaviour of the profile in compression, which may affect its cyclic behaviour. If the behaviour in compression is poor, rapid decrease of stiffness and strength when re-loading in tension may occur.

All the other non-dissipative elements are designed over-resistant with respect to the dissipative ones, by applying the capacity design rules indicated within Eurocode8 (prEN 1998-1-2:2019).
Two configurations are designed and numerically assessed through NLTH analyses (preformed through Opensees® FEM software (Mazzoni et al. 2017)): the one with the 'fixed spacers', and the one with the 'hinged spacers' (where the spacers are the elements that connect the consecutive adjacent shelves). The first configuration allows the consecutive shelves to behave as coupled, and so the stiffness of the system and the distribution of forces on uprights due to seismic action changes. All the non-dissipative elements are modelled as elastic, while to the dissipating ones *pinching4* material is associated, that is calibrated on the cyclic behaviour obtained by the applying a gradually increasing displacement history (Figure 8-15).
In the 'fixed spacers' structure (Figure 8-32), at SF3 (10% probability of exceedance, which is beyond the design one (20%)), most of the diagonals reach ultimate deformation, especially between the 6th and 9th levels. As regard upright elements, between the 5th and the 7th level, the elements fail due to axial compression and bending force. At SF4 (5%), most of the diagonal reach the ultimate deformation, and most of the uprights fail due to axial compression and bending force. Focusing on SF3, it is obvious that the diagonals between the 6th and the 11th level request a higher level of ductility, and this is confirmed by the relative inter-storey displacements of the structure (Figure 8-33). Concerning uprights, the premature failure of these elements is concentrated in the levels where diagonal reach their ultimate failure, and where there is an increment of relative displacement. Looking at their D/C ratios, it can be noticed that in the levels where the relative displacement demand increase, a relevant contribution to D/C ratios is given by bending. Based on this consideration, and assuming that uprights are characterized by a limited value of section modulus, the approach used for the design of these elements (see equation (8-II)) may be not be fully appropriate, being precautionary to amplify not only the axial force but also the value of the bending force.
Basically, the more appropriate design formulas for to design an over-resistant upright may be the followings:

$$\begin{aligned} N_{Ed} &= N_{Ed,G} + \Omega \cdot N_{Ed,E} \\ M_{Ed} &= M_{Ed,G} + \Omega \cdot M_{Ed,E} \\ V_{Ed} &= V_{Ed,G} + V_{Ed,E} \end{aligned} \tag{8-IX}$$

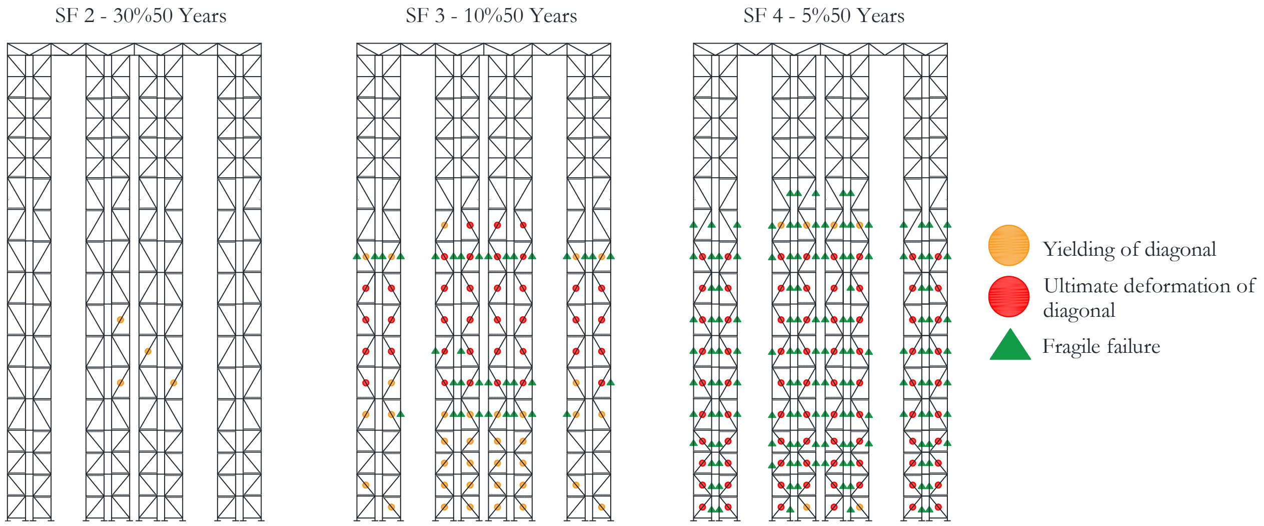


Figure 8-32: Graphic representation of ductile and fragile failures for SF2, SF3 and SF4 (structure with fixed spacer).

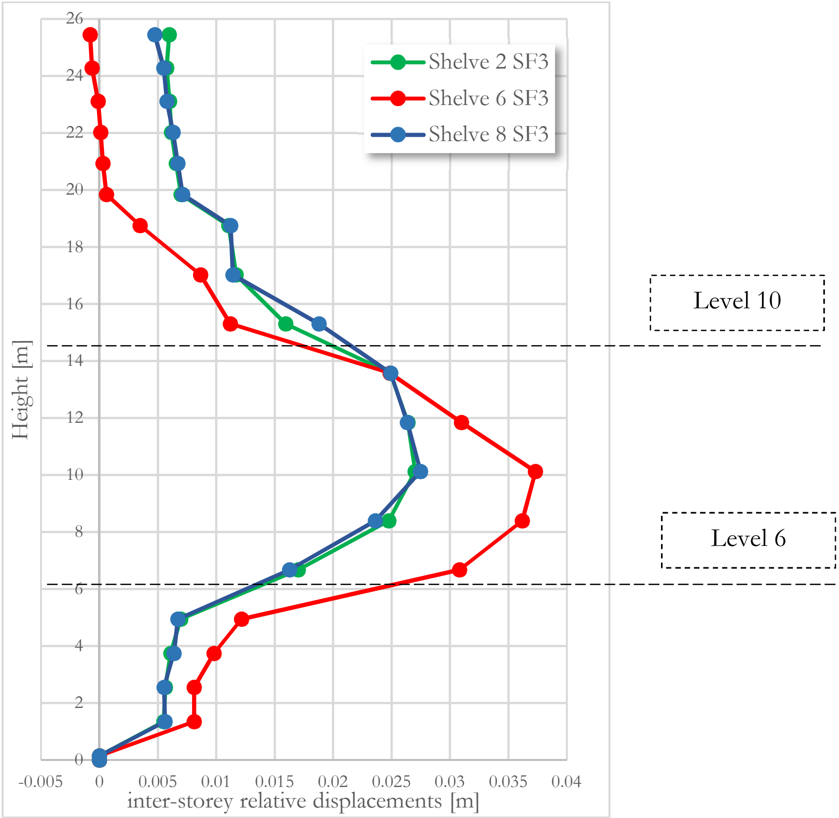


Figure 8-33: Inter-storey relative displacement relative to SF3.

This last approach is applied to design the uprights of the 'hinges spacers' structure, where the ductility of the diagonals has also been increased, too. From the execution of NLTH analyses, it results that at SF3 (10%), one diagonal at the 1st level of the central shelve and the diagonal of the central shelve at 5th level reach yielding deformation. Between the 6th and 10th level, diagonals reach ultimate deformation (Figure 8-30). This is happening because, being the structure more flexible, the ductility request is increased with respect to the one of the 'fixed spacers' structure. Besides, looking at the load-deformation diagrams of the dissipative elements, in most cases it happens that the diagonal, after little amplitude cycles in the elastic filed, has a big amplitude cycle in tension (Table 8-13). According to how *pinghing4* material is modelled, the load-deformation path follows the backbone curve in tension. The backbone curve represents the envelope of the cyclic behaviour of the diagonal due to a different displacement history (Figure 8-15), where the cycles gradually increase in amplitude. The issue is that this behaviour does not correspond to the actual deformation demand of diagonals, which should not cause any degradation of force (the diagram should follow the path given by the application of the monotonic load in tension). If *pinching4* material is re-calibrated on the effective displacement demand, at SF3, that corresponds to a probability of exceedance of 10%) all the elements satisfy the safety levels.

If the behaviour of the two solutions is compared (the 'fixed spacer' structure and the 'hinged spacer' structure), in the first one, the design approach does not allow to satisfy the safety levels at SF3 (10% probability of exceedance in 50 years). The other solution (the 'hinged spacer' one), where the design approach is adjusted and the effective displacement demand is considered for the definition of the hysteretic behaviour of the diagonal, allows to satisfy the safety levels at SF3, that is beyond the design probability of exceedance (20%).

9.1. Future developments

All the previous considerations and the results obtained need to be experimentally validated, starting from the cyclic behaviour of the diagonals, that, although calibrated also in relation to previous similar researches, is numerically determined. This solution will be actually experimentally tested and validated within the STTELWAR Research Project, and this will allow to adjust and re-calibrate the numerical model in order to obtain the final definition of the design approach.

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