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Estimation of the behavior factor of steel storage pallet racks

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Abstract. *European Racking Federation (ERF) is currently developing normative documents for the seismic design of pallet racks due to lack of sufficient design rules and bibliography. The special geometry of these thin-walled members of high slenderness and their non linear behaviour require specific rules for a successful and accurate modeling. The design of these non-traditional steel structures is even more complicated in seismic zones, where they must be able to withstand horizontal and vertical dynamic forces. Furthermore, the analysis must represent the additional limit state of the fall of pallets, which would lead to the subsequent damage to goods, people and to the structure itself. Bracing systems are widely used to enhance the stiffness of the structures under seismic loads. Braced systems exhibit, however, unwanted phenomena, such as torsional dynamic modes. On the other hand, unbraced systems have translational modes and uncoupled behaviour, but they are very flexible.*

In this paper, the earthquake performance of such structures is investigated using pushover analyses. The work presented, is a part of a research project which is carried out within the framework of the European Research Program SEISRACKS 2. Detailed analyses of components using finite elements software are performed in order to clarify their failure and post-failure behaviour. Finally, a comparison of the available q factor and the used value during the design takes place to verify the efficiency of the design procedure

1 INTRODUCTION

The need for a multiple, easier and efficient storage of goods is growing as the logistics are continuously developed. As a consequence a thorough design based on safety and economy is necessary. These special steel structures, otherwise called "storage pallet racks", do not follow the international norms applying for buildings since they are not conventional steel structures. For this reason nowadays multiple attempts are made for the publication of an independent and complete normative document.
Due to the absence of further research and experiments, the seismic performance of racking systems is not yet well known and therefore their members are generally over-calculated. The behaviour factors usually range from 1.5 to 2 although greater values are also allowed [1]. However, these values are not frequently used.
In this report, the results of nine different real case studies are presented, provided by 4 different industrial partners (I.P). Among them there are 6 unbraced and 3 braced racks, which are tested both on down and cross aisle direction.

2 ANALYSIS PROCEDURE

Pushover analysis involves the incremental application of horizontal loads (in a prescribed pattern) on a computer model of the structure. The structure is thus "pushed" until it reaches a limit state or a collapse condition, while the total applied shear force and the respective lateral displacements are plotted at each step.
The horizontal loads coexist with vertical loads, in order to create a real loading situation. The analysis takes into consideration nonlinearities of both geometry and material.

2.1 Geometry

The geometry of the provided racks is shown in figure 1 and 2. The first one refers to a braced system which is actually used for high seismicity zones, while the second one to an unbraced system that can be found in low or medium seismicity zones. Both figures present the longitudinal direction, also called "down aisle" direction.

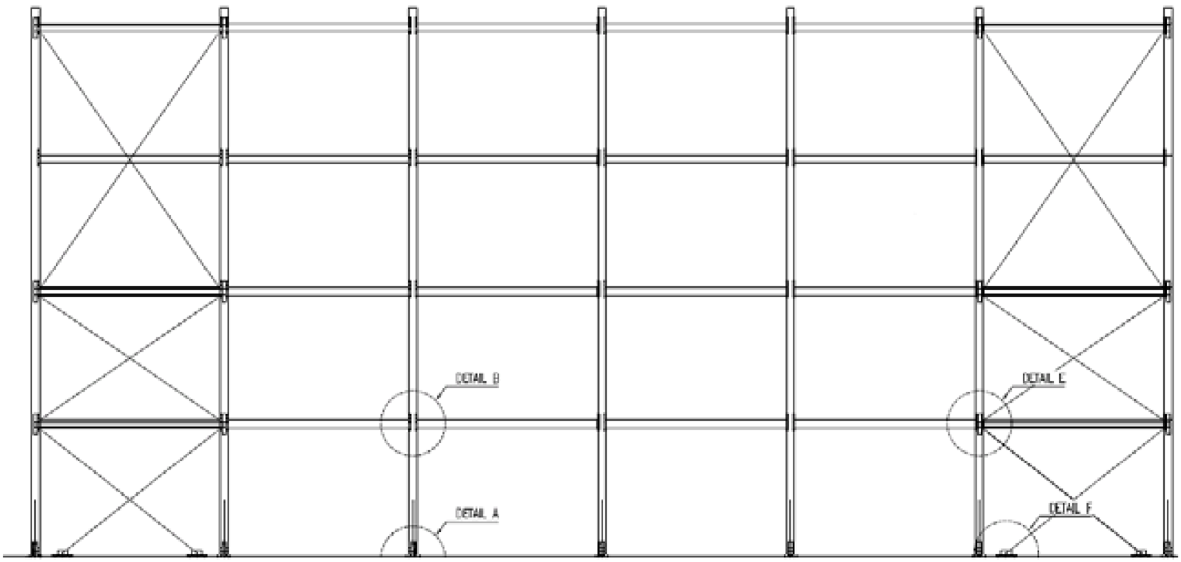


Figure 1- View of braced model at down aisle direction

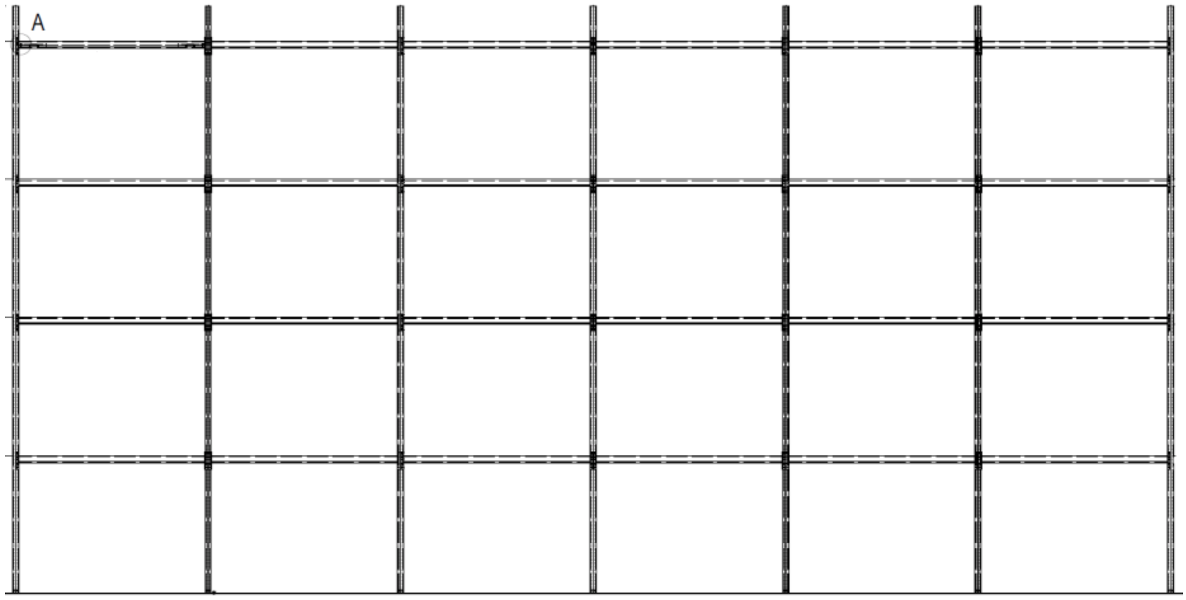


Figure 2- View of unbraced model at down aisle direction

Figure 3 shows the transversal, also called "cross aisle" direction of such a system with two different configurations. The major differences lie on the amount of the used diagonal members and the existence of symmetry or not of the assembly. The first case is used mainly for high seismicity zones. In this picture (on the left) the additional structure of the down aisle bracing system is also evident.

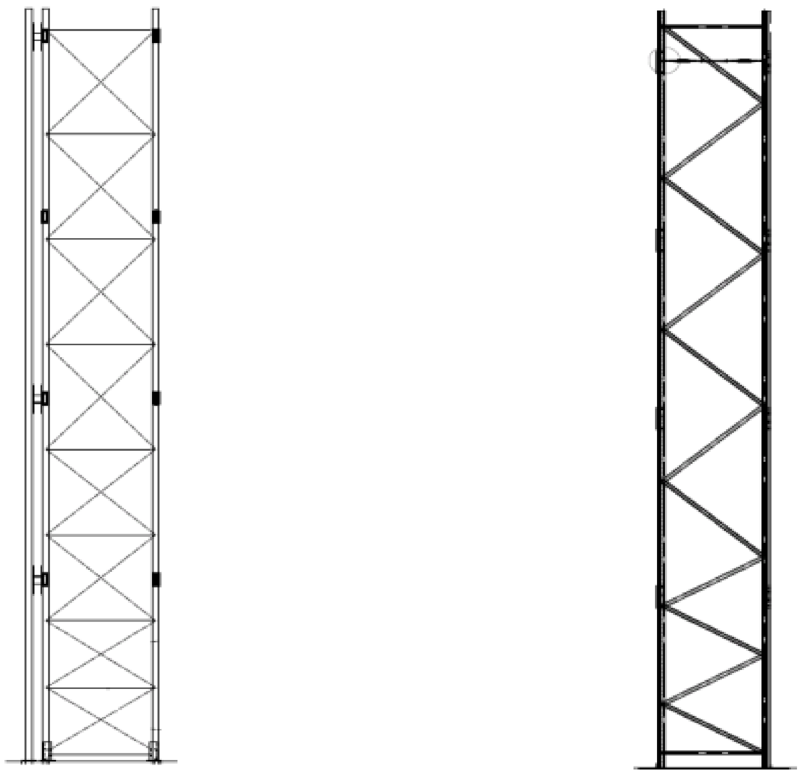


Figure 3- View of cross aisle direction

a) Typical bracing for high seismic zones

b) typical bracing for low seismic zone

2.2 Loads

The applied loads are vertical and horizontal. The vertical loads consist of the dead loads of the structure as well as of the pallets, assuming that the racks are fully loaded. The horizontal load is equivalent to the earthquake load and is applied incrementally. This load is assigned with a uniform distribution along the vertical axis in an attempt to overcome the significant difference between braced and unbraced racks in terms of modal shapes. More specifically, braced racks displayed torsional dynamic modes due to the eccentricity of the spine bracing, thus having lower participating mass ratios and consequently requiring the use of 30 modes in order to reach the required 90% of mass in a multimodal pushover analysis. On the other hand, unbraced racks exhibited a clear translational behaviour. The uniform load was therefore chosen in order to produce comparable results.

It should also be noted that P-delta effects have been taken into account, as the racking systems are generally flexible and the critical buckling factor α_{cr}, where:

$$\alpha_{\mathrm{cr}} = \mathrm{F}_{\mathrm{cr}}/\mathrm{F}_{\mathrm{Ed}} \qquad (1)$$

was not exceeding the critical value of 10 in most cases. Eurocode 3 suggests neglecting P-Delta phenomena when this factor is greater than 10 for elastic and 15 for plastic analysis [2]. In any case, it is always more accurate even though time consuming.

2.3 Inelastic structural properties

To perform pushover analyses the plastic characteristics of the components have to be defined. In this example, the members are made of thin walled steel sections that belong to class 4 and thus their plastic properties cannot be developed. However, the behaviour of these components is nonlinear and it is simulated as equivalent “plastic”. Therefore what will be referred to as "plastic hinge" and "plastic behaviour" from this point on, actually refers to this aforementioned nonlinear behaviour.

In general, the members/connections where plastic hinges are expected to occur are:

The beam-column connections (beam end connectors), the ground connections (base plates), the uprights and the diagonal members, as past experiments have already indicated [3].

As far as the diagonals are concerned, they can be found in the racking systems in three different versions: those of spine bracing, those of the upright frames and the horizontal ones.

2.4 Beam End Connector

The behaviour of the beam end connectors is crucial for the stability of the whole structure since it provides the frame action (moment resistance) longitudinally. These connectors are hooked, with or without safety pins, and their calculations are only experimental. Each manufacturer is obliged to provide test results in order to specify the rotational stiffness of the connection and its strength. In Figure 4, four different examples of this peculiar connection are displayed, in terms of moment rotation curves. These curves were used in the simulation of the different case studies. More specifically, they were input in the models as nonlinear link elements [4].

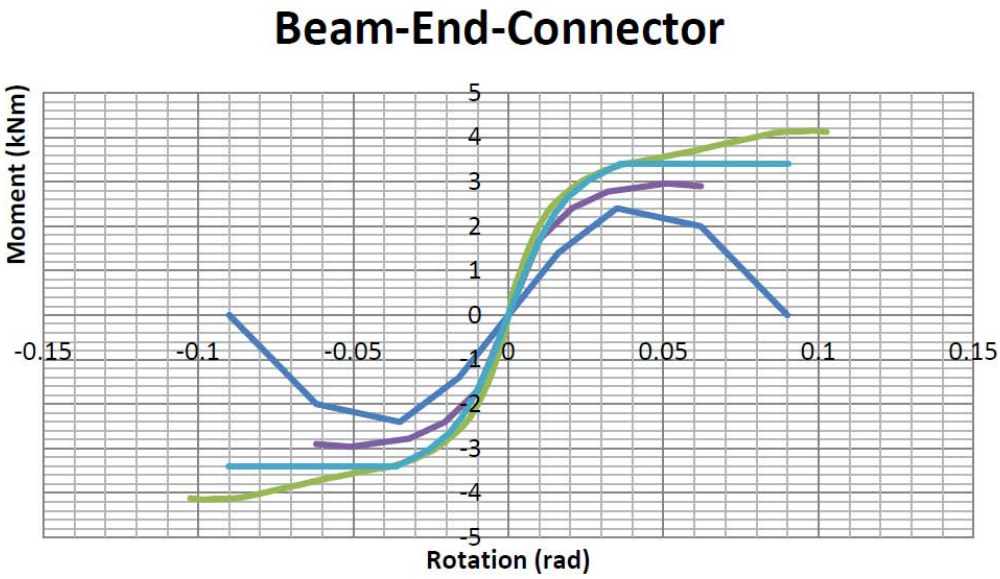


Figure 4- Moment- rotation curve for beam end connectors

2.5 Base- plate

The base-plate connections are also moment resistant connections, the behaviour of which must be defined experimentally. It is important to mention that the total behaviour of the connection is directly influenced by the normal force of the uprights.

Tests for different load levels took place; those with the axial force closest to our case study are presented in Figure 5, in terms of moment-rotation curves, as provided by the experiments of the industrial partners.

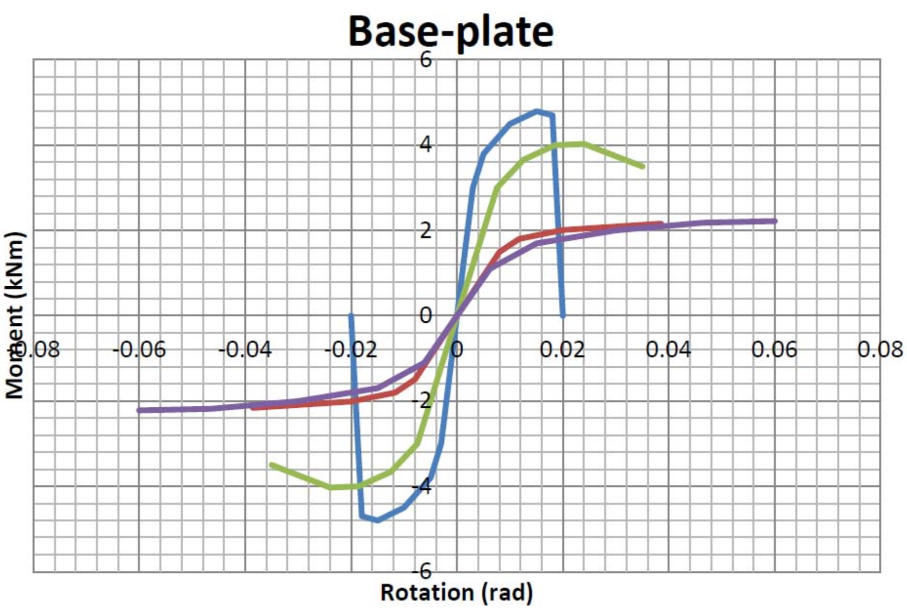


Figure 5- Moment-rotation curve of base plates

2.6 Uprights

The uprights consist of open sections of class 4, so their failure mode is likely to be due to local buckling or lateral- torsional buckling. Using the effective properties in the design stage, the failure is considered to be due to lateral torsional buckling. As is the case with any column, the uprights' behaviour can be described with an interaction curve of biaxial bending moments and axial force. The interaction relation taken into account can be found in Eurocode 3 regarding lateral- torsional buckling.

$$\frac{N_{Ed}}{\chi_{min}A_{eff}f_y/\gamma_M}+\frac{\kappa_y M_{y,Ed}}{\chi_{LT}W_{eff,y}f_y/\gamma_M}+\frac{\kappa_z M_{z,Ed}}{W_{eff,z}f_y/\gamma_M}<1 \quad (2)$$

Since, the failure occurs before the section enters the plastic region, the interaction relation is linear.

Another point of interest is to define properly the plastic hinges of uprights as well as the post buckling behaviour. Due to lack of experiments, numerical analyses took place in order to determine the moment-rotation curve of such a member. ABAQUS software was used to model a single (upright) member of two meters height, with both ends restrained against torsion and out of plane displacement. The members' base was also restrained in any translation, whereas the top was free to move not only axially but also in the uprights respective down aisle direction.

In the first step an axial force of 48kN was applied, as the one undertaken by an upright of a fully loaded rack, and then a horizontal load pushed the upright to failure.

Regarding the simulation assumptions, shell elements were used, the radius of the real section and the perforations was taken into account, the support was considered as a rolling fixity, and the advantage of symmetry was reckoned. Figure 6 shows the initial modelling and Figure 7 the deformed member at the end of analyses.

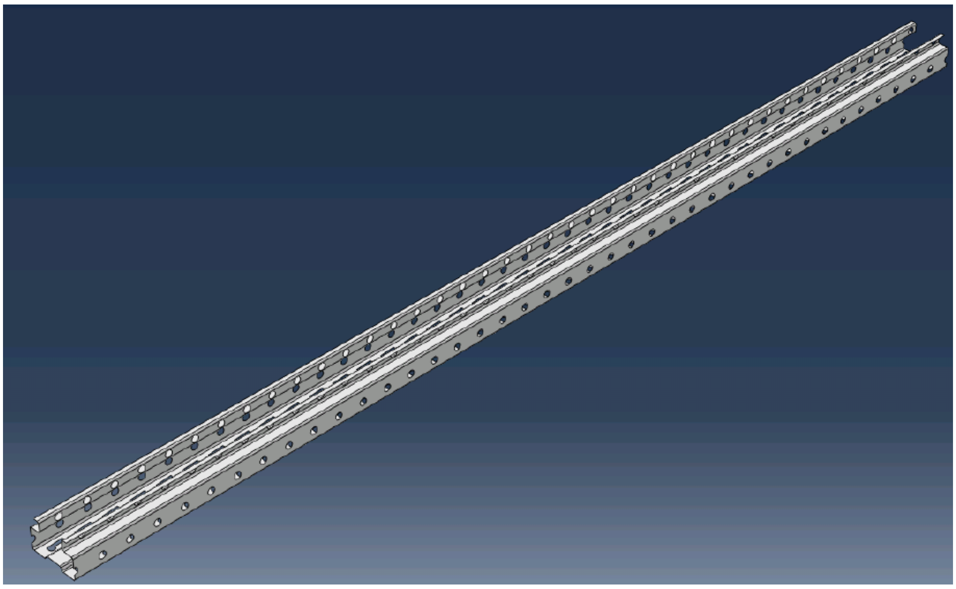


Figure 6- Simulated upright member

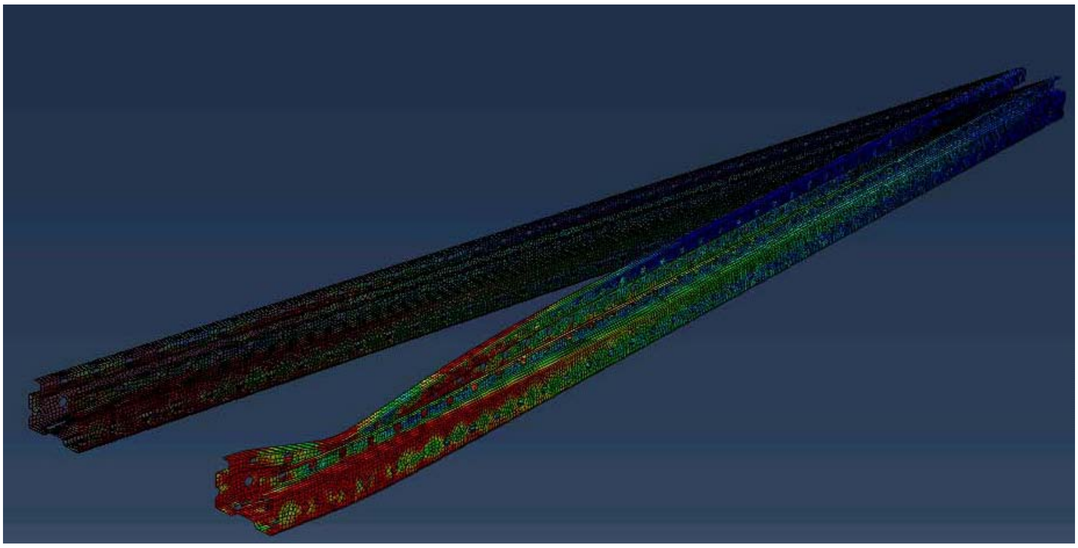


Figure 7- Deformed shape of upright at the end of analysis

The results from the previous analyses are presented to figure 8, as a diagram of moment-rotation.

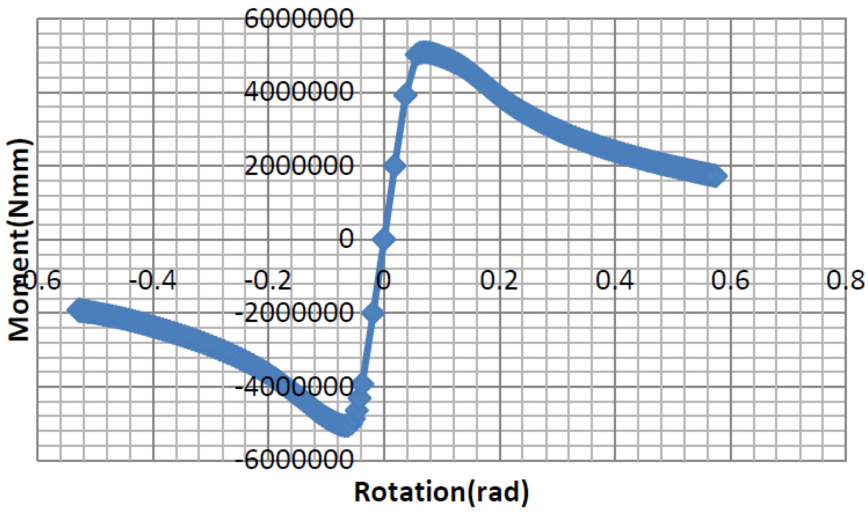


Figure 8- Moment rotation curve of the upright section

2.7 Diagonals

The diagonal members are also significant for the overall behaviour. However, their ultimate capacity is not clear enough, as there is sometimes an interaction between the buckling of the member, the capacity of the section in tension, the shear strength of the bolts and the bearing resistance of the connection. Hence, a further investigation took place numerically, in order to also define the equivalent plastic hinge for each member. The modelling procedure included the use of shell elements and imperfections, to define the theoretical buckling load of the member. Additionally, solid elements were used in a contact model, between holes and bolts, in an attempt to clarify the influence of the bearing resistance in the total behaviour. The results are summarized in Figure 9, and they refer to a channel section, with a single hole in the web.

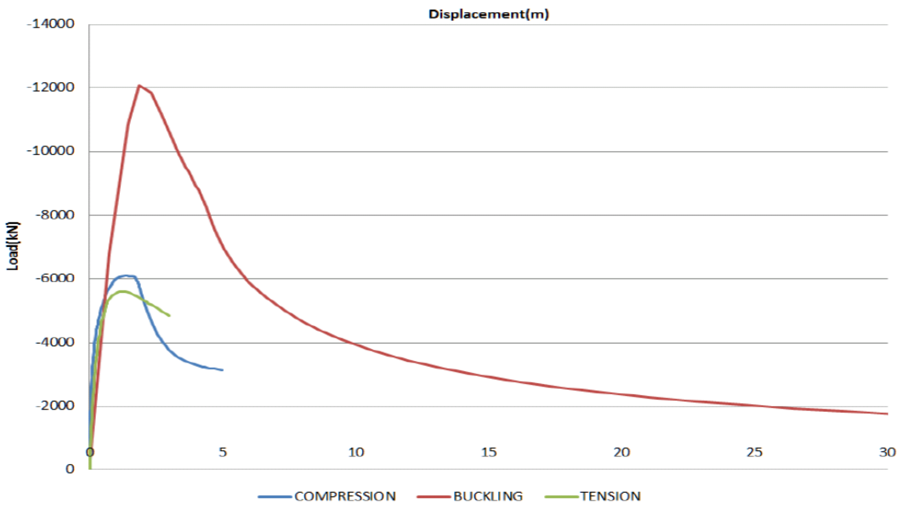


Figure 9- Force displacement curve for different failure modes of diagonals

The critical one is selected to be linearized and simulated as plastic hinge in the software. In this specific case the green and the blue line correspond to the bearing failure of the connection.

3 GLOBAL ANALYSES RESULTS

3.1 General

Nine different case studies of the industrial partners were simulated under equal horizontal loads at all levels as described in previous paragraph. The configurations were considered fully loaded with pallet loads, the inelastic properties of the system are simulated with plastic hinges or nonlinear link elements as described in previous paragraphs. The pushover curves were superimposed in order to compare quantitatively the different configurations. Two main groups of curves were drawn; those referring in down aisle direction and those in cross aisle direction. Figure 26 and 27 present the results for the nine cases on down and cross aisle direction respectively.

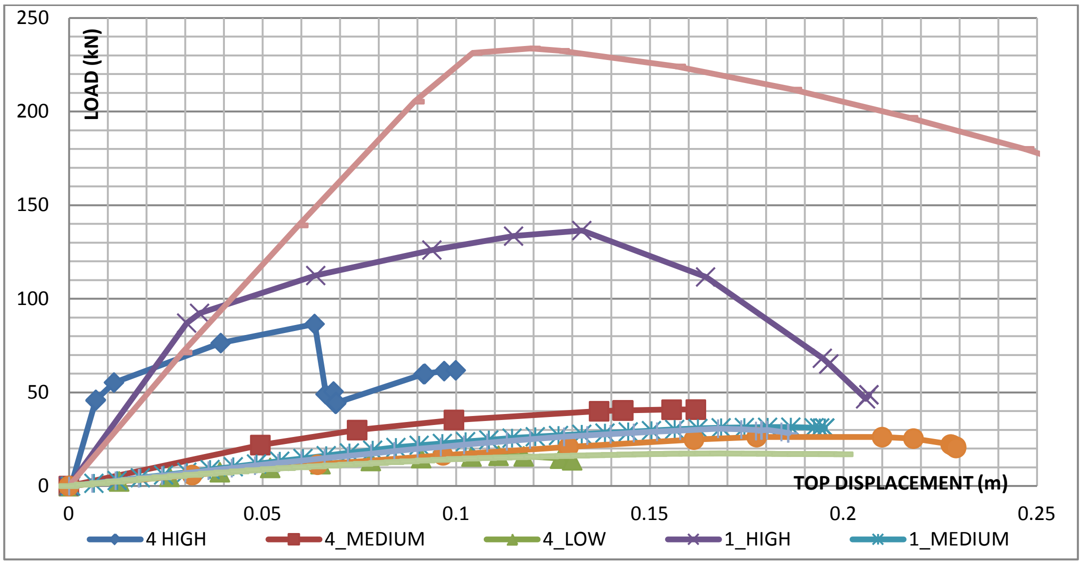


Figure 10- pushover curves for down aisle direction

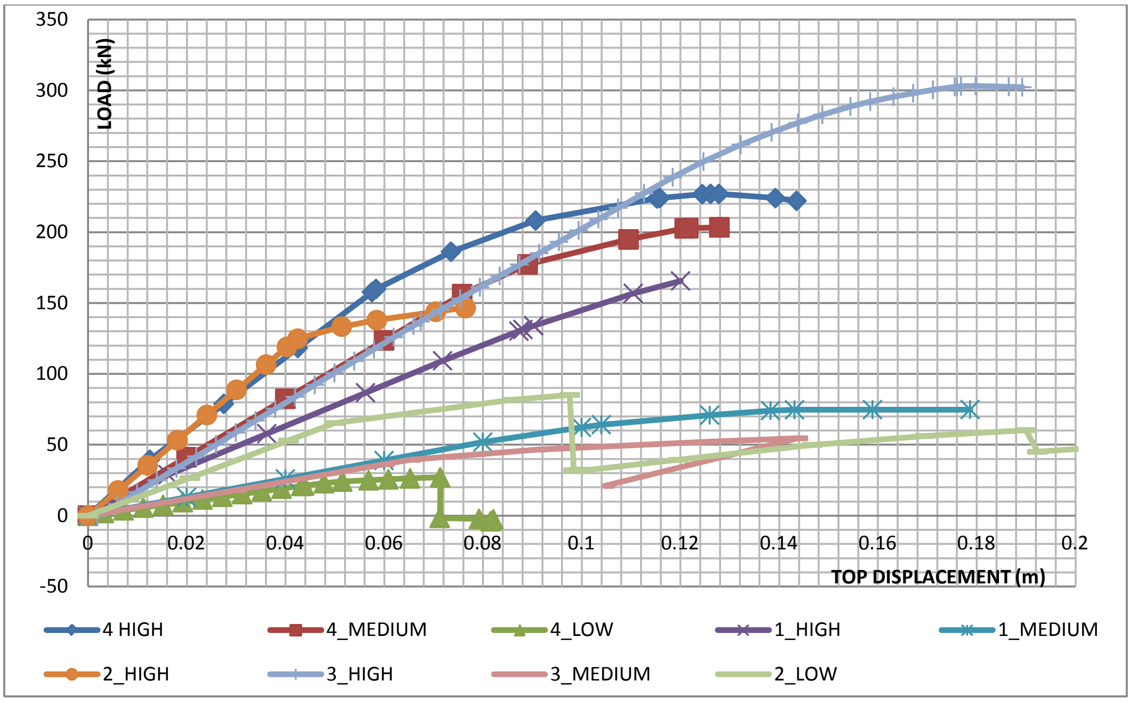


Figure 11- pushover curves for cross aisle direction

One can easily observe that the high seismic zone racks are quite stiffer than those designed for low seismic zones due to the fact that the former possess bracing systems on their back.

3.2 Behaviour factors

In order to estimate the demand imposed by the earthquake the performance points for each structure have to be determined. The relevant calculations are made by SAP2000, based on the procedure described in ATC-40 [5]. The parameters Ca and Cv that define the elastic spectrum are determined as following:

$$C_A = a_g Sn \tag{3}$$
$$C_V = 2.5 a_g Sn T_c \tag{4}$$

In order to calculate the q- factor the pushover curves have to be linearized. The q- factor is defined as the product of the overstrength Ω and the ductility ratio μ [6], as it is given in eq. (7). The definitions of the overstrength and the ductility are given by equations (5) and (6) respectively. The terms of these equations are defined in figure 12, where the linearization of the pushover curves is presented as well. In absence of other criteria, the “ultimate” state is defined as the state where the base shear reaches its maximum value. The linearized and the original pushover curves, the performance points and the calculated q- factors are illustrated in Figures 13 to 21 for all case studies. The pushover curve is illustrated with blue line, the linearized curve with green line. The performance point is indicated in red, the point of first significant yield in purple. It is interesting to see that in some systems the performance point is below first yielding, indicating that the system is over-dimensioned.

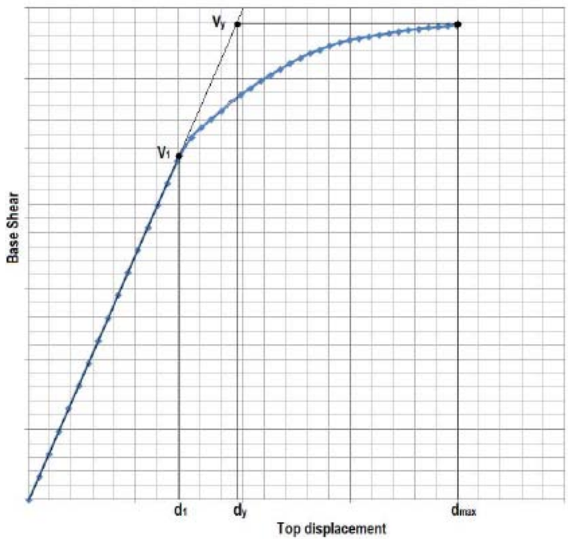


Figure 12- Linearization of a pushover curve

$$\Omega = \frac{V_y}{V_1} = \frac{d_y}{d_1} \tag{5}$$

$$q_0 = \mu = \frac{d_{\max}}{d_y} \tag{6}$$

$$q = q_0 \Omega \tag{7}$$

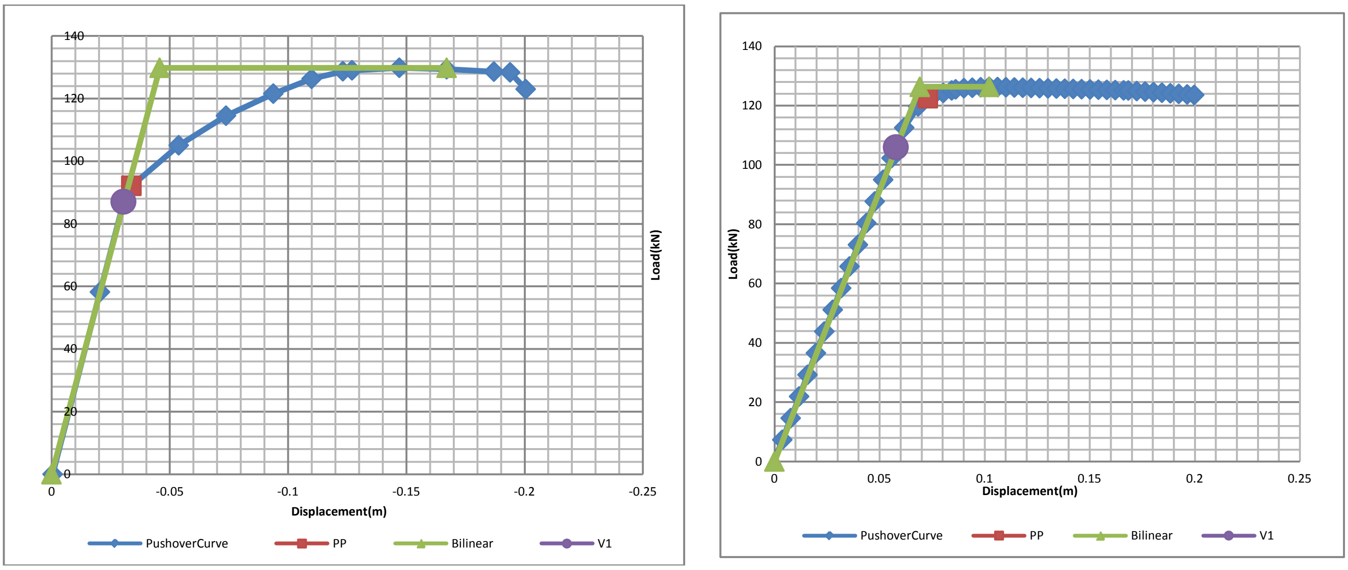


Figure 13 System A, high seismic zone

a) Down aisle b) Cross aisle

Tables 1-9 present the ductility, the overstrength and the q- factor of each system. The system name (A, B, C, or D) indicates the IP.

	μ	Ω	q
Down	3.65	1.50	5.47
Cross	1.47	1.2	1.76

Table 1- Ductility, overstrength and q- factor for system A-high seismic zone

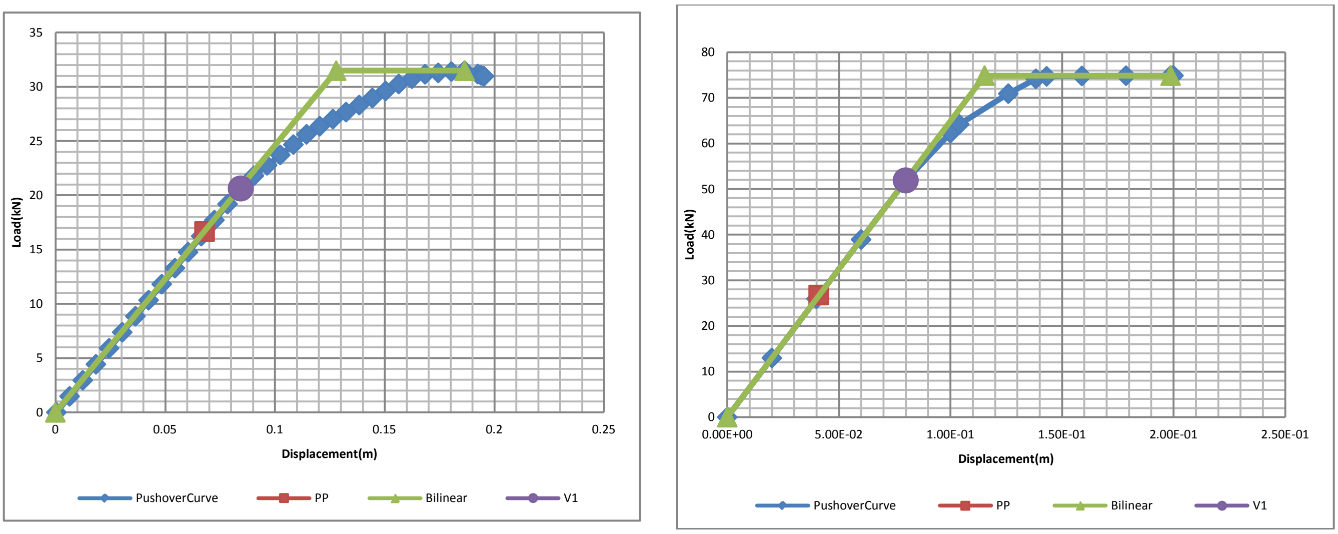


Figure 14- System A, Medium Seismic Zone

a) down aisle b) cross aisle

	μ	Ω	q
Down	1.45	1.52	2.22
Cross	1.72	1.44	2.48

Table 2- Ductility, Overstrength and q- factor for the case study A-Medium seismic zone

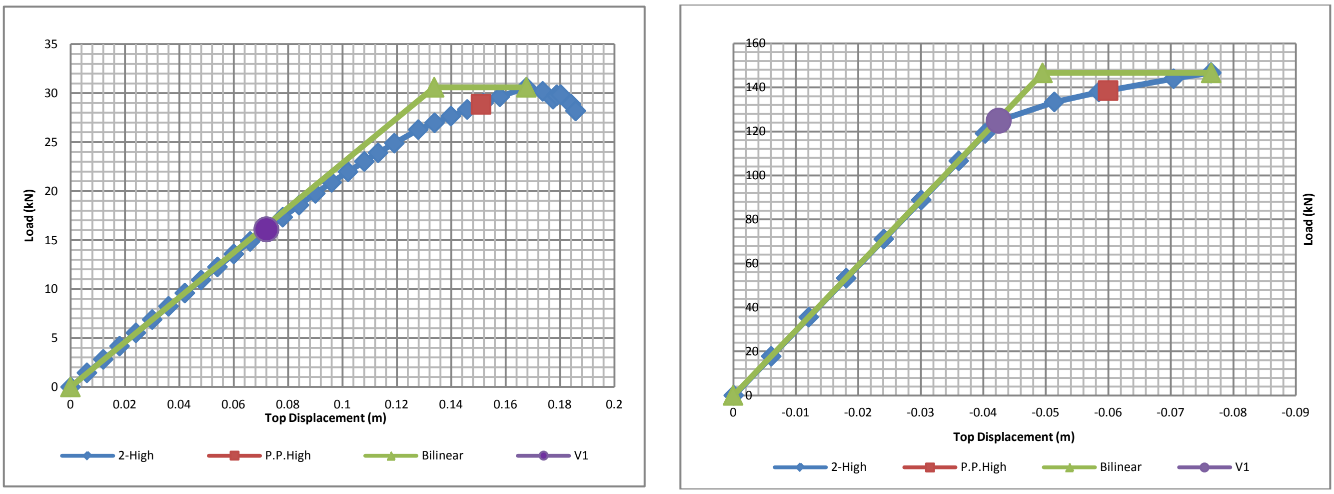


Figure 15- System B, High Seismic Zone

a) down aisle b) cross aisle

	μ	Ω	q
Down	1.25	2.06	2.58
Cross	1.54	1.17	1.81

Table 3- Ductility, Overstrength and q- factor for the case study B-high seismic zone

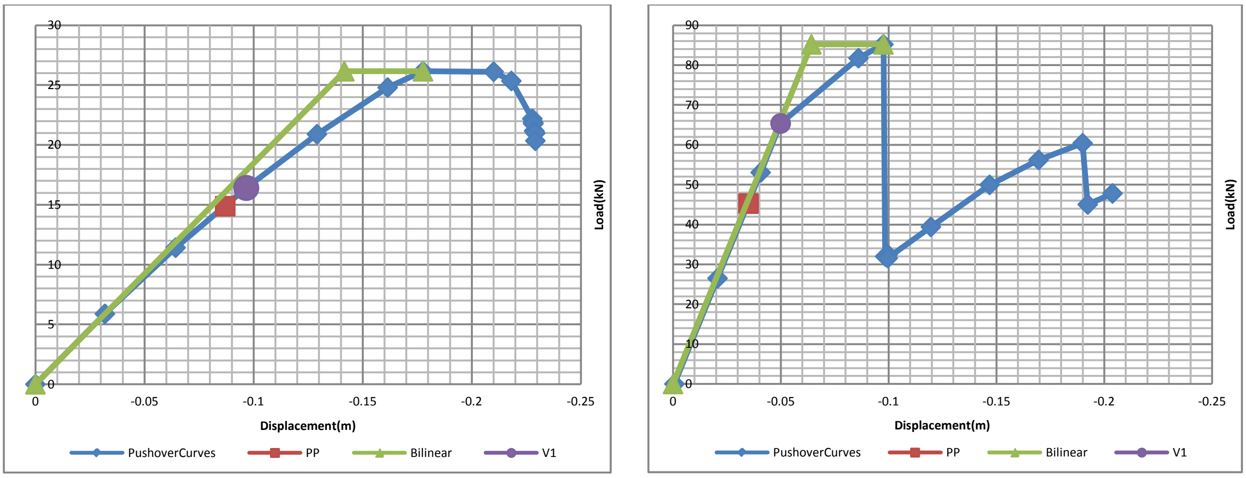


Figure 16- System B, Low seismic zone

a) down aisle

b) cross aisle

	μ	Ω	q
Down	1.25	1.59	2.00
Cross	1.52	1.30	1.98

Table 4- Ductility, Overstrength and q- factor for the case study B-Low seismic zone

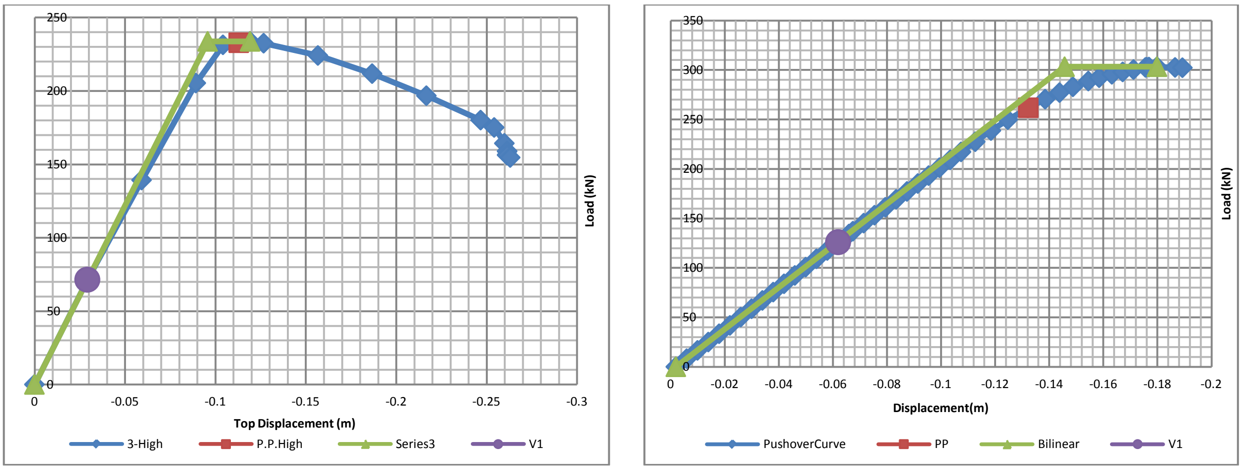


Figure 17- System C, High seismic zone

a) down aisle

b) cross aisle

	μ	Ω	q
Down	1.24	3.27	4.07
Cross	1.23	2.4	2.97

Table 5- Ductility, Overstrength and q- factor for the case study C-high seismic zone

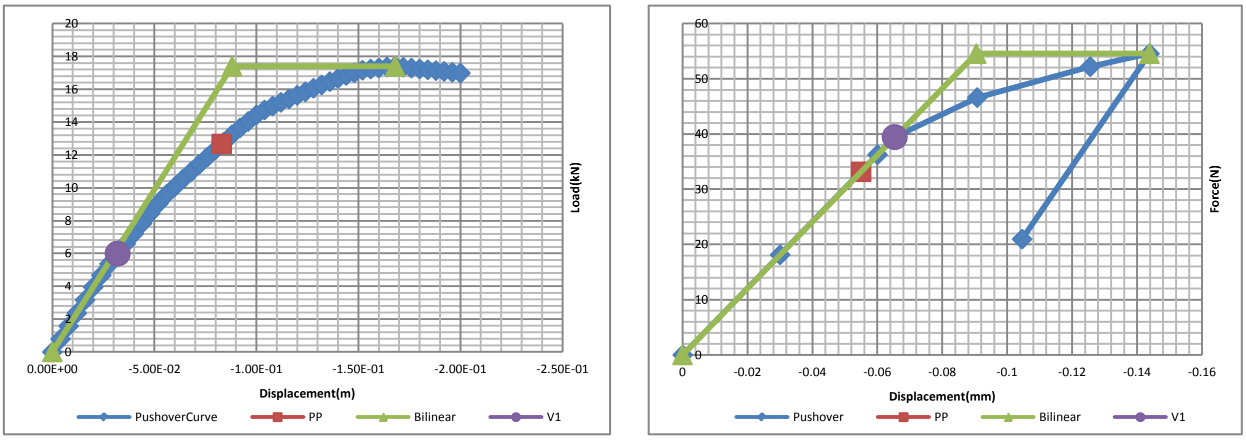


Figure 18- System C, Medium seismic zone

a) down aisle b) cross aisle

	μ	Ω	q
Down	1.90	2.90	5.51
Cross	1.58	1.38	2.2

Table 6- Ductility, Overstrength and q- factor for the case study C- Medium seismic zone

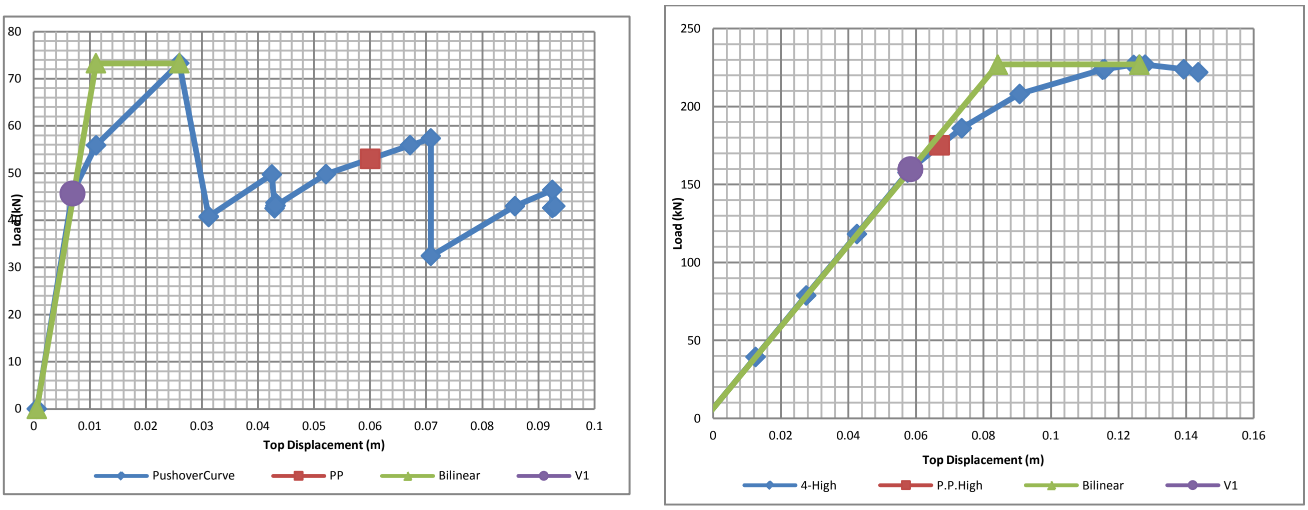
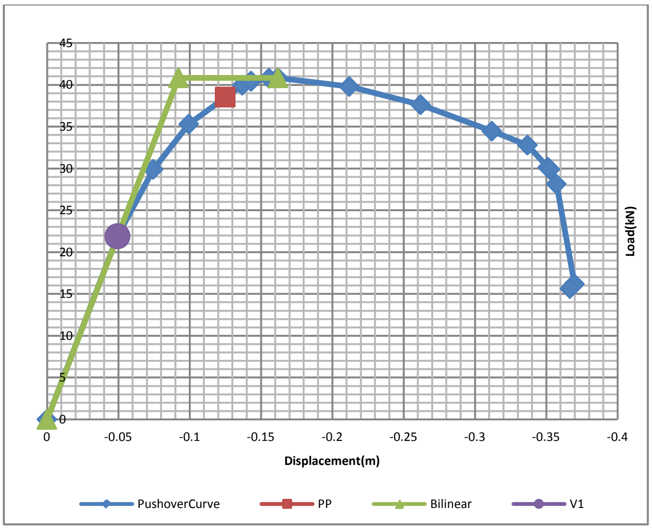


Figure 19- System D, High seismic zone

a) down aisle b) cross aisle

	μ	Ω	q
Down	2.34	1.59	3.72
Cross	1.49	1.42	2.12

Table 7- Ductility, Overstrength and q- factor for the case study D-high seismic zone



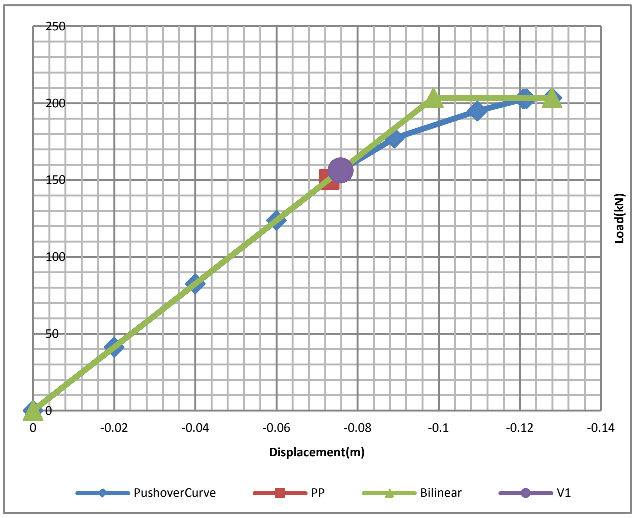
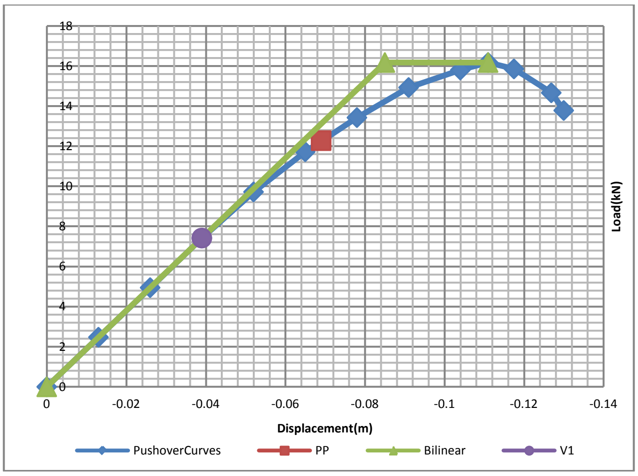


Figure 20- System D, Medium seismic zone

a) down aisle b) cross aisle

	μ	Ω	q
Down	1.75	1.86	3.27
Cross	1.29	1.30	1.68

Table 8- Ductility, Overstrength and q- factor for the case study D- Medium seismic zone



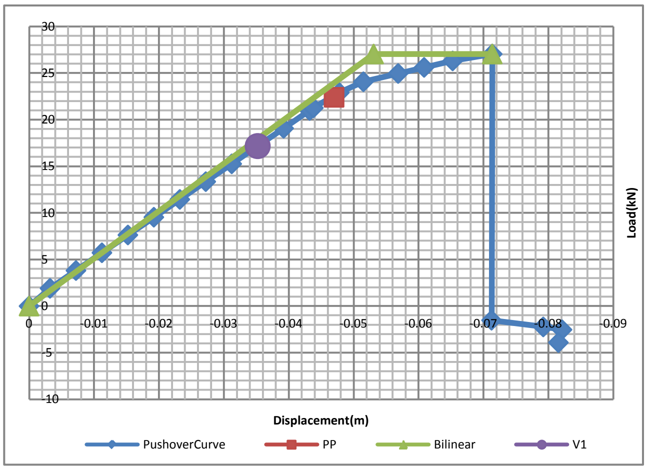


Figure 21- System D, Low seismic zone

a) down aisle b) cross aisle

	μ	Ω	q
Down	1.30	2.18	2.84
Cross	1.34	1.57	2.11

Table 9- Ductility, Overstrength and q- factor for the case study D- Low seismic zone

4 CONCLUSIONS

The previous calculations indicate that q- factor does not follow a specific rule. In general, in down aisle direction there can be calculated has higher values of q- factor than in cross aisle direction. In down aisle direction the braced structures appear to be more ductile compared with the unbraced ones due to the ductility provided by the tension diagonals. The beam end connectors and the base plate connections have a ductile response, although it is relative to the considered yield point. On the other hand, the bracing of the upright frame, which is activated in the cross aisle direction, is not really ductile since buckling; local buckling and bearing failure of thin walled plates appear.

As far as the q factors are concerned, for the analyses in down aisle direction they range from 2 to 5.47, whereas in cross aisle direction they range between 1.68 and 2.97. It should be noted that these values were determined based on the information that was provided by the IPs.

Acknowledgment

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