Canadian Conference - Pacific Conference on Earthquake Engineering 2023

Vancouver, British Columbia

June 25th – June 30th, 2023

Incremental dynamic analysis of steel storage racks subjected to Chilean earthquakes

Eduardo Nuñez[1*] and Ramon Mata[2]

[1]Professor, Department of Civil Engineering, Universidad Católica de La Santísima Concepción, Concepción, Chile.
[2]PhD Student, Department of Civil Engineering, Universidad Católica de La Santísima Concepción, Concepción, Chile.
[*]enunez@ucsc.cl (Corresponding Author)

ABSTRACT

Steel storage racks are structures commonly used by all industries in Chile; nevertheless, due the seismic hazard in the country and its configurations, these structures are highly vulnerable to earthquakes and no specific regulations exist to design them. In this research, the seismic performance of steel storage racks subjected to Chilean Earthquakes was evaluated using nonlinear pushover and nonlinear dynamic analysis. The studied models consider different heights and global slenderness ratios in both directions and soil types. Racks were evaluated using an Incremental Dynamic Analysis (IDA) according to FEMA P695. Deformation values were exceeded in the down-aisle direction and the use of braces was necessary to control the interstory drift and high deformation levels. Also, the expected level of damage for drift design limits in models unbracing is not enough to keep the operation of the structures and more severe regulations are necessary to achieve a performance in agreement with Chilean design philosophy.

Keywords: Steel storage racks, incremental dynamic analysis, steel structures, seismic behavior.

1. INTRODUCTION

Selective steel storage racks are commonly used by manufacturers in Chile. In recent years, the warehousing of goods has increased due to the effects of globalization and new market trends. In this sense, different suppliers have introduced in the Chilean market storage systems that are widely used in Europe and in the United States. However, the seismic hazard in Chile has higher recurrence and magnitude characteristics than in the countries where the new storage systems were implemented. On the other hand, evidence on the seismic behavior of racks in Chilean subduction earthquakes is limited. An experimental study on the behavior of braced steel storage rack systems using full-scale pushover tests was performed by [1]. The results showed that bracing connections must be designed for high overstrength levels in order to avoid brittle collapse. Subsequently, an experimental evaluation of the seismic behavior of unbraced steel storage racks was studied by [2]. The results showed the relationship between the behavior of beam-column connections and the ductility of the rack. In addition, the study provided design requirements to ensure the global ductility under seismic actions.

A nonlinear dynamic analysis to evaluate the seismic behavior of racks was performed by [3]. In this study, an advanced design procedure for the safe use of steel pallet racks in seismic zones is proposed. The study consisted of a case study with double-entry, medium-height racks subjected to two Italian earthquakes. A numerical study of selective storage systems using pushover analysis was performed by [4]. The study focused on the calibration of connections from a previous experimental study where five connection configurations were studied. Results showed that rack models with bolted and hooked connections achieved a better performance in terms of resistance and stiffness in comparison to hooked connections. Subsequently, a dynamic incremental analysis of the racks was conducted by [5]. The seismic behavior of selective racks was conducted by [6]. The uplift and overturning phenomenon of three full-scale selective racks considering three types of base plates: ductile, high strength and unanchored was studied. Results indicated that a failure of the foundations can be reached for loads higher than 1.5 times the design level. In addition, unanchored racks failed due to the overturning phenomenon. However, ductile base plates reached up to 2.3 times the design level.

An experimental study of selective racks subjected to cyclic loading and shaking table was conducted by [7]. Results showed that the racks studied achieved up to 10% drifts without collapse. Additionally, a numerical model to predict the seismic response of the racks, including pallet slip was proposed. Another experimental dynamic test using shaking table was performed by [8]. In this study, pallet slip in industrial racking systems subjected to earthquake-induced actions was characterized. Different beam and pallet materials were studied. Results showed slippage at very low acceleration levels. For biaxial seismic testing, lower bound acceleration in the cross-aisle direction was higher than in dynamic cyclic tests, whereas the opposite was observed in the down-aisle direction.

In research performed by [9], a methodology for the seismic vulnerability assessment of steel racks using fragility curves was proposed. The obtained results are addressed to engineering demand parameters, allowing carrying out a damage mitigation of steel storage racks. A parametric study of the mechanical properties of the connection members was performed by [10]. In this study, the influence on the structural response of the connections was evaluated. The study examined the application of Monte Carlo simulation method to obtain the impact of these variables on the design parameters. The results obtained showed that the flexural resistance and initial elastic flexural stiffness of racks connection are highly controlled by the local response of connection component. Also, this effect is influenced by the uncertainty in steel mechanical properties and geometrical features. The evaluation of progressive collapse in steel storage racks was studied by [11]. The results showed that racks are susceptible to global failure, especially if the bracing is in few spans and the stiffness of the connections is low. Nonlinear static thrust analysis can provide satisfactory results with the least computational effort. On the other hand, dynamic nonlinear analysis of racks with a new constitutive model of beam-column connections was performed by [12]. From defining four parameters, the model can simulate the pinching and low cycle fatigue of a beam-column connection.

An experimental study of the seismic behavior of rack structures was performed by [13]. The beam-to-column connection subjected to cyclic loads with hooks fabricated with the beam, inserted in special slots at the columns was analyzed. The results obtained showed a continuous hardening, enabling the connections to reach about half of the beam plastic moment. Nonlinear analyses of the rack structure from 2D models subjected to different seismic conditions were performed, including one record of a Chilean earthquake. A recent numerical study of selective racks subjected to Chilean earthquakes was performed by [14]. In this study, the seismic behavior of selective steel storage racks subjected to earthquakes in Chile was evaluated using nonlinear pushover analysis and nonlinear dynamic analysis. The studied models examine different heights and global slenderness ratios in both directions and soil types. The design of the racks was performed in accordance with the NCh2369 standard [15]. The design considered the possibility of exceeding the lateral deformation limit established in the standard of 1.5% if P-delta effects are considered according to [15]. Additionally, the use of horizontal bracing was not deemed, which affected the behavior of the models due to torsional effects. A total of 22 Chilean seismic records were used to perform the seismic evaluation. Results showed that not all models reach a response factor R=4, established in the standard [15], deformation values were exceeded in the down-aisle direction and the use of bracing towers was necessary to avoid high stress levels in the columns and high deformation levels. However, a dynamic incremental analysis was not used to establish robustly reliable values of overstrength, ductility and response reduction factor.

In summary, previous studies have shown that racks designed according to [15] can achieve R values lower than 3 [13, 14]. The goal of this research is to assess the seismic performance of selective steel storage racks considering an Incremental Dynamic Analysis (IDA) [16] established in FEMA P695 [17]. Additionally, bolted non-engaged connections commonly used in Chile, horizontal bracing and subductive Chilean earthquakes were deemed. Moreover, a review of Reduction factor (R), established in Chilean seismic code is performed.

2. STRUCTURAL ARCHETYPES: SELECTION AND DESCRIPTION OF RACK MODELS

In Chile, the use of selective steel storage racks has been preferred for years due to their storage capacity and adaptability to space. Selective racks are an arrangement of columns known as up-rights and beams called pallet beams. All its components are designed according to specifications [20]. The lateral load resisting system consists of unbraced frames in the down-aisle direction and braced frames in cross-aisle direction. The beam-column connection is a moment connection with limited strength and stiffness. In this sense, its behavior is highly nonlinear and has a direct effect on the seismic performance of the rack structure. The connections between braces and columns have a pinned behavior and can be considered as uniquely resistant to axial load. When drift is very high, the use of lateral bracing in the down-aisle direction may be required. This improves lateral stiffness and strength in the desired direction [4][5][9][13][14].

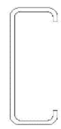
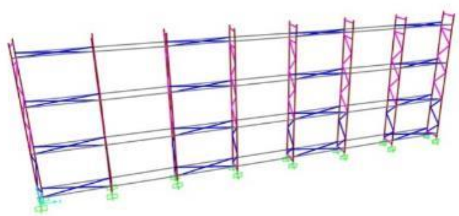
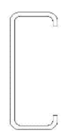
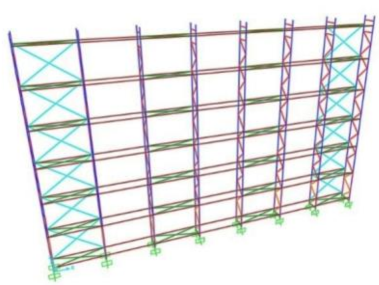
In this research, global slenderness in each direction is considered in order to evaluate the influence of slenderness on rack performance. Therefore, different global slenderness ratios were studied. The slenderness parameter was considered in the cross-aisle slenderness (λCA=H/Ly; H=total height and Ly=width in the cross-aisle direction) and in the down-aisle slenderness (λDA=H/Lx; H=total height and Lx=length in the down-aisle direction). In Table 1, the total height (H) and slenderness parameters per archetype studied are shown. Figure 1, shows the four types of selective racking structures studied: CB, is a selective rack type with short-low slenderness ratio; CA, is a selective rack with short-high slenderness ratio; LB, is a selective rack with large-low slenderness ratio; and LA is a selective rack with large-high slenderness ratio.

Table 1. Slenderness by type of racks analyzed.

Model	H (m)	Lx (m)	Ly (m)	λ_{DA}	λ_{CA}

CB	7.88	18.90	0.75	0.42	10.51
CA	13.58	35.10	0.75	0.39	18.11
LB	7.88	18.90	0.75	0.42	10.51
LA	13.58	35.10	0.75	0.39	18.11

The ASTM-A36 (Young's Module E=210000 MPa, yielding stress Fy=250 MPa and Maximum stress Fu=400 MPa) [25] material was considered for all members, according to [14]. This material is commonly used in the fabrication of cold-formed sections for racks in Chile. The analysis and seismic design were performed using modal response spectrum analysis (MSRA) according to [15]. Dead and live loads were considered. Two-unit loads (unit load = 9.8067 kN) were considered to simulate the goods on pallet beams. The seismic mass was considered from 100% dead plus 100% unit- loads.

Archetype	Up-rights	Pallet beam	Lateral brace in cross-aisle direction	Lateral brace in down-aisle direction
Short-Low **CB**	TX 100 × 105 × 3 	TC 100× 50 × 2.5 	CA45 × 22× 7.5 x 2 	-
				
Short-High **CA**	TX 140 × 105 × 3 	TC 100 × 50 × 2.5 	CA 70 × 26 × 8.5 × 3 	Tube 60 × 60× 3 
				

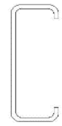
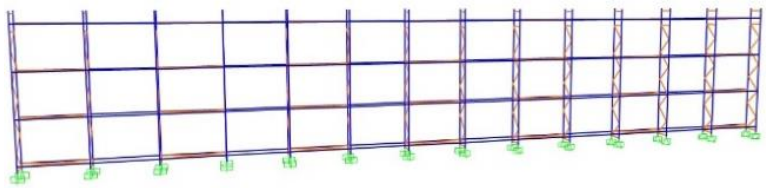
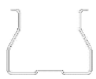
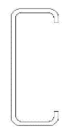
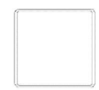
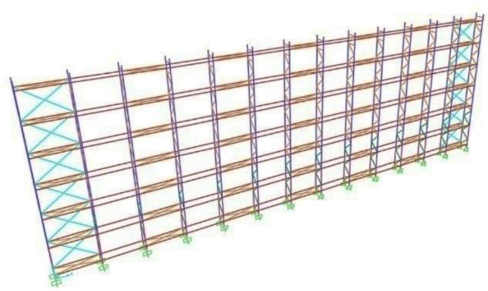
Large-Low **LB**	TX 100 × 105 × 3 	TC 100× 50 × 2.5 	CA 70 × 26 × 8.5× 3 	-
				
Large-High **LA**	TX 140 × 105 × 3 	TC 100× 50 × 2.5 	CA 70 × 26 × 8.5× 3 	Tube 75× 75× 3 
				

Figure 1. Dimensional properties of rack members by type of archetype (dimensions in millimeters).

On the other hand, to quantify the seismic demand, the two higher zones with maximum effective acceleration were used (Zone 2, Ao=0.30g and Zone 3, Ao=0.4g) and the three types of soils with higher amplification were considered (soil type 2, soil type 3 and soil type 4) according to [15]. The response reduction factor R=4 is used for all models. The steel rack structures were modeled using the software SAP2000 v22 [21]. Up-rights, beams and brace members were modeled using frame elements with two end nodes and six degrees of freedom per node. The plastic hinges of typical moment connection used in Chile were numerically studied according to [22] and calibrated from an experimental study performed by [23]. In the linear analysis, the elastic stiffness of beam-to-column connections was considered in all models, while in nonlinear dynamic analysis, a pivot hysteretic model proposed by [24] was used to allow the cyclic response of moment connections in models unbraced and braced. The pivot hysteresis parameters resulting from calibration are mentioned as follows: $\alpha1 = 100$; $\alpha2 = 100$; $\beta1 = 0.12$; $\beta2 = 0.12$; $\eta = 0$. The αi values are used to estimate the resistance degradation, while the βi values are used to control the pinching. Finally, the η value is used to consider the stiffness degradation. In Figure 2, the hysteretic response of bolted moment connection used is shown.

For the linear analysis, base connections in the down-aisle direction were modeled considering its elastic stiffness from a previous numerical model using combined finite elements and component method, while in cross-aisle direction a pinned restraint was deemed. In order to nonlinear analysis, the plastic hinges were defined as concentrated damage in up-right members from default fiber-based hinges considering P-M2-M3 interaction (axial-primary moment-secondary moment) according to [4][5]. The base connection was considered as pinned in cross-aisle direction, while a default fiber-based hinges considering P-M3 were used in down-aisle direction [4][5]. The second-order effects were defined from P-delta plus large displacements analysis modifying the stiffness matrix; equilibrium in deformed position was estimated from a previous nonlinear case [14]. The load cases were started from the P-delta analysis case, which the stiffness matrix was previously modified. Similar to studies performed by [4][5][14], the local bucking was not deemed in the scope of this research.

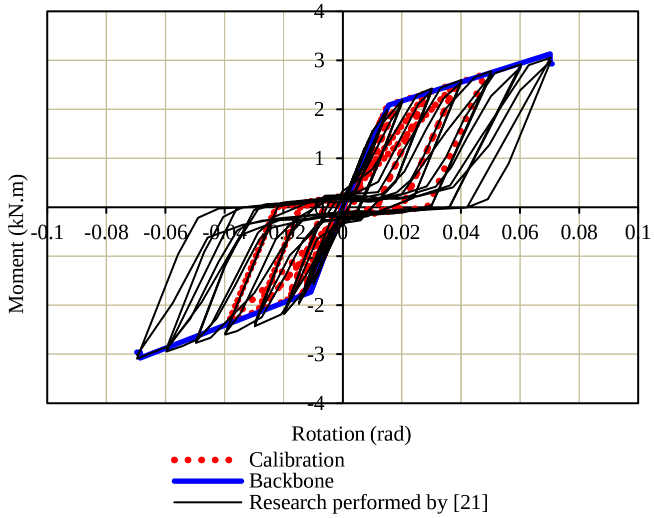


Figure 2. Normalized moment rotation curve of typical bolted joint used in Chilean racks calibrated from study performed by [21].

A total of sixteen models were designed for different combinations of soil, seismic zone and slenderness, obtaining their elastic response. To satisfy the drift and strength requirements in accordance with [15], lateral bracing in the down-aisle direction was required in the short-high (CA) and large-high (LA) models. Additionally, some models of short-low (CB) such as CBZ2S4, CBZ3S3 and large-low (LB) such as LBZ2S4 and LBZ3S3 were also required lateral bracing in down-aisle direction. The FEMA P695 [17] evaluation methodology two types of sets of ground motion records for collapse evaluation using incremental dynamic analysis, referred to as the Far-Field and Near-Field record sets could be used. The set selected includes 22 records (44 horizontal components) of ground motions. The methodology specifies the use of Far-Field record set or ground motions from sites located greater than or equal to 10km from fault rupture for collapse evaluation for archetype designed for seismic design category B, C or D, according to ASCE 7 [26]. Steel storage racks in accordance with the Chilean code are classified as an equivalent of Category D according to ASCE 7 [26]. These records are representative of Chilean subductive ground motions. The records must have a source magnitude higher to Mw = 6.0 or PGA≥0.2g, which are levels generally representative of the threshold of structural damage. In this research, the magnitude of records selected varies between 6.3≤Mw≤8.8, and the PGA varies from 0.211g to 0.87g. The response spectrum was calculated from seismic records using the Nigam, N. & Jennings, P. (1968) [27] method. According to FEMA P695 [17], the median for the group of response spectrum values was obtained, as well as an estimation of the scaling factor per archetype to scale the registers to Maximum Considered Earthquake (MCE). Details of MCE, total response spectrum and median spectrum obtained are shown in the Figure 3.

Generally, the MCE commonly used for seismic performance evaluations in buildings is obtained from the spectrum established in NCh2745 [28]. However, this spectrum is characterized for isolated building structures, which have a behavior different to steel storage racks with a fixed base. In this sense, the MCE considered in this research was obtained from the NCh433 [29] multiplied by a factor of 1.4, which is the result of revisions made from the data obtained from the earthquake of 27 F in 2010, obtaining a smoother and more representative spectral shape of the hazard throughout the range of periods considered. On the other hand, the elastic spectrum of NCh2745 [28] does not fit the spectra of records obtained in the intermediate and long period zone. This is valid for isolated structures and not for fixed base structures, which may induce excessive deformation demands not applicable to steel racks structures.

3. NONLINEAR ANALYSIS

In this research, seismic design parameters in steel storage racks were obtained using nonlinear static analysis (pushover) and nonlinear dynamic analysis. These analyses were performed to the designed archetypes in section 3, in accordance with FEMA P695 [17]. In the pushover analysis, the seismic assessment of rack models was performed using a triangular load pattern proportional to the seismic design forces in down-aisle and cross-aisle directions, according to FEMA 440 [31]. A uniformly distributed load of 3.6 kN/m was used in pallet beams. No mechanisms of dissipation between pallets and beams were considered; therefore, 100% of the load is contemplated for all analysis cases. In pushover analysis, a node control on the top of structure was assigned for displacement control of drifts. The target displacement was estimated from the method of coefficients, according to FEMA 356 [30]. The capacity curves were defined as base shear vs. displacement of single degree-of-freedom (SDOF) oscillator, equivalent to the 3D structure. The performance point was obtained from the intersection between capacity curve and spectrum demand. The response reduction factor $R=R_{\mu} \times R_{\Omega}$ was obtained. The ductility factor R_{μ} is the elastic shear divided by maximum shear, and R_{Ω} is the overstrength of the structure obtained from the maximum shear divided by design shear. The ductility $\mu=\Delta_{max}/\Delta_{y}$ was obtained, dividing the maximum displacement by the yield displacement from the linearization of the capacity curve [17][31].

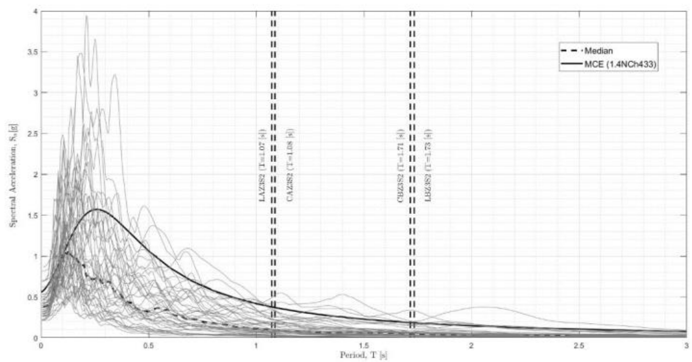
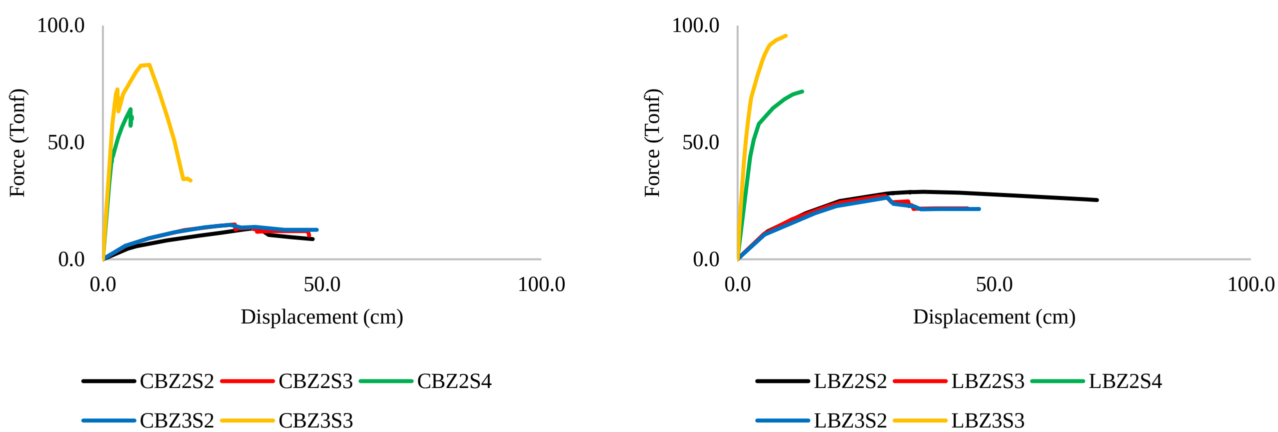
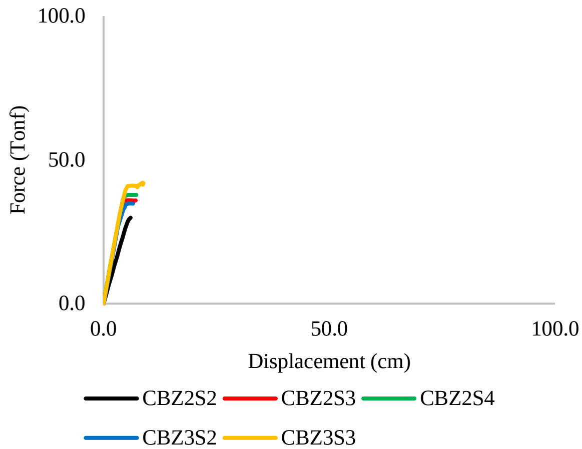


Figure 3. Scaled spectra of ground motions records used for this research.

In Figure 4, the failure mechanisms in short-low models are shown in the down-aisle direction and cross-aisle direction. In the down-aisle direction, plastic hinges are mainly concentrated in beams for unbraced models, while in braced models the hinges are developed in braces and beams. In the cross-aisle direction, hinges in beams are reached. Capacity curves are shown for both down-aisle and cross-aisle directions. In this sense, higher ductility and deformation are obtained in down-aisle direction for models without braces, while in models with braces 8 times more stiffness and 2 times less ductile behavior are achieved. In the cross-aisle direction the plastic deformation is limited, therefore, the behavior can be assumed as elastic for all archetypes. Likewise, the behavior of the braced models in the down-aisle direction is similar to behavior of models in cross-aisle direction. In Figure 5, plastic hinges are developed in beams for large-low (LB) models without braces, while a combined mechanism (plastic hinges in braces and beams) is reached in large-low (LB) models with lateral brace in down-aisle direction. In Figure 8, a ductile behavior in down-aisle direction and brittle behavior in cross-aisle direction for large-low models is shown. This behavior is similar to short-low models reported in Figure 4. Furthermore, the failure mechanisms obtained are consistent with the nonlinear performance. In the short-high models analyzed the plastic hinges are reached mainly in beams, however some hinges are developed in braces for CAZ2S3 model (see Figure 6). The nonlinear behavior is controlled by a ductile mechanism for CAZ2S3 model. However, a limited plastic behavior was obtained for CAZ2S2 and CAZ3S2 models. In this case, the ductile behavior is related with a higher global slenderness. In Figure 7, a higher displacement was achieved in cross-aisle direction in comparison to down-aisle direction. The large-high models reached a behavior controlled by failure mechanism of beams and braces in down-aisle direction.



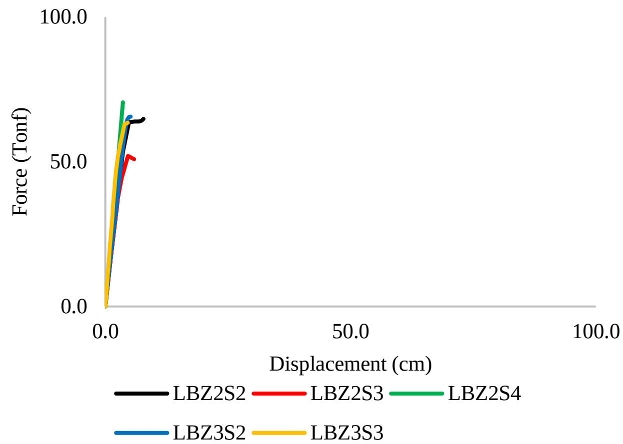
Down-aisle direction



Cross-aisle direction

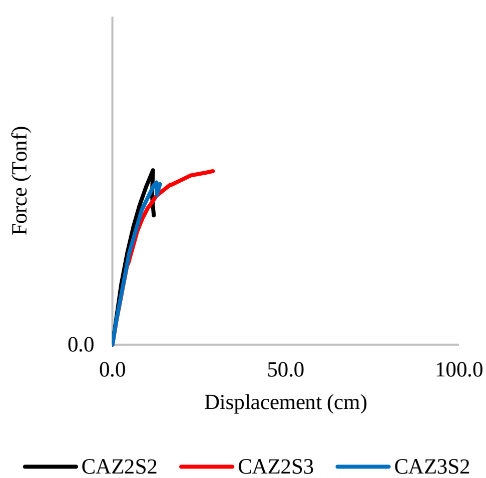
Figure 4. Curve of capacity in models short-low for pushover analysis.

Down-aisle direction

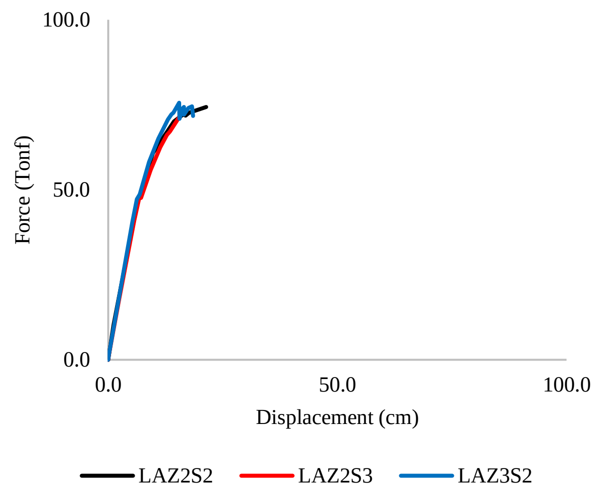


Cross-aisle direction

Figure 5. Curve of capacity in models large-low for pushover analysis.



Down-aisle direction



Down-aisle direction

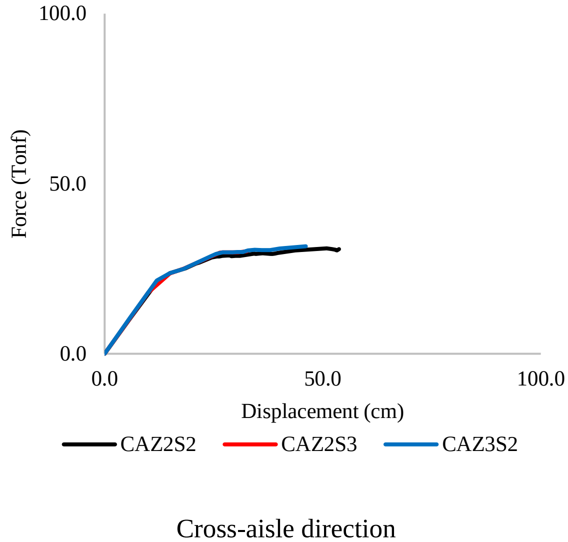


Figure 6. Curve of capacity in models short-high for pushover analysis.

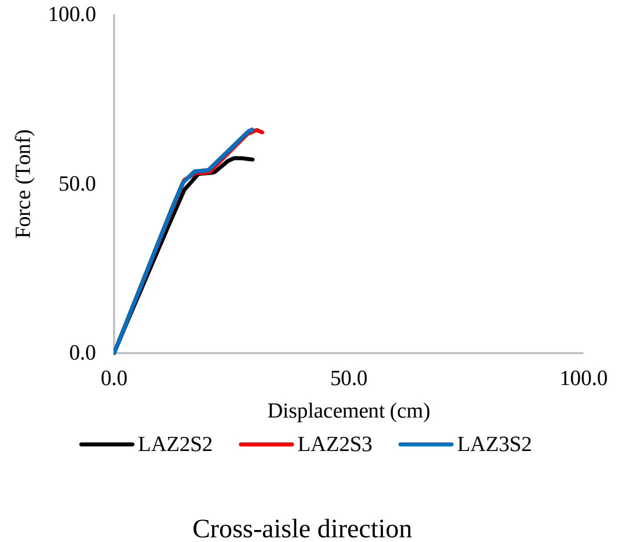


Figure 7. Curve of capacity in models large-high for pushover analysis.

4. INCREMENTAL DYNAMIC ANALYSIS

Incremental dynamic analysis (IDA) [16] is a parametric method to estimate the structural performance from a probabilistic approach of a group of archetypes under diverse ground motion records. In this research the procedure established in FEMA P695 [17] is used to assess the seismic performance of steel storage racks and estimate its collapse probability. A group of nonlinear dynamic analyses with scaled ground motion records described before are developed in order to compute the response in terms of story drift, base shear, and spectral acceleration. The analysis was developed in the software SAP2000 [25], which can capture the strength and stiffness degradation in structural components at large deformations. The elements were modeled as frame elements with concentrated plastic hinges calibrated from [14] in the beams and fiber hinges in the columns and braces. The nonlinear dynamic analysis was conducted under the factored gravity load combination given in FEMA P695 [17]. The direct integration time method was employed to minimize errors in the analysis and achieve convergence at large inelastic deformation.

This nonlinear dynamic analysis is used to establish the median collapse capacity and collapse margin ratio for each of the index archetype models. In this sense, each archetype is subjected to each individual ground motion record, which is scaled to increasing intensities until the structure reaches collapse. For the structures analyzed, collapse occurs when the first hinge located at the beam or column reaches the maximum rotation capacity, according the moment-rotation curve defined in section 3. At this time, the spectral acceleration of the ground motion is computed, obtaining various points of drift and spectral acceleration for each increase of intensities until the collapse. These points are plotted in terms of the spectral intensity of the ground motion (on the vertical axis) versus maximum story drift ratio recorded in the analysis (on the horizontal axis) for each nonlinear dynamic analysis and ground motion selected, obtaining the IDA curves for the archetype analyzed. Since three dimensional models are analyzed, the nonlinear dynamic analyses were applied to the twenty-two record pairs, and applied twice to each model: once with the ground motion records oriented along the cross-aisle direction, and again with the records rotated 90 degrees. IDA curves were obtained for each analysis direction from more than 880 nonlinear dynamic analyses, which are shown in figures 8, 9, 10 and 11, for each archetype.

The CBZ3S2 and LBZ3S2 models (without down-aisle lateral bracing) were selected, while the CAZ3S2 and LAZ3S2 models (with down-aisle lateral bracing) were selected. Horizontal bracing was considered for all models considering the improvement offered by the use of such elements. The selection criteria used contemplates the possibility of evaluating the performance of models located in similar seismic zones and soil types, varying the effect of lateral bracing on the seismic behavior in terms of Reduction factor (R) and Overstrength (Ω). This reduction of models analyzed allowed to reduce the analysis time without loss of representativeness.

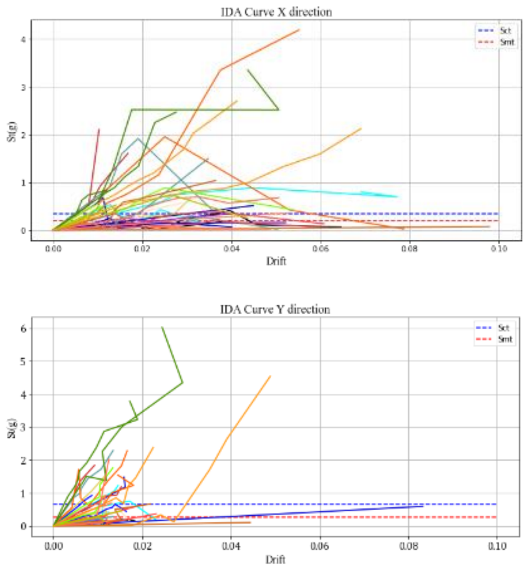


Figure 8. IDA curves for CBZ3S2 archetype.

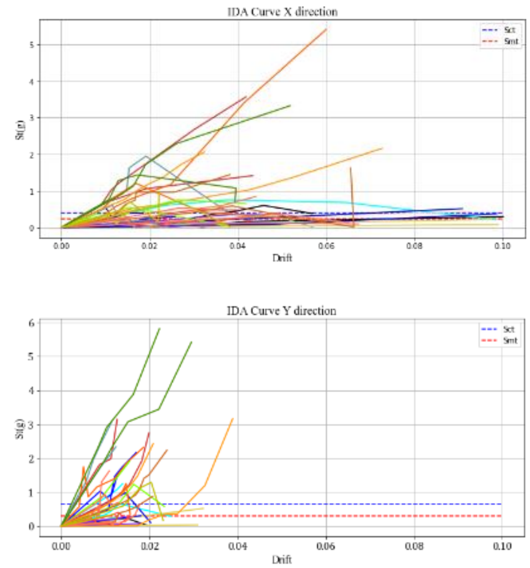


Figure 9. IDA curves for LBZ3S2 archetype.

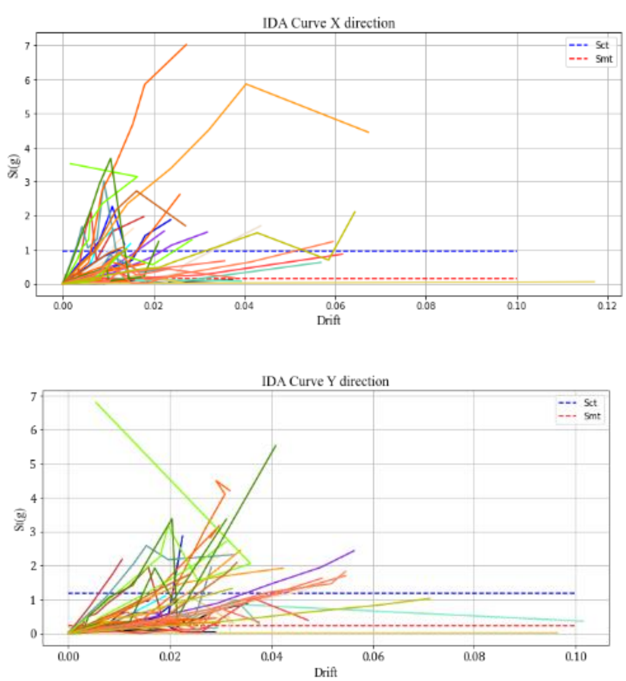


Figure 10. IDA curves for CAZ3S2 archetype.

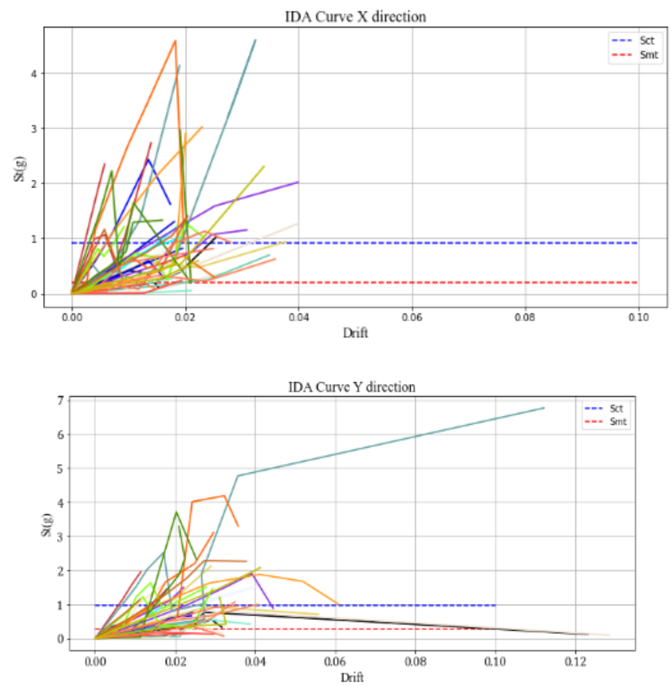


Figure 11. IDA curves for LAZ3S2 archetype.

From these curves it is evident that the collapse behavior in the X direction (down-aisle direction) is highly brittle, with a lower resistance against most seismic records. On the other hand, in the Y direction (cross-aisle direction) the behavior is essentially elastic, with hardening and collapse at the end. Furthermore, for higher height models the collapse mechanism is prevalent in the cross-aisle direction, because these models have a higher lateral stiffness in the down-aisle direction due to the presence of posterior diagonal braces. This allows the steel storage racks to reach a higher deformation before the collapse appears in the cross-aisle direction. Using this IDA results, a collapse fragility curve can be determined through a Log-normal cumulative distribution function (CDF) defined by the median collapse intensity (S_{CT}) and the standard deviation of the natural logarithm (β_{RTR}). Initially, FEMA P695 [17] allows to consider β_{RTR} as 0.4 for systems with ductility up to 3, as shown in pushover analysis results. However, this value can be verified calculating the total system collapse uncertainty, according to chapter 7 of FEMA P695 [17], modifying the collapse fragility curve, as shown in figure 12.

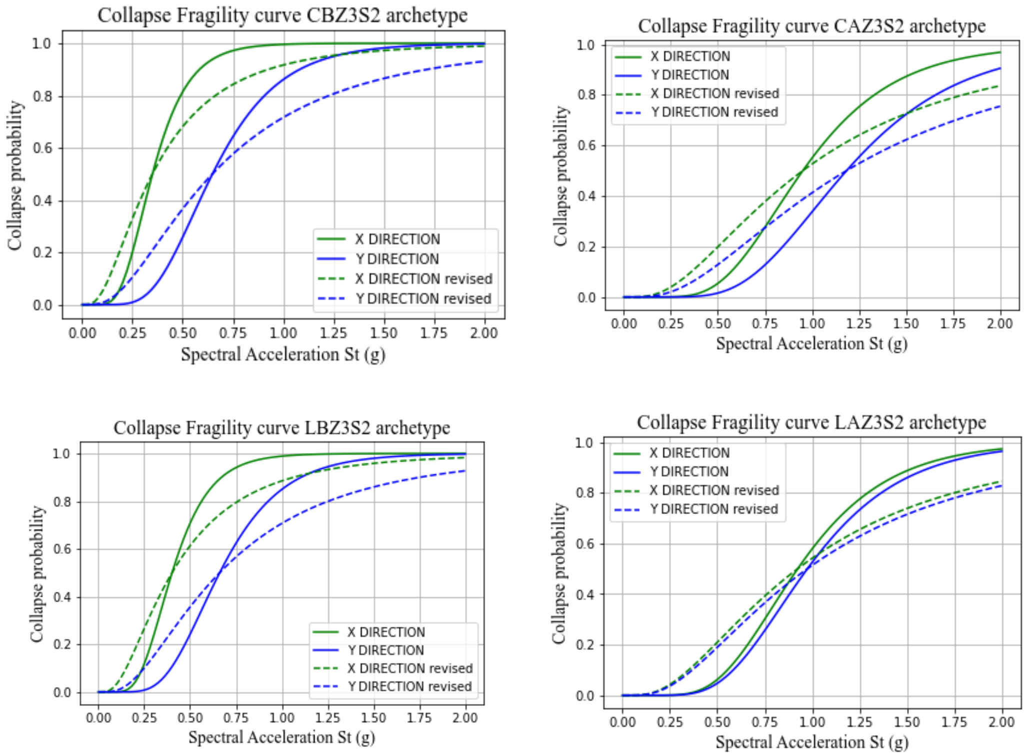


Figure 12. Collapse fragility curves for all archetypes analyzed.

Models with high global slenderness (CAZ3S2 and LAZ3S2) reached collapse probability values between 0.4 - 0.6 for 1.0g of spectral acceleration in down-aisle and cross-aisle directions. While models with low global slenderness (CBZ3S2 and LBZ3S2) reached collapse probability values between 0.7 - 0.9 for 1.0g of spectral acceleration in down-aisle and cross-aisle directions. Therefore, models with lateral brace in down-aisle direction achieved a lower collapse probability. Additionally, the collapse probability is higher in down-aisle direction for all models studied. The results show that for a drift limit (0.015) established in [15], the severe damage probability can to reach a 30% in down-aisle direction. However, the severe damage probability can to reach 80% in cross-aisle direction, which introduces a big uncertainty to the design, according to the Chilean design philosophy. An increase of severe damage probability was obtained in down-aisle direction as the global slenderness is higher. The archetypes with low height achieved a collapse probability greater than models with high height. Furthermore, the content that is stored in the steel storage racks is susceptible to falling, when the drifts are under the 0.015 limit. From the results of nonlinear dynamic analysis developed to IDA, the strength reduction factor (R) was estimated for all ground motions records scaled to MCE. In this sense, R was computed 44 times for each archetype according to the procedure described in [17] and the overstrength factor obtained from Pushover analysis. The minimum, maximum, mean and median values of R are shown in the Table 2.

Table 2. Strength reduction factors obtained from nonlinear dynamic analysis.

Archetype	***X direction***				***Y direction***			
	R_{min}	R_{max}	R_{mean}	R_{median}	R_{min}	R_{max}	R_{mean}	R_{median}
CBZ3S2	3.47	10.16	4.81	4.31	4.52	11.35	5.17	4.75
LBZ3S2	3.31	11.09	4.57	4.07	4.22	13.36	5.30	4.62
CAZ3S2	3.72	7.73	5.17	5.05	4.84	6.08	5.25	5.10
LAZ3S2	4.23	8.75	5.17	5.05	4.91	6.61	5.33	5.17

5. CONCLUSIONS

In this research, the seismic behavior of selective steel storage racks subjected to Chilean earthquakes was evaluated using nonlinear pushover analysis and nonlinear dynamic analysis. Four types of selective steel storage racks were analyzed for two seismic zones and four types of soils. The nonlinear static analysis was performed in 32 different structures in both directions. In addition, an incremental dynamic analysis was carried out to analyze the seismic performance of the common archetypes subjected to 22 Chilean seismic records until collapse. Fragility curves were obtained at different levels of damage and the seismic design parameters were evaluated for overstrength and strength reduction factor according to FEMAP695 recommendations. The main conclusions of this study are described as follows:

- The beam-to-column joint connection plays an important role in the seismic behavior of steel storage racks. A highly nonlinear behavior controls the seismic behavior of this type of structures. Therefore, a great elastic capacity should be proportioned to the structural system.
- To satisfy the design requirements according to Chilean Code, lateral braces in down-aisle and cross-aisle direction are commonly necessary to control the displacements, mainly for racks with high global slenderness. Consequently, the direction of the collapse mechanism can be to change if braces are used in both directions.
- The IDA curves obtained from down-aisle and cross-aisle direction exhibit a different behavior in comparison to conventional industrial structures. An elastic behavior was obtained in cross-aisle direction, while a brittle behavior was reported in down-aisle direction. Furthermore, when diagonal braces are incorporated elastic behavior is observed in both directions.
- The seismic performance of the common archetypes of steel storage racks evaluated from IDA analysis was satisfactory according to FEMA P695 guidelines. However, the severe damage probability at the Chilean design drift can reach 50%. This level of damage does not allow to keep the operation of the structure and its contents according to the Chilean Industrial design philosophy. In this sense, another margin ratio different from the CMR must be evaluated to assess the seismic performance of steel storage racks.

REFERENCES

[1] Kanyilmaz, A.; Brambilla, G.; Chiarelli, G.; Castiglioni, C. (2016) “Assessment of the behavior of braced steel storage racking systems by means of full-scale push over tests”. *Thin-Walled Struct.* 107, 138–155.

[2] Kanyilmaz, A.; Castiglioni, C.; Brambilla, G.; Chiarelli, G. (2016) “Experimental assessment of the seismic behavior of unbraced steel storage pallet racks”. *Thin-Walled Struct.* 108, 391–405.

[3] Bernuzzi, C.; Simoncelli, M. (2016). “An advanced design procedure for the safe use of steel storage pallet racks in seismic zones”. *Thin-Walled Struct*. 109, 73–87.

[4] Yin, L.; Tang, G.; Li, Z.; Zhang, M.; Feng, B. (2018). “Responses of cold-formed steel storage racks with spine bracings using speed-lock connections with bolts I: Static elastic-plastic pushover analysis”. *Thin-Walled Struct*. 125, 51–62.

[5] Yin, L.; Tang, G.; Li, Z.; Zhang, M.; Feng, B. (2018). “Responses of cold-formed steel storage racks with spine bracings using speed-lock connections with bolts II: Nonlinear dynamic response history analysis”. *Thin-Walled Struct*. 125, 89–99.

[6] Maguire, J.; Teh, L.; Clifton, G.; Tang, Z.; Lim, J. (2020). “Cross aisle seismic performance of selective storage racks”. *J. Constr. Steel Res*., 168, doi: 10.1016/j.jcsr.2020.105999.

[7] Jacobsen, E.; Tremblay, R. (2017). “Shake table testing and numerical modelling of inelastic seismic response of semi-rigid cold-formed rack moment frames”. *Thin-Walled Struct*. 119, 190–210.

[8] Castiglioni, C.; Drei, A.; Kanyilmaz, A.; Mouzakis, H. (2018). “Earthquake-induced pallet sliding in industrial racking systems”. *J. Build. Eng.*, 19, 122–133.

[9] Gabbianelli, G, Cavalieri, F. and Nascimbene, R. (2020). “Seismic vulnerability assessment of steel storage pallet racks”. *Ingegneria Sismica*, 37(2), pp. 18-40.

[10] Gusella, F.; Arwade, S.; Orlando, M.; Peterman, K. (2019). “Influence of mechanical and geometric uncertainty on rack connection structural response”. *J. Constr. Steel Res*. 153, 343–355.

[11] Dubina, D.; Marginean, I.; Dinu, F. (2019). “Impact modelling for progressive collapse assessment of selective rack systems”. *Thin-Walled Struct.* 143, doi:10.1016/j.tws.2019.106201.

[12] Jovanović, Đ.; Ẑarković, D.; Vukobratović, V.; Brujić, Z. (2019). “Hysteresis model for beam-to-column connections of steel storage racks”. *Thin-Walled Struct.* 189, 189–204.

[13] Aguirre, C. “Seismic behavior of rack structures”. (2005) *J. Constr. Steel Res.*, 61, 607–624.

[14] Nuñez, E., Aguayo, C. and Herrera, R. (2020). “Assessment of the Seismic Behavior of Selective Storage Racks Subjected to Chilean Earthquakes” *Metals* 10, no. 7: 855.

[15] INN. NCh 2369.Of2003. Earthquake Resistant Design of Industrial Structures and Facilities; Instituto Nacional de Normalización: Santiago, Chile, 2003.

[16] Vamvatsikos D., and Cornell C.A. (2002) *Incremental dynamic analysis*. Earthquake Engineering & Structural Dynamics 31: 491–514.
[17] Federal Emergency Management Agency (FEMA) (2009) Quantification of Building Seismic Performance Factors. FEMA P695, 1 June, Washington, DC: Federal Emergency Management Agency.
[18] Farzin Zareian, Carlos Aguirre Juan Felipe Beltrán, Ernesto Cruz, Ricardo Herrera, Roberto León, Arturo Millán, Alejandro Verdugo. "Reconnaissance Report of Chilean Industrial facilities Affected by the 2010 Chile Offshore Bío-Bío Earthquake". *Earthquake Spectra*, 2012: 513-532.
[19] INE (2010), "Análisis económico 2010", Documento técnico – Instituto Nacional de Estadísticas, Santiago de Chile, Chile.
[20] American Iron and Steel Institute - AISI. North American Specification for the Design of Cold-Formed Steel Structural Members, 1st ed., Washington, DC, USA, 2012.
[21] CSI, "SAP2000 v.22: Integrated Software for Structural Analysis and Design." CA, USA, 2020.
[22] López, C. (2017). "Caracterización mecánica de uniones en sistemas de almacenamiento industrial construido con perfiles conformados en frio". *Thesis to obtain the profesional title of Civil Engineer. Licentiate Thesis,* Universidad Católica de la Santísima Concepción, Concepción, Chile. (In Spanish).
[23] Pavéz, I. (2017) "Análisis Experimental del Comportamiento de Elementos Conformantes de Sistemas de Almacenaje en Diversas Condiciones de Carga". *Thesis to obtain the professional title of Mechanical Engineer*. Licentiate Thesis, University of Concepción, Concepción, Chile. (In Spanish).
[24] Dowell, O.K.; Seible, F.; Wilson, E.L. "Pivot hysteresis model for reinforced concrete members". *Acids Struct*. J. 1998, 95, 607–617.
[25] American Society for Testing and Materials - ASTM International A36/A36M-14 Standard Specification for Carbon Structural Steel; ASTM International: West Conshohocken, PA, USA, 2014.
[26] American Society of Civil Engineers (ASCE) (2017) Minimum Design Loads and Associated Criteria for Buildings and Other Structures, ASCE/SEI 7-16. Reston, VA: American Society of Civil Engineers.
[27] Nigam, N.C.; Jennings, P.C. "Calculation of response spectra from strong-motion earthquake records. Bull. Seism. Soc. Am. 1969, 59, 909–922.
[28] Instituto Nacional de Normalización - INN. NCh 2745.Of2013. Analysis and Design of Buildings with Seismic Isolation; Santiago, Chile, 2013.
[29] INN. NCh 433.Of1996Mod2012. Seismic Design of Buildings; Instituto Nacional de Normalización: Santiago, Chile, 2012.
[30] FEMA 356. Prestandard and Commentary for the Seismic Rehabilitation of Buildings; Federal Emergency Management Agency: Washington, DC, USA, 2000.
[31] FEMA 440. Improvement of Nonlinear Static Seismic Analysis Procedures; Applied Technology Council: Redwood City, CA, USA, 2005.
[32] American Society of Civil Engineers - ASCE. Seismic Evaluation and Rehabilitation of Existing Buildings; ASCE/SEI 41-13; Reston, VA, USA, 2013.