16th World Conference on Earthquakes, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 514
Registration Code: S-I1461959765

COMPARISON OF THE SEISMIC BEHAVIOUR OF TWO INDUSTRIAL STEEL STRUCTURES DESIGNED IN ACCORDANCE WITH CHILEAN PRACTICES AND AISC REQUIREMENTS

C. Urzúa(1), R. Herrera(2)

(1) Structural Engineer, JRI Ingeniería, Chile, curzua@jri.cl
(2) Assistant Professor, University of Chile, riherrer@ing.uchile.cl

Abstract

In Chile, earthquake-resistant design of industrial structures and facilities is regulated by Chilean code NCh2369. Even when NCh2369 refers to seismic provisions of AISC as a complementary source of information, both codes present conceptual differences regarding the preferred source of energy dissipation. AISC seismic provisions enforce energy dissipation only through inelastic deformation of specific elements in the structure, which requires a widely distributed inelastic incursion in order to maximize energy dissipation. In an industrial context, providing structural detailing that allows a widely distributed plastification is not simple, given the usual irregularities of mass, stiffness, and geometry that structures exhibit. Considering this, NCh2369 allows dissipation additionally in the structure anchorage to the foundation. The advantage to this approach is that it limits in some way the seismic energy entering the structure, thereby reducing damage during strong earthquakes.

In order to study the impact on seismic behavior when either practice is used for design, in this paper two typical structures of mining industry are designed in parallel according to NCh2369 and AISC341 Seismic Provisions requirements. Seismic behavior is characterized through performance factors and methodology as indicated in FEMA P695. Furthermore, seismic energy distribution of the different dissipation mechanisms is studied for each practice. It is observed that utilization of ductile anchorage improves the structural behavior, reducing the seismic demand on resistant elements and increasing the overall ductility.

Keywords: Industrial structures, dynamic nonlinear analysis, plastic anchorage modeling.

1. Introduction

In Chile, seismic design of industrial buildings and facilities is regulated by Chilean code NCh2369.Of2003 [1]. Even when NCh2369 provides requirements for design and detailing of steel structures it also refers to AISC seismic provisions [2] as a complementary source of information. Both codes allow seismic energy dissipation in the structure, but the Chilean code additionally allows energy dissipation in the anchorage to the foundation, specifically in anchor bolts. The Chilean approach has exhibited an adequate behavior in the last eleven earthquakes with Richter-Kanamori magnitude greater than 7.5 [1].

In an industrial context, where structures are intended to support the operation of equipment or machinery, it is usual to find structural configurations with irregular distribution of mass and stiffness both in plan and elevation. This results in a complex seismic force distribution that makes steel detailing difficult for evenly distributed plastification. Additionally, in Chilean practice underlies the performance objective of operational continuity, therefore, it is not desirable to concentrate damage in the structure. Thus, utilization of ductile anchor bolts provides an alternative approach to overcome these design restraints.

In order to evaluate the effect of ductile anchor bolts in the structural response during strong earthquakes, two typical structures of mining industry are designed in parallel using the Chilean and AISC requirements. These structures present concentrically braced frames (*CBF*) as its main lateral resistant system since *CBF* is the most used configuration in Chilean industrial construction. The first structure (Fig. 1) is a 17.5m high building that is used for the operation of a vertical mill and a hydrocyclone battery. This building presents vertical concentric braces in both the longitudinal and transverse directions at all levels except the last, where moment frames are used in the transverse direction to allow transit of a monorail crane. The second structure (Fig. 2) is a 13m high building that transfers lithium ore for stocking. This structure presents both concentric X-braced frames and inverted-V braced frames (chevron). The upper level is configured based on moment frames since it has to allow eventual operation of a monorail crane. It may be observed that both structures present high irregularity of mass and stiffness in both plan and elevation.

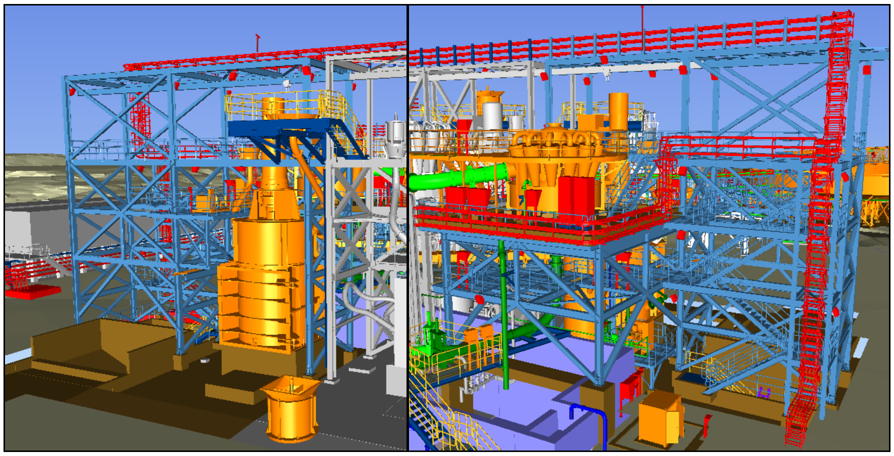

Fig. 1 – Vertical Mill Building. (Left) Front view. (Right) Rear view.

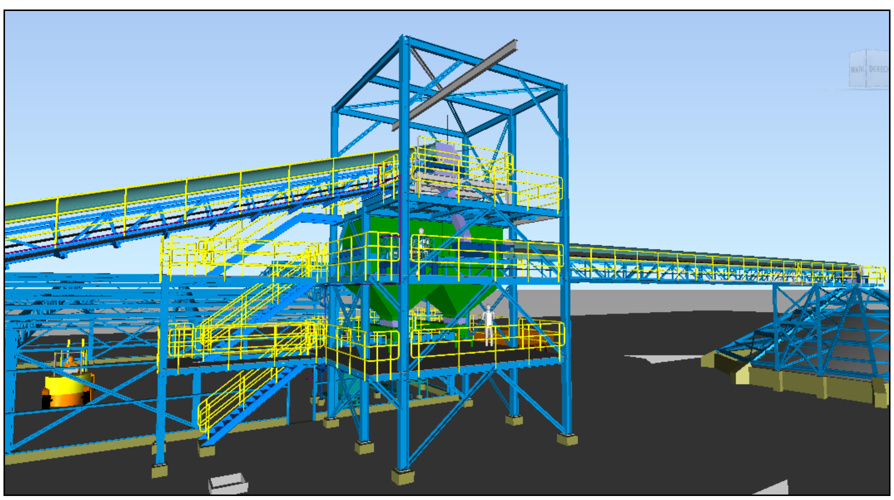

Fig. 2 – Transference tower.

Seismic behavior of structures is characterized by response factors such as system overstrength (Ω_0), period based ductility (μ), response modification coefficient (R_μ), seismic energy distribution, collapse margin ratio (*CMR*), and dynamic response modification coefficient (R_d). Response factors are determined using the methodology shown in Appendix F of FEMA P695 [3].

2. Methodology

In this work, the seismic behavior of structures is studied and evaluated by the application of the methodology detailed in Appendix F of FEMA P695 since behavior is studied for two predetermined structures. Methodology presented in FEMA P695 (referred to herein as the Methodology) is intended to provide a rational basis for determining global seismic performance factors, including the response modification coefficient (R), the system overstrength factor (Ω_0) and deflection amplification factor (C_d) that, when properly implemented in the seismic design process, will result in *equivalent safety against collapse in an earthquake, comparable to the inherent safety against collapse intended by current seismic codes, for buildings with different seismic-force-resisting systems* [3]. The process includes the following steps:

2.1 Required System Information

Structures studied are typical buildings of the Chilean mining industry. Each structure is designed in parallel under Chilean [1] and American [2] seismic provisions. Both practices present similar requirements in terms of materials, load cases, load combinations, and steel detailing. The main difference between both approaches comes from the fact that in Chilean practice seismic energy dissipation through anchorage plastification is both permitted and recommended. Dead and live loads are the same for either practice, and since structures are intended to operate in Chile, the design spectrum is obtained from NCh2369 for all cases considering a seismic zone 3 (maximum), soil type II (stiff soil), and a design damping ratio of 3%. The first structure, which is used for the operation of a vertical mill, is designed considering an importance factor (I) of 1.2, a response reduction factor (R) of 3, and a lateral force resisting system (*LFRS*) mainly based on concentrically braced frames (*CBF*). The second structure, a transference tower, is designed considering I=1.0, R=5, and a *LFRS* based on *CBFs* with both X and chevron brace configurations. It should be noted that in Chilean practice detailing requirements do not depend on the ductility level pursued. For designs in accordance with American practice, provisions for Special Concentric Braced Frames (*SCBF*) are followed. Studied structures roughly represent the boundaries of the practical overstrength range given to industrial structures in Chile. Envelope utilization ratios of structural members for LRFD design load combinations are shown in Fig. 3 for each case. Anchor bolts utilization ratio for Chilean practice is around 60% for both structures.

2.2 Quality Rating of Design Requirements

According to section F.4.3 of FEMA P695 design requirements of AISC341 classify as (A) “Superior”. Since requirements contained in NCh2369 are similar to those presented in AISC341, and considering that Chilean practice has exhibited a satisfactory performance in several strong earthquakes, design requirements of both practices will be classified as (A) “Superior”.

2.3 Test Data

In this work, nonlinear models of braces are calibrated using experimental data obtained by Black et al. [4] by the application of quasi-static cyclic displacement histories to individual braces with different shape, aspect ratio, and global slenderness. A nonlinear model of a plastic hinge forming in the beam of a chevron braced frame is calibrated using experimental data shown in the report PEER/ATC-72-1 [5]. It is noteworthy that data presented in this report are based on plastic hinges forming in beams of moment frames, but are used for model calibration due to the lack of available experimental data specifically oriented to plastic hinges forming in beams of chevron frames. A nonlinear model of anchor bolts is not calibrated by experimental data due to the lack of information on this topic; nevertheless, the anchor bolts model is simple enough to validate its behavior by simple inspection of its cyclic response.

2.4 Quality Rating of Test Data

Considering that cross-sectional shapes of individual tested braces are smaller than shapes typically used for vertical braces of industrial structures; test data of plastic hinges is based on plastification of beams of moment frames instead of chevron braced frames; the imposed displacement history may not be representative of the expected seismic loading; and test data for anchor bolt calibration was not available, a quality rating of (B) “Good” is assigned to the test data used in this work.

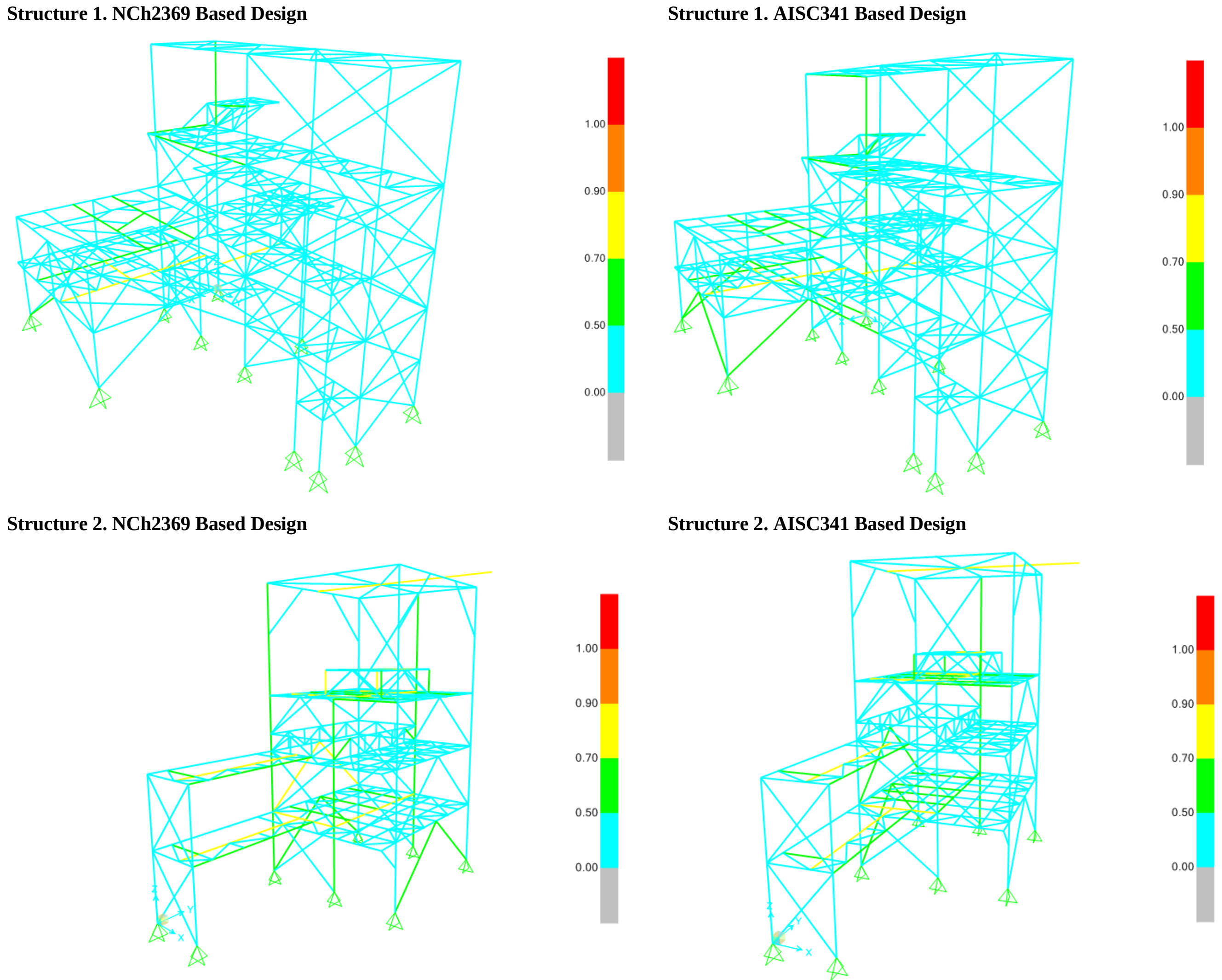

Fig. 3 – Envelope utilization ratios of structural members for LRFD design load combinations.

2.5 Archetype Configurations

Since two specific structures are being analyzed in this work, a definition of an archetype space covering the possible range of geometric and loading configurations of the lateral systems studied is not needed, thereby limiting the probabilistic collapse evaluation to those structures only.

2.6 Nonlinear Model Development

2.6.1 Modeling Approach

In this work, tridimensional models concentrate nonlinear behavior in vertical braces, plastic hinges, and anchor bolts. The eventual inelastic incursion of other members such as columns, floor beams, and braces is evaluated as non-simulated collapse modes. Models are implemented in SAP2000 Ultimate analysis program through nonlinear *Link* elements. Since nonlinear analysis of tridimensional structures by direct integration of full equations of motion is computationally very demanding, the Fast Nonlinear Analysis (*FNA*), as implemented in SAP2000, is utilized. *FNA* is extremely efficient and less dependent on the chosen time step compared to the direct integration method. Rigid offsets were assumed at the beam-column connections to represent the physical size and stiffening effect of the gusset plates.

2.6.2 Nonlinear Model of Braces

Vertical braces are modeled in SAP2000 using nonlinear *Link* elements of *Multilinear Plastic* type. Fig. 4 shows the parametric backbone curve utilized for brace elements. The tension side of the curve is defined according to FEMA356 [6] accounting for yielding, strength loss, and fracture. In compression, strength is assumed to increase elastically until a buckling load defined according to AISC360-05 provisions [7]. For all purposes, steel yield stress is taken as the expected yield stress. After buckling, the backbone curve decays exponentially until reaching a residual strength equal to 20% of the buckling load. Effective length factor (K) is taken as 0.5 for braces in an X configuration and 0.8 for individual braces considering the stiffness of the gusset plates [8]. Hysteretic degradation of strength and stiffness of braces during loading cycles is incorporated using a *Pivot* model as implemented in SAP2000. The response of the brace model is compared against experimental data for shapes used in the studied structures (HSS and TL as representative of XL) as shown in Fig. 5.

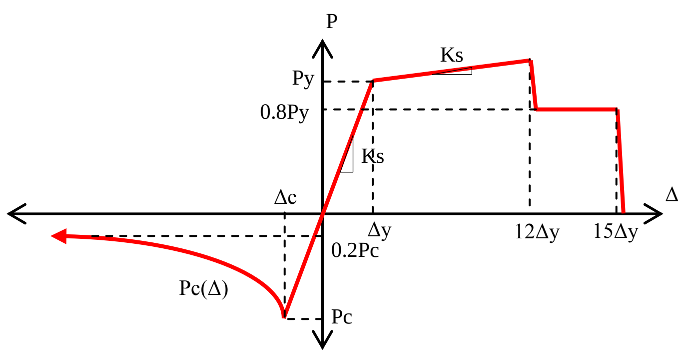

Fig. 4 – Parametric backbone curve for vertical braces.

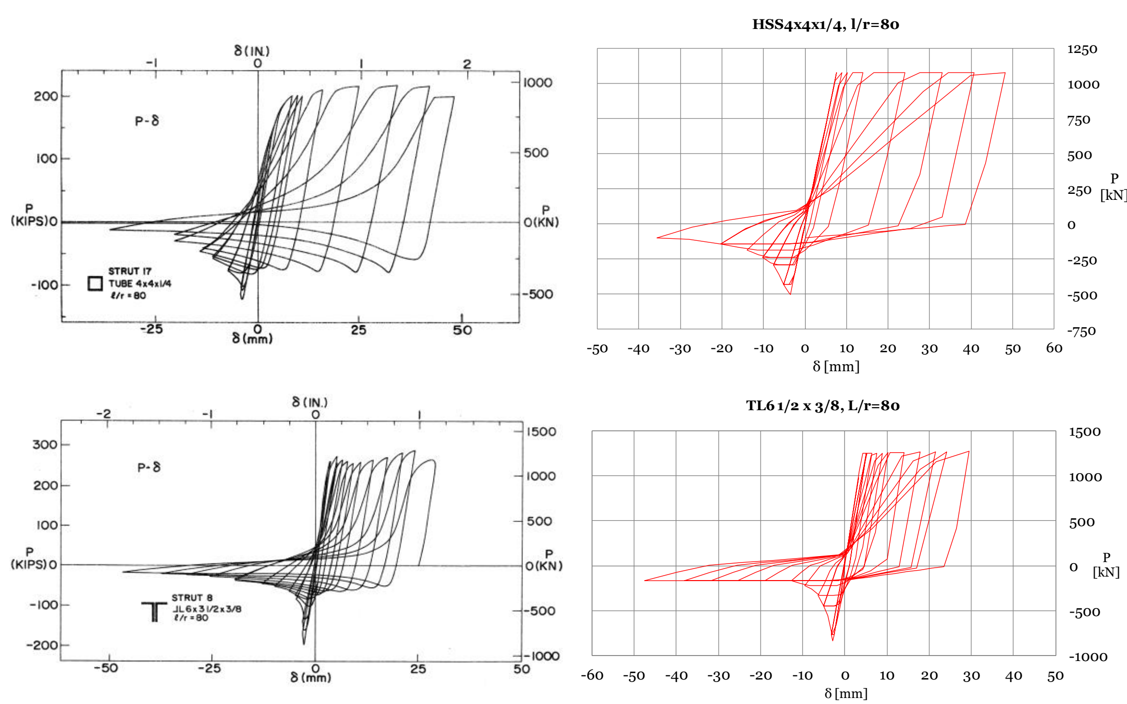

Fig. 5 – Experimental [4] (left) vs. analytical (right) response of HSS4x4x1/4 and TL1/2x3/8

2.6.3 Nonlinear Model of Plastic Hinges

Plastic hinges of chevron braces are modeled in SAP2000 using nonlinear *Link* elements of *Multilinear Plastic* type. The backbone curve is shown in Fig. 6 and it is based on the model proposed by Lignos and Krawinkler in [5] modifying the strength loss segment. For all purposes, steel yield stress is taken as the expected yield stress. Hysteretic degradation of strength and stiffness of hinges during loading cycles is incorporated using a *Kinematic* model as implemented in SAP2000. The hinge model is calibrated against experimental data contained in [5] as shown in Fig. 7.

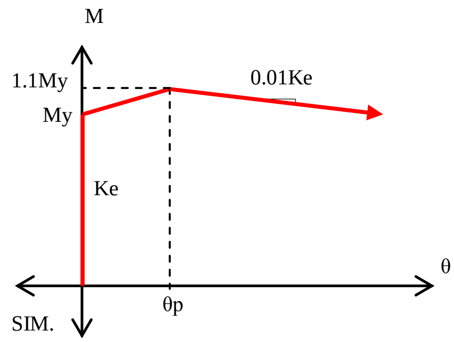

Fig. 6 – Parametric backbone curve for plastic hinges of beams of chevron braces.

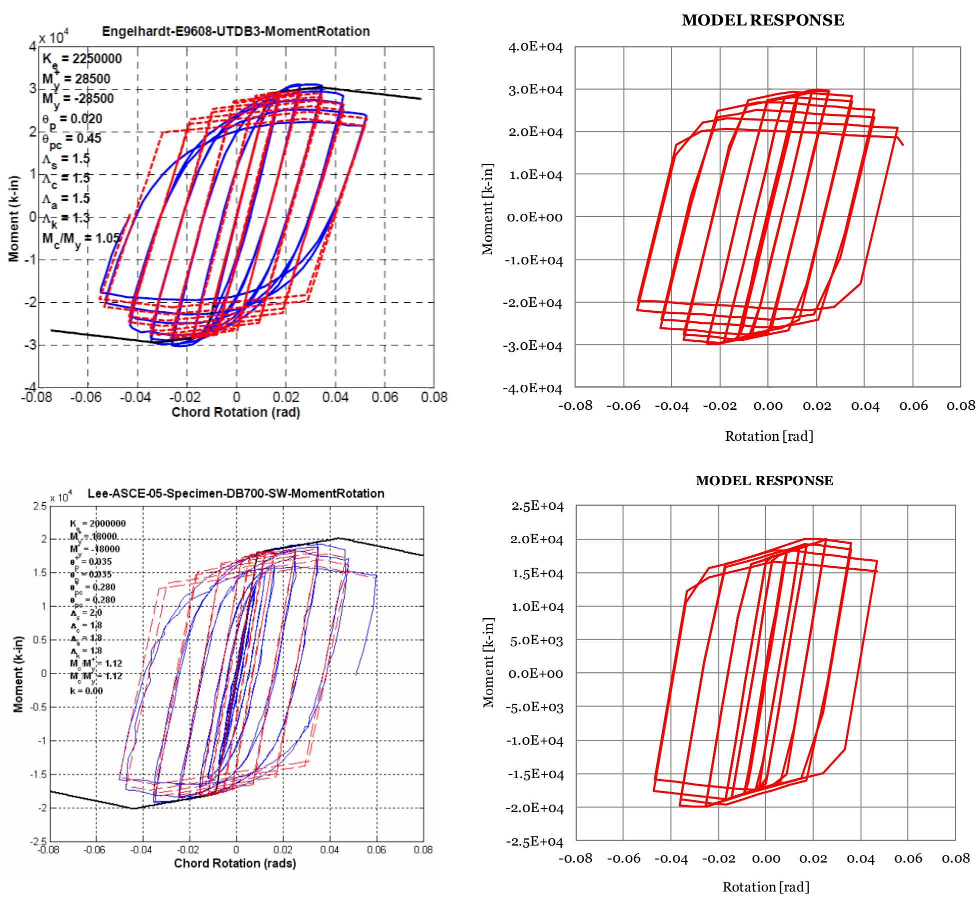

Fig. 7 – Experimental [5] (left) vs. analytical (right) response of plastic hinges.

2.6.4 Nonlinear Model of Anchor Bolts

In Chile, column anchorage to foundation transmitting shear and tension typically consists of anchor bolts and a shear lug. Shear lugs and the embedment in the pedestal, when properly designed, have not presented premature failure in severe past earthquakes, unlike anchor bolts where inelastic incursion is expected. The anchorage model includes lateral restraints along with two vertical nonlinear elements arranged in parallel. The first element, of the *Multi-linear Plastic* type, represents the anchor bolts working only in tension. The backbone curve and the *Kinematic* hysteretic rule for this element are shown in Fig. 8. Capacity and stiffness of anchor bolts are based on expected yield stress and nominal area. The maximum elongation allowed for anchor bolts is 0.2L based on minimum requirements for anchor bolts material according to NCh2369. The cyclic response of the model is capable to reproduce the progressive elongation that bolts experience. The second element, of the *Gap* type, represents the pedestal working elastically only in compression.

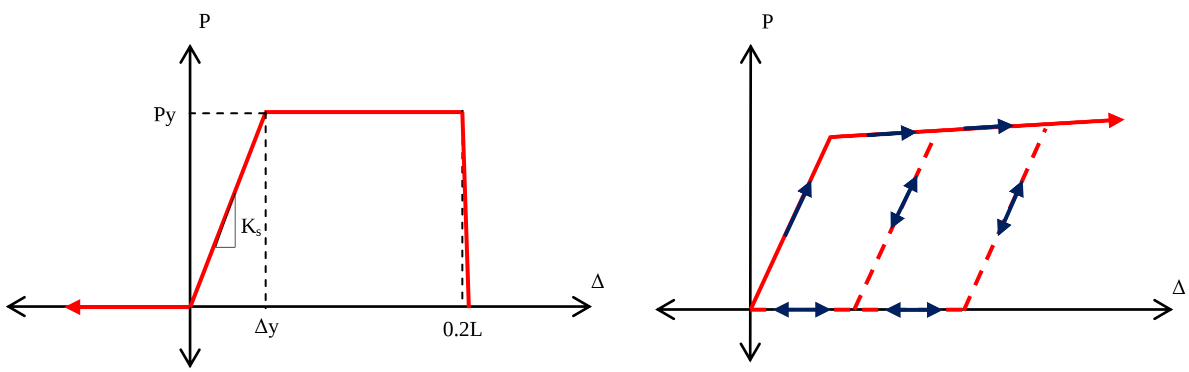

Fig. 8 – (Left) Backbone curve for anchor bolts model. (Right) Hysteretic response of anchor bolts model.

2.6.5 Quality Rating of Analytical Models

Analytical models of braces are capable of reasonably reproducing the experimental response when local buckling is not the governing failure mode, which is prevented by design criteria. Likewise, plastic hinge model matches adequately the experimental data used for calibration. Even when the anchor bolt model has not been calibrated, nonlinear behavior of the model is associated mainly with the monotonic plastic deformation of the bolts, where no geometric instability of components occurs. This simple model produces a cyclic response that is consistent with the expected behavior for the column base. Taking these factors into account, a quality rating of (B) “Good” is assigned to analytical models in this work.

2.7 Nonlinear Analysis

2.7.1 Static Analysis

Pushover analyses are used to evaluate system overstrength, period-based ductility, and response modification coefficient. *FNA* method is used to improve the convergence of solutions. Load application is slow and a high damping ratio (0.999) is utilized in order to avoid the triggering of dynamic effects. Gravity loads for load combination *1.05D+0.25L* are applied first; then, lateral loads at each level are applied proportionally to the level mass (m_x) and the fundamental modal shape for the direction of analysis ($\phi n,x$). The overstrength factor (Ω) is calculated as the ratio between the maximum shear force obtained from pushover analysis (V_{max}) and the design shear force (V_d):

$$\Omega = \frac{V_{\max}}{V_d} \tag{1}$$

Period-based ductility (μ) is obtained by dividing the ultimate roof drift by the effective yield drift obtained from static pushover analysis. Since studied structures present moment frames in the upper level, the roof level is considered as the level below.

$$\mu_T = \delta_u / \delta_{y,eff} \tag{2}$$

Since studied structures are rigid, the response modification coefficient (R_μ) is defined as:

$$R_\mu = \sqrt{2\mu - 1} \tag{3}$$

Pushover curves for the studied structures are shown in Fig. 9. Designs based on Chilean practice exhibit a higher ductility capacity product of anchor bolt plastification. Designs based on AISC341 seismic provisions present a more fragile behavior due to the formation of a soft story collapse mechanism, with no ability of deformation redistribution provided by ductile anchor bolts. In the second structure, the Chilean design shows low overstrength due to the premature yielding and fracture of anchor bolts, which can be indicative of the need for special requirements for ductile anchor bolt design.

In Table 1 are shown the performance factors obtained from pushover analyses. For the first structure, Chilean design presents similar overstrength (Ω) and response reduction capacity (R_μ) to that in the AISC design; nevertheless, period-based ductility is 50% higher in the Chilean design. For the second structure, the Chilean design presents 50% less overstrength due to the premature yielding of anchor bolts but exhibits twice the ductility and almost 60% more response reduction capacity than AISC design.

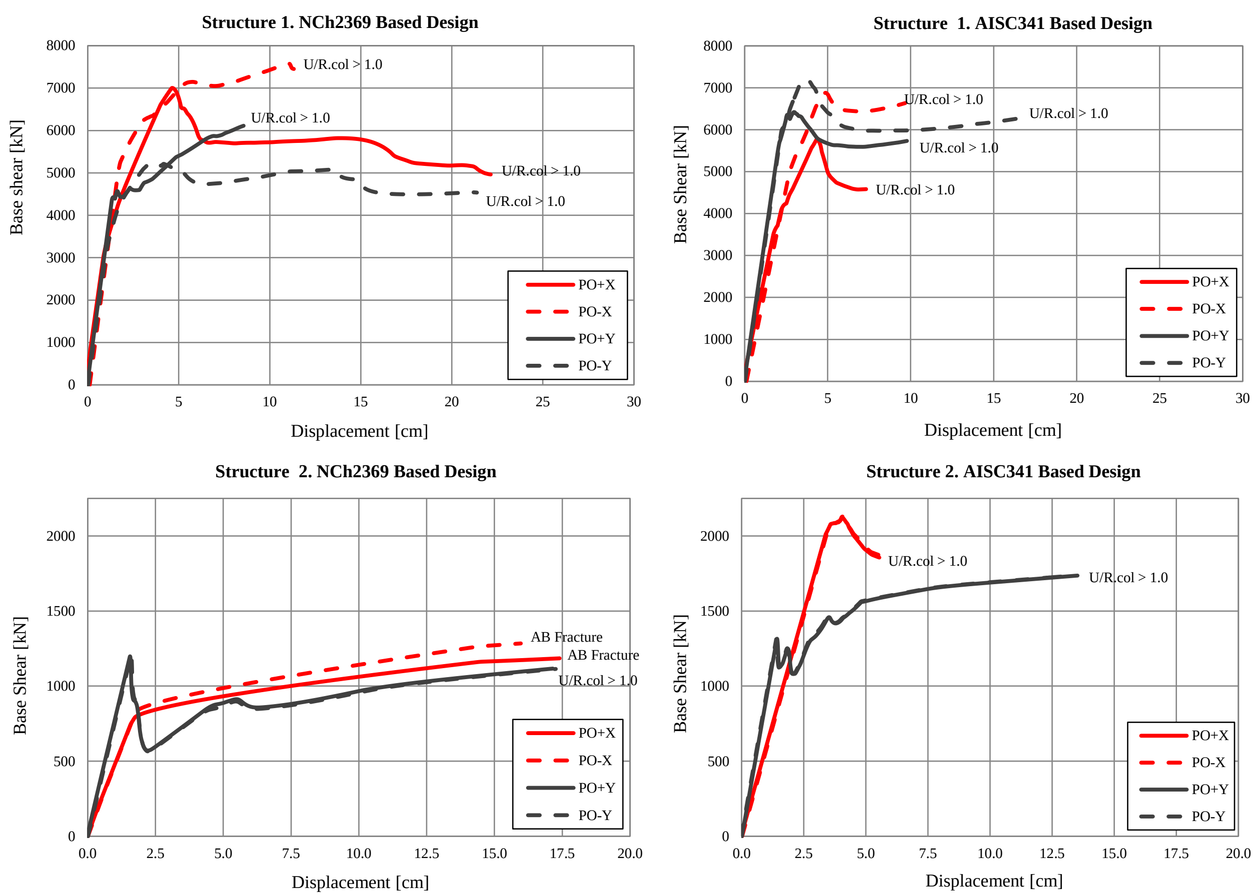

Fig. 9 – Pushover curves for Structure 1 (Up) and Structure 2 (Down). Left plots correspond to designs based on Chilean Practices (NCh2369), and right plots to designs based on AISC341 Seismic Provisions.

Table 1 – Performance factors from pushover analyses.

Load Case	V_0=1.4V [kN]	C_0	Vmax [kN]	W [kN]	Tn [s]	δy,eff [cm]	δu [cm]	μ_T	Ω	Rμ
Structure 1/NCh2369 Based Design (I=1.2, Seismic Zone 3, Soil Type II, Rdsgn=3, CBF)										
PO+X	710	1.0	7003	2538	0.181	2.3	22.1	9.7	9.9	4.3
PO-X	710	1.0	7579	2538	0.181	2.5	11.3	4.6	10.7	2.9
PO+Y	958	1.0	6112	2538	0.169	1.7	8.6	5.2	6.4	3.1
PO-Y	958	1.0	5234	2538	0.169	1.4	21.4	15.1	5.5	5.4
Mean	*834*	*1.0*	*6482*	*2538*	*0.175*	*2.0*	*15.9*	*8.6*	*8.1*	*3.9*
Structure 1/AISC341 Based Design (I=1.2, Seismic Zone 3, Soil Type II, Rdsgn=3, SCBF)										
PO+X	690	1.3	5773	2384	0.160	1.9	7.3	3.8	8.4	2.6
PO-X	690	1.3	6880	2384	0.160	2.3	9.7	4.2	10.0	2.7
PO+Y	906	0.8	6421	2384	0.172	1.7	9.8	5.9	7.1	3.3
PO-Y	906	0.8	7167	2384	0.172	1.9	16.7	9.0	7.9	4.1
Mean	*798*	*1.1*	*6560*	*2384*	*0.166*	*1.9*	*10.9*	*5.7*	*8.3*	*3.2*
Structure 2/NCh2369 Based Design (I=1.0, Seismic Zone 3, Soil Type II, Rdsgn=5, CBF)										
PO+X	269	1.3	1185	1046	0.228	1.9	18.0	9.5	4.4	4.2
PO-X	269	1.3	1284	1046	0.228	2.1	16.0	7.8	4.8	3.8
PO+Y	303	1.4	1198	1046	0.198	1.5	17.2	11.3	4.0	4.6
PO-Y	303	1.4	1205	1046	0.198	1.5	17.3	11.3	4.0	4.6
Mean	*286*	*1.3*	*1218*	*1046*	*0.213*	*1.7*	*17.1*	*10.0*	*4.3*	*4.3*
Structure 2/AISC341 Based Design (I=1.0, Seismic Zone 3, Soil Type II, Rdsgn=5, SCBF)										
PO+X	287	1.3	2122	1046	0.216	3.0	5.5	1.8	7.4	1.6
PO-X	287	1.3	2130	1046	0.216	3.1	5.5	1.8	7.4	1.6
PO+Y	321	1.4	1751	1046	0.176	1.8	14.7	8.3	5.5	4.0
PO-Y	321	1.4	1753	1046	0.176	1.8	14.6	8.3	5.5	3.9
Mean	*304*	*1.3*	*1939*	*1046*	*0.196*	*2.4*	*10.1*	*5.0*	*6.4*	*2.8*

2.7.2 Dynamic Analyses

Time history analyses are used to evaluate the collapse margin ratio, dissipated energy distribution, and dynamic response reduction coefficient. *FNA* method is used for analyses. Gravity loads are applied first slowly with a high damping ratio (0.999) and then ground motions are applied considering a damping ratio of 1% of critical damping only in modes with a major modal mass contribution. This low damping ratio is used because most of the dissipation mechanisms are explicitly modeled. All three ground motion components are applied to the model since vertical load is relevant for anchor bolts and chevron braced frames.

The collapse margin ratio (*CMR*) is the ratio of the median 5%-damped spectral acceleration of the collapse level ground motions (S_{CT}) to the 5%-damped spectral acceleration of the MCE ground motions (S_{MT}), at the fundamental period of the seismic-force-resisting system:

$$CMR = S_{CT} / S_{MT} \tag{4}$$

Median collapse intensity (S_{CT}) is obtained by scaling all the records of the record set shown in Table 2 until just over one-half of the scaled ground motions records cause collapse. The *MCE* intensity (S_{MT}) is taken from Chilean code NCh2745 [9] where it is defined as the seismicity having a 10% probability of exceedance in 100 years. Since all records of the record set present thrust focal mechanisms there is no need for spectral shape correction. According to FEMA P695, S_{CT} resulting from three-dimensional analyses is on average about 20% less than S_{CT} resulting from two-dimensional analyses; therefore, S_{CT} is amplified by 1.2 to correct this effect.

The Methodology requires a maximum probability of collapse of 10% when the evaluation is being made for individual structures. The acceptable value of the *CMR* ($CMR_{10\%}$) is a function of the total system collapse uncertainty (β_{TOT}) that for ratings of design criteria, test data, and models quality determined before can be estimated as $\beta_{TOT}=0.50$, resulting in a $CMR_{10\%}=1.90$.

Table 2 – Record set for nonlinear dynamic analysis.

Epicenter	Date	Magnitude	Station	Dur. [s]	Comp.	PGA [g]	Arias Intensity [m/s]	Comp.	PGA [g]	Arias Intensity [m/s]	Comp.	PGA [g]	Arias Intensity [m/s]
Valparaíso	03-03-1985	7.8 Ms	Melipilla	79.3	EW	0.53	9.68	NS	0.69	8.95	V	0.25	1.87
			San Isidro	100.0	L	0.72	19.90	T	0.71	20.77	V	0.40	4.91
Punitaqui	14-10-1997	6.8 Ms	Illapel	80.0	L	0.27	2.11	T	0.35	2.65	V	0.18	0.64
South Perú	23-06-2001	8.4 Mw	Cementerio	63.4	L	0.27	1.40	T	0.31	1.66	V	0.18	0.64
			Costanera	76.2	L	0.34	1.39	T	0.27	1.26	V	0.08	0.21
Tocopilla	14-11-2007	7.7 Mw	Mejillones	170.0	EW	0.41	2.58	NS	0.42	2.98	V	0.34	2.63
			Tocopilla	70.9	L	0.50	7.27	T	0.59	8.65	V	0.57	5.24
Maule	27-02-2010	8.8 Mw	Angol	83.0	EW	0.70	17.46	NS	0.93	19.86	V	0.29	4.24
			Constitución	143.3	L	0.54	19.65	T	0.63	26.00	V	0.35	3.74
Iquique	01-04-2014	8.2 Mw	Chusmiza	222.0	EW	0.24	1.61	NS	0.36	2.43	V	0.16	0.90

Table 3 summarizes *CMR* calculation from incremental nonlinear dynamic analysis of the studied structures. For structure 1, both Chilean and American designs show similar safety against collapse, exceeding the minimum acceptable preset value. For structure 2, neither design accomplishes the minimum safety against collapse, though the Chilean design is closer to compliance with the preset limit.

Table 3 – Summary of collapse results for studied structures.

	Structure 1				**Structure 2**			
Seismic Provisions	**NCh2369**		**AISC341**		**NCh2369**		**AISC341**	
	Dir. X	Dir. Y	Dir. X	Dir. Y	Dir. X	Dir. Y	Dir. X	Dir. Y
Scaling factor, SF	2.23	2.23	2.20	2.20	1.90	1.90	1.20	1.20
Median collapse intensity, Sa50% [g]	1.31	1.12	1.02	1.19	1.17	1.16	1.16	1.19
S_{CT} = 1.2 • SF • Sa50%, [g]	3.52	3.01	2.70	3.14	2.67	2.65	1.66	1.71
MCE intensity, S_{MT} [g]	1.56	1.43	1.37	1.50	1.68	1.62	1.68	1.50
CMR = S_{CT}/S_{MT}	2.26	2.10	1.98	2.10	1.58	1.63	0.99	1.15
$CMR/CMR_{10\%}$	1.19	1.11	1.04	1.11	0.83	0.86	0.52	0.60
Mean $CMR/CMR_{10\%}$	*1.15*		*1.07*		*0.85*		*0.56*	

The dynamic modification factor (R_d) is the ratio of the maximum base shear obtained from linear dynamic analysis to the maximum base shear obtained from nonlinear dynamic analysis. R_d can be directly compared with R_μ from pushover analysis since both factors implicitly consider unit overstrength:

$$R_d = V_{Lin} / V_{NL} \tag{5}$$

Dissipated energy distribution and dynamic response modification factor are evaluated for 6 ground motions at imminent collapse level. Table 4 shows the summary of R_d calculation for the studied structures. R_d values follow a similar tendency than R_μ, being R_μ values around 20% higher. From both R_d and R_μ it is observed that for structure 1 Chilean and American designs present a similar response reduction capacity, while for structure 2 Chilean design has around 70% more reduction capacity than American design.

Table 4 – Summary of dynamic response modification factors for studied structures.

Seismic Provisions	Structure 1				Structure 2			
	NCh2369		AISC341		NCh2369		AISC341	
	Dir. X	Dir. Y	Dir. X	Dir. Y	Dir. X	Dir. Y	Dir. X	Dir. Y
Rd = Vlin/Vnl (Mean)	2.6	3.4	2.9	2.8	3.4	4.1	1.6	2.6

Dissipated energy distribution is shown in Fig. 10. For structure 1, Chilean design shows an early participation of anchor bolts as the main energy dissipation system. As input seismic energy increases braces start to become more relevant in energy dissipation. American design shows that energy dissipation in braces slightly increases as input energy grows up. For structure 2, Chilean design shows a minor and constant participation of anchor bolts in energy dissipation. American design shows a similar tendency than for structure 1. The lower dissipation contribution of anchor bolts can be explained by its smaller available strength.

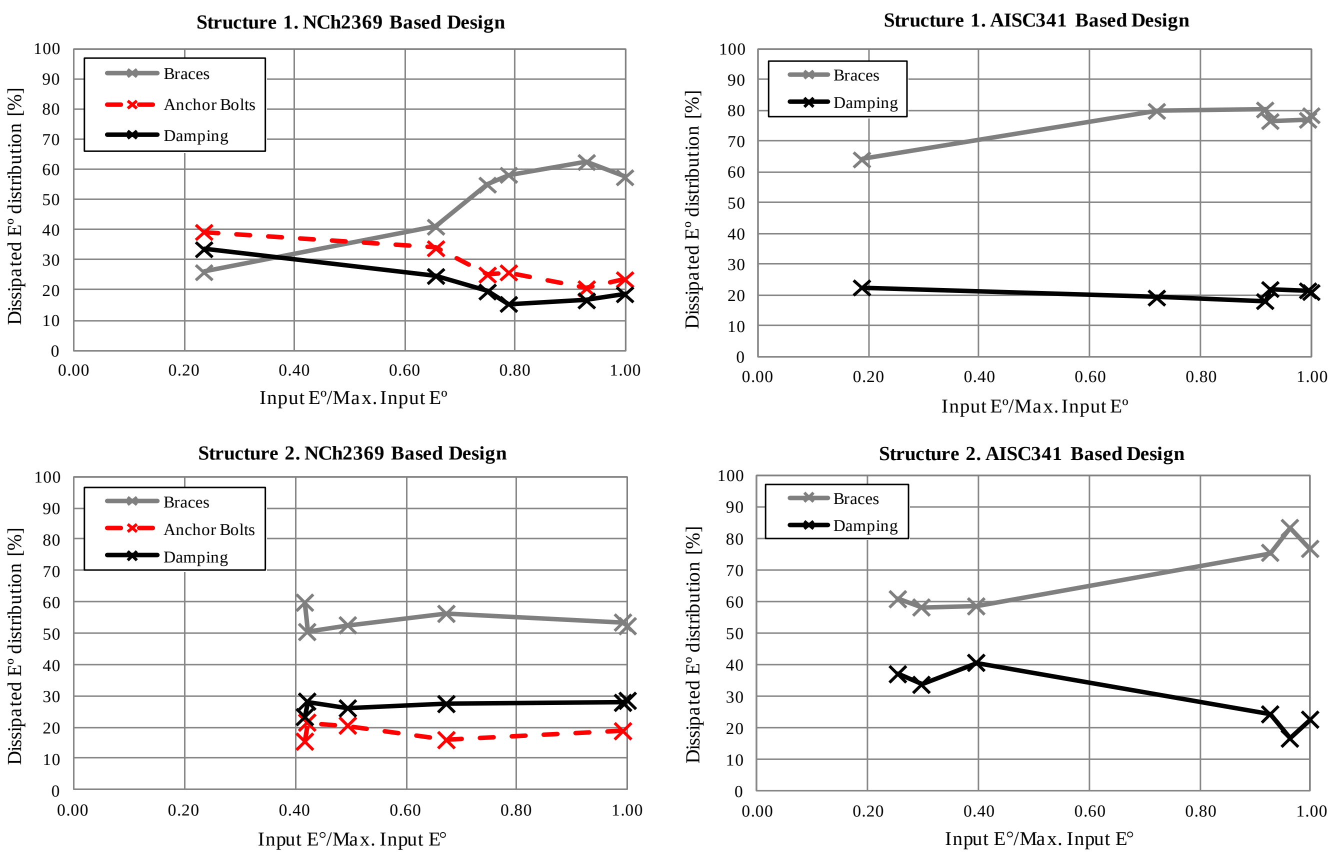

Fig. 10 – Dissipated energy distribution vs. input seismic energy for 6 ground motions at collapse level.

3. Conclusions

Seismic behavior of two typical structures of the Chilean mining industry was studied. Each structure was designed in parallel under Chilean and American seismic provisions. The main difference between both practices comes from the use of ductile anchor bolts by the Chilean practice. It is observed for both structures that the use of anchor bolts allow a more ductile global behavior, increasing the response reduction capacity of the structures. Considering the studied structures as representative of the practical boundaries of the usual overstrength range given to industrial buildings, it is observed that anchor bolts, *when properly sized*, act as an early seismic energy dissipation mechanism that limits the energy entering the structure, thereby reducing damage during strong earthquakes.

4. References

[1] INN (2003): Diseño Sísmico de Estructuras e Instalaciones Industriales. *NCh2369.Of2003*, Instituto Nacional de Normalización, Santiago, Chile.

[2] AISC (2005): Seismic Provisions for Structural Steel Buildings, *ANSI/AISC 341-05*. American Institute for Steel Construction, Chicago, Illinois, USA.

[3] FEMA (2009): Quantification of Building Seismic Performance Factors. *FEMA P695*. Prepared by Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C., USA.

[4] Black, G. R., Wenger, B. A., and Popov, E. P. (1980): Inelastic Buckling of Steel Struts under Cyclic Load Reversals, *UCB/EERC-80/40*, Earthquake Engineering Research Center, Berkeley, USA.

[5] ATC (2010): Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings. *PEER/ATC-72-1*, Applied Technology Council, California, Redwood City, USA.

[6] FEMA (2000): Prestandard and Commentary for the Seismic Rehabilitation of Buildings. *FEMA356*, prepared by American Society of Civil Engineers for the Federal Emergency Management Agency, Washington, D.C., USA.

[7] AISC (2005): "Specification for Structural Steel Buildings", AISC/ANSI 360-05. American Institute of Steel Construction, Chicago, Illinois, USA.

[8] Uriz, P., Mahin, S. A. (2008): Towards Earthquake-Resistant Design of Concentrically Braced Steel-Frame Structures. PEER Report 2008/08, Pacific Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley, USA.

[9] INN (2003): Análisis y Diseño de Edificios con Aislación Sísmica. *NCh2745-2003*, Instituto Nacional de Normalización, Santiago, Chile.