SEISMIC PERFORMANCE OF BUCKLING RESTRAINED BRACED FRAMES IN A CHILEAN BUILDING

M. Canales(1), R. Herrera(2) , L. Fahnestock(3) , S. Zaruma(4) , R. Tremblay(5)

(1) Graduate Researcher, Dept. of Civil Engineering, University of Chile, Santiago, Chile, marisol.canales@ing.uchile.cl
(2) Assistant Professor, Dept. of Civil Engineering, University of Chile, Santiago, Chile, riherrer@ing.uchile.cl
(3) Associate Professor, Dept. of Civil and Env. Eng., University of Illinois at Urbana-Champaign, USA, fhnstck@illinois.edu
(4) Graduate Student, Dept. of Civil and Env. Eng., University of Illinois at Urbana-Champaign, USA, zarumao2@illinois.edu
(5) Professor, Dept. of Civil, Geological and Mining Eng., Polytechnique Montreal, Canada, robert.tremblay@polymtl.ca

Abstract

Buckling restrained braced frames (BRBFs) are slowly being introduced in Chile, considering their advantages over conventional concentrically braced frames (CBFs). However, they are not included in Chilean seismic design codes yet. The seismic performance of a buckling restrained braced frame is studied for a 9-story office prototype building structure designed under Chilean standards. The structure is located in the central coast zone of the country, a seismic region dominated by large subduction earthquakes. The resulting building is modeled considering the relevant nonlinearities of the problem and the model is subjected to nonlinear static pushover and dynamic time-history analyses, using several ground motion records from the last ten years. The performance of the prototype is studied in terms of base shear, story drift, importance of second order effects, and likelihood of collapse. An analogous procedure is followed for a buckling restrained braced frame for the same prototype building located in the United States, in an area with similar seismic conditions. This structure is designed under provisions in the U.S. and it is subjected to static pushover analysis as a simple point of comparison. Pushover results show a high ductility response of the structure, but with limited overstrength. Time-history analysis results for the Chilean prototype indicate that with the current seismic code, the structure exhibits an adequate performance and it has significant reserve capacity.

Keywords: Buckling Restrained Brace; Seismic Performance; Nonlinear Analysis.

1. Introduction

Steel braced frames are one of the most efficient steel structural systems to resist lateral forces. The principal advantage of this type of structures is the stiffness contribution from the braces, allowing a better control of floor displacements. Buckling Restrained Braces (BRBs) have a great advantage over conventional steel braces because they are able to yield both in tension and compression without buckling [1-4]. To achieve this behavior, buckling in compression is inhibited assembly of several key components. A BRB consists of a steel core coated with a low-frictional material and encased by a hollow steel structural section filled with a specialized mortar or another confining material. BRBs offer robust cyclic performance and significant cost savings, compared to conventional bracing systems.

In Chile, due to the high seismic activity, braced frames are a solution often used in steel buildings, so it is interesting to study the seismic performance of Buckling Restrained Braced Frames (BRBFs) designed under the Chilean building seismic design code, particularly since this kind of buildings have not been incorporated in the codes yet. There are no studies of BRBF performance for a Chilean design with the actual seismic building code, which was modified after the 8.8 (Mw) earthquake of February 27, 2010, struck the central part of Chile. Therefore, it is necessary to study BRBF performance to incorporate it in the Chilean code, for use in new construction or rehabilitation projects. In this paper, a BRBF designed according the Chilean code is evaluated with static and dynamic analyses, and a BRBF designed according to the United States code provides a reference comparison.

2. Background

Tremblay et al. [5] describes the testing of two BRBs and an analytical study carried out to evaluate the seismic performance of BRBFs. A 3-story braced structure with BRBs, designed according to the Canadian building code, was evaluated and compared to the same building structure with conventional steel braces. The influence of specifying different brace core lengths on the seismic performance of BRBFs was also examined through nonlinear dynamic analysis of a 3-story building model located in Vancouver, B.C., along the Pacific west coast of Canada. The structure was divided in a storage area and a retail area, separated by a construction joint and behaving individually. To model the structures, the BRBs were considered as bar elements with an equivalent cross-sectional area. The design of the 3-story building showed that story drifts can be reduced by specifying BRBs with shorter core dimensions, but this results in higher strain demands imposed on the brace cores. A significant advantage of using BRBs is the reduction in the forces imposed on foundations and surrounding structural elements, compared to conventional concentrically braced frames structures. Nonlinear dynamic analyses indicated that inelastic demands tend to concentrate at the bottom floor, resulting in core strain demands exceeding the design values, especially when short brace cores are specified. The nonlinear analyses also demonstrated that conventional CBF structures can experience smaller lateral deformations compared to BRBFs, but similar drift amplification at the lower floor was observed and much larger forces were imposed on the surrounding structural elements. These forces can be adequately predicted for BRBFs using appropriate capacity design rules.

Fermandois [6] designed, under the previous Chilean building seismic design code, a set of five office buildings of normal importance of 4, 8, 12, 16 and 24 stories, with the same floor plan and structured with BRBFs and with a two-story X braced configuration. The building was located in seismic zone 3 in soil type II. The design results showed that the base shear for the 4 story building was controlled by maximum base shear, the 8 and 12 story buildings were in the intermediate range, and the 16 and 24 story buildings were controlled by the minimum base shear. Fermandois defined four structural failure criteria: maximum absolute ductility of BRBs, maximum cumulative ductility of BRBs, maximum rotation of connections nodes, and formation of a possible collapse mechanism. To evaluate the lateral displacement demands and strength of elements, pushovers with different lateral load profiles and incremental dynamic analysis (IDA) were applied to the models of the prototypes. These analysis results show that the ultimate limit state of the structure is always related to the limit state of BRB elements, reaching high ductility capacities and with an adequate control of the lateral displacements. The frame yielding progresses from the braces at the upper levels to the lower floors. Failures on beam-column connections appeared eventually beyond the BRB failure point. The capacity obtained from IDA analysis is greater than that from pushover analyses when the number of floors increase; however, the results are in the same order. The inelastic deformation demands on the structures were obtained through time-history analyses, showing residual drift at the roof. The 4-story building exhibited a response where the fundamental period dominates, while for the rest of the buildings, higher vibration modes contribute.

3. Prototype Design

3.1 Geometry and gravity loading

The prototype braced frame building corresponds to a 9-story model building studied in a comparison of seismic design provisions for BRBFs between United States, Canada, Chile, and New Zealand [7]. The building has four buckling restrained braced frames in a chevron bracing configuration in the E-W perimeter for the Chilean design. For the US design, two BRBFs are used in the E-W direction. The structure plan view, the braced frame elevation and design gravity loads are shown in Fig. 1. The building of 45.72 m x 45.72 m floor plan, is an office building of the normal importance category. As shown, the building has a single-level basement and taller first story height, a common feature in office buildings.

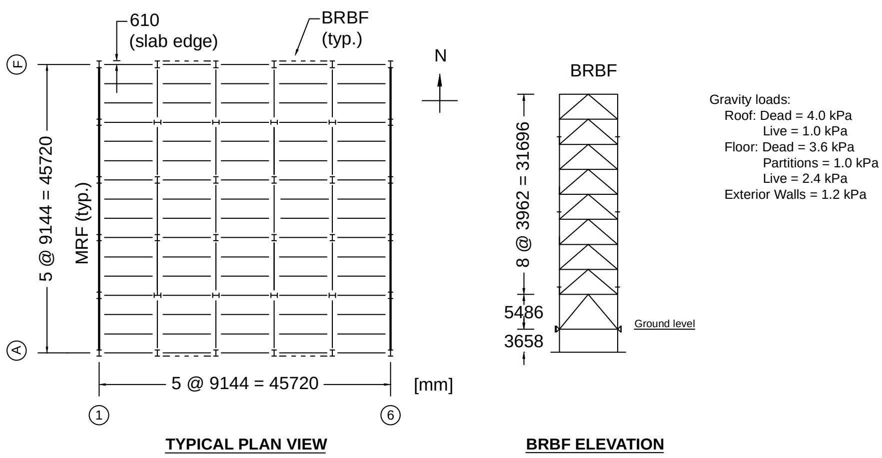


Fig. 1 – Prototype Structure

3.2 Building location and seismic data

The structure designed under Chilean standards is located in Valparaiso, Region V. The seismicity in Valparaiso is dominated by large subduction earthquakes that occur frequently at the boundary of the Nazca and South American tectonic plates. The building is assumed to be constructed on firm soil conditions, corresponding to a site class C according to NCh433 [8]. In Chile, the seismic input for design is essentially characterized by the effective peak ground acceleration at the site. Valparaiso is located in seismic zone 3, where this value is equal to 0.40 g.

The structure designed according to US provisions is located in Seatle, WA. Sites in Seattle are exposed to crustal and sub-crustal earthquakes as well as seismic ground motions originating from the Cascadia subduction zone. Firm soil conditions are also assumed for this case, with a site class C and corresponding spectral values for the seismic input, S_{MS} = 1.365g and S_{M1} = 0.686g.

3.2 Seismic Design

The BRB yield stress is F_{ysc} = 290 MPa and the strength adjustment factors R_y, ω, and β are taken equal to 1.0, 1.4, and 1.1, respectively. In the analyses, the bracing members are assumed to have an equivalent cross-sectional area over the brace workpoint length equal to 1.5 times the core yielding cross-section area. Beams and columns are assumed to be fabricated from ASTM A992 I-shaped members with a steel yield strength of 345 MPa. Beams are non-composite and the frames are analyzed and designed assuming that the beam-to-column connections are pinned. The beams are assumed to be vertically braced by the BRB members at mid-length and laterally braced at quarter points and mid-length.

For the seismic design and detailing of the building frames, the 2010 edition of the AISC Seismic Provisions, AISC 341-10 [9], AISC Specification for Structural Steel Buildings, AISC 360-10 [10] and Chilean code for seismic design NCh433 [8] are used. The final design in Chile is controlled by the minimum base shear and a strict drift limit of 0.2 percent of the story height, h_s, specified by NCh433. The effective value of the force modification factor (R_{eff}) used for the design is 3.98. This value is the actual response modification factor that results after consideration of the minimum base shear requirement. The final member sizes for the prototype design obtained using a response spectrum analysis, are listed in Table 1. The model for design was implemented in MIDAS Gen software, obtaining a fundamental period for this system of T=1.55 s.

For the US scenario, the frame was designed including stability provisions from AISC 360-10 [10] and after sizing the members to satisfy strength requirements, the story drifts and stability coefficients were within the applicable limits, hence these factors did not control the design. The final member sizes, listed in Table 1, were also obtained using a response spectrum analysis. A model in SAP2000 was generated and the obtained fundamental period of the system is T=2.64 s.

Table 1 – Prototype frame member sizes.

	Chilean Design			**U. S. Design**		
Story	**BRB (mm^2)**	**Column**	**Beam**	**BRB (mm^2)**	**Column**	**Beam**
9	2520	W 200 x 59	W 200 x 71	1844	W 200 x 59	W 250 x 38.5
8	4625	W 200 x 59	W 460 x 82	2786	W 200 x 59	W 360 x 57.8
7	5600	W 310 x 226	W 460 x 89	3337	W 310 x 158	W 360 x 57.8
6	6020	W 310 x 226	W 460 x 97	3778	W 310 x 158	W 410 x 67
5	6580	W 310 x 226	W 460 x 97	4188	W 310 x 158	W 410 x 67
4	7520	W 360 x 421	W 460 x 128	4606	W 360 x 287	W 410 x 75
3	8640	W 360 x 421	W 460 x 128	5072	W 360 x 287	W 460 x 82
2	9600	W 360 x 421	W 460 x 144	5589	W 360 x 287	W 460 x 89
1	12000	W 360 x 592	W 460 x 177	6968	W 360 x 421	W 530 x 101
-1	-	W 360 x 592	W 200 x 100		W 360 x 421	W 310 x 67

4. Numerical Model

To perform nonlinear analyses of the prototype, a model was developed on the OpenSees platform. This model was based in the BRBF model proposed by Ariyaratana and Fahnestock [11]. Force-based beam-column elements with fiber sections were used to model the BRBF beams and columns, considering a uniaxial bilinear steel material with kinematic hardening (Steel01). Columns were fixed at the base and their lateral displacement was restrained at the ground level. The gusset plates in the beams and columns are incorporated as elastic beam-column elements near the beam-column connections. Beams are continuous between columns and pins are introduced in the beams adjacent to the gusset plates.

Each BRB was modeled as a truss element of constant area and the variation of sections along the brace (i.e. BRB connection region, non-yielding BRB core region, and yielding BRB core region) was represented by employing an equivalent elastic modulus. This modulus is based on a BRB length that is taken as 70% of the distance between the working points, a yielding core length that is taken as 70% of the BRB length, and a non-yielding BRB core region that is assigned an area five times that of the yielding core. Thus, the truss element area was equal to the area of the yielding BRB core region. Giuffré-Menegotto-Pinto steel material (Steel02) was used for BRB elements, which were connected to gusset plates modeled as elastic elements.

To account for second-order effects due to the gravity loads, a single leaning column was included. This column, shown in Fig. 2, is pinned at the base, and is constrained to have the same lateral displacement as the braced frame at each floor level, simulating a rigid diaphragm. The leaning column is also made up of force-based fiber elements, with properties based on the sum of the cross-sectional properties of the gravity columns tributary to the BRBF. The seismic mass of each floor of the frame corresponds to the total mass of the floor divided by the number of braced frames acting in the direction of analysis, and it was distributed between the center of gravity of the BRBF and the node of the leaning column at each floor level. As indicated in NCh433 [8], these values are based on the structural self-weight, superimposed dead load and a 25% of the live load. However, for the US design the live load is not included. Rayleigh damping was used with viscous damping ratio of 3% assumed in the first and second modes.

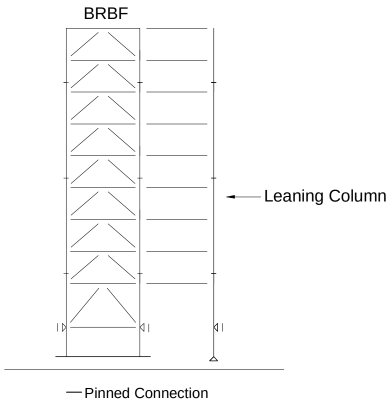


Fig. 2 – OpenSees Model

5. Nonlinear Analyses

Two types of nonlinear analyses were performed using the OpenSees model to evaluate the seismic performance of the prototypes: a nonlinear static pushover analysis using a load profile consistent with the first mode of the structure for the Chilean and the US prototypes, and dynamic analyses with eighteen acceleration records from different Chilean earthquakes, only for the Chilean prototype.

Two of the structural failure criteria proposed by Fermandois [6] were used to define the ultimate limit state of the building, and hence the structure's capacity:

1. A maximum axial ductility (μ_c) of BRBs of 20.
2. A cumulative plastic ductility (CPD_c) of BRBs of 300.

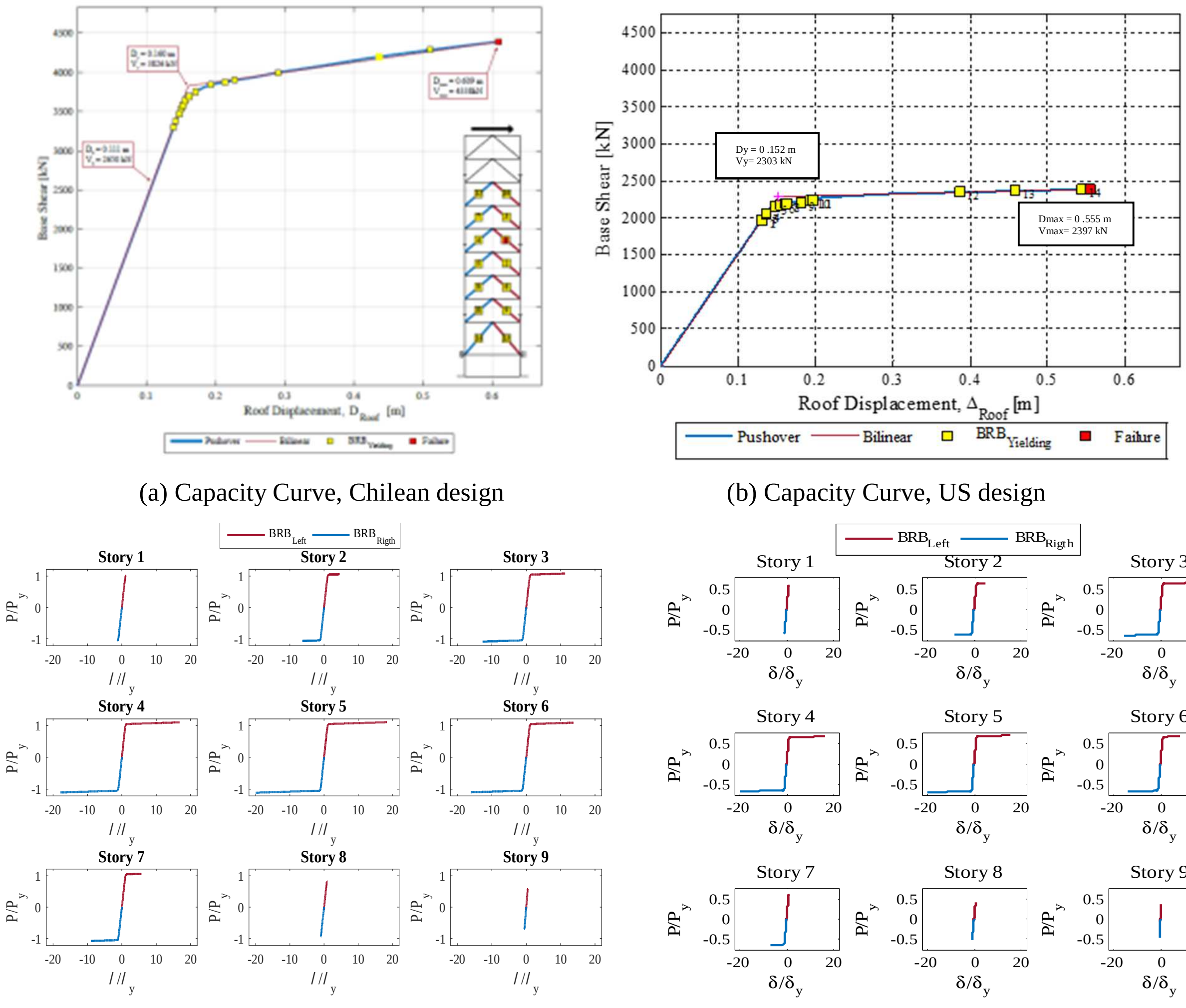
The first criteria is valid for both static and dynamic analyses, while the second is valid only for dynamic and cyclic analyses. The node rotation criteria was not considered because the beam to column connections were modeled as pinned.

5.1 Pushover analysis results

This static analysis is defined by FEMA P695 [12] recommendations, which establishes that the combination used for gravity loads is given by 1.05 times the dead load and a 25% of live load. The lateral force pattern indicates that, at each floor, the force must be proportional to the product of the story mass and the modal form associated to the structure's fundamental period.

Figs. 3(a) and 3(b) shows the structures response for this analysis. There is no strength degradation and no evidence of P-Δ effects due to the gravity column. The ultimate state is reached at a roof displacement of 1.49% of the building height for both the Chilean and US designs. Figs. 3(c) and 3(d) shows that the inelastic deformation concentrates in the middle stories, while the first and top stories remain almost elastic. The limit state of the building is reached by the failure of the BRB on the right (in compression) at the 5th story for both designs.

According to the story drift reached by the structure (Figs. 4(a) and 4(b)), which is directly related to the ductility of the BRB members, the building response in this static analyses is affected by the continuity and boundary conditions of the columns at the structure base. These columns are capable of resisting large story shears through bending due to the fact that they are "laterally anchored" in the basement level. This is confirmed in Figs. 4(c) and 4(d), where floor displacements are larger at the intermediate levels. For both frames, the design was done according to the modal response spectrum procedure, which accounts for higher mode contributions. In contrast, the static pushover analysis was conducted with a first-mode profile. This difference between the force profile used in design and the force profile used for analysis leads to a lack of uniformity in the distribution of BRB ductility demand. The main energy dissipation (Figs. 4(g) and 4(h)) is produced in the middle story BRB members. The story shear distribution is consistent with the previous results; there is no significant difference between the first and second floor due to the low level of inelastic demand on the first level, while the top floor nearly reaches the design shear.



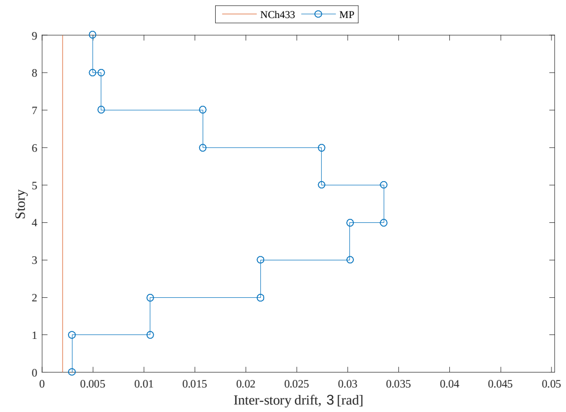
(a) Capacity Curve, Chilean design

(b) Capacity Curve, US design

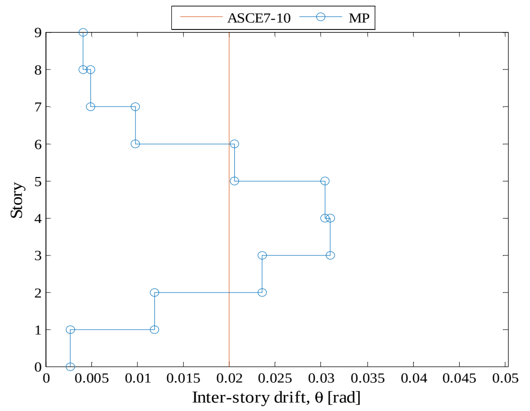
(c) BRB response, Chilean design

(d) BRB response, US design

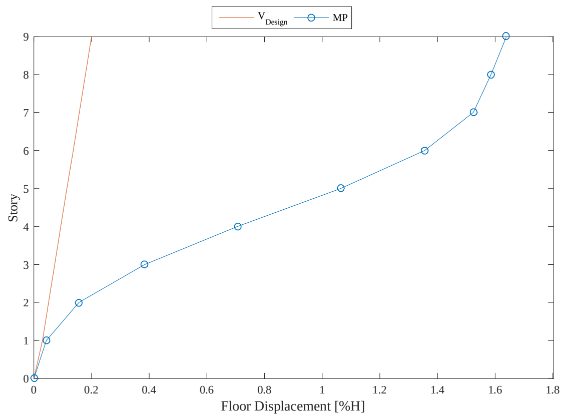
Fig. 3 – Pushover general results



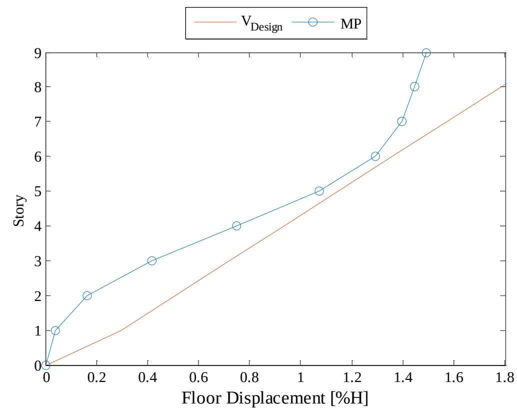
(a) Story Drifts, Chilean design



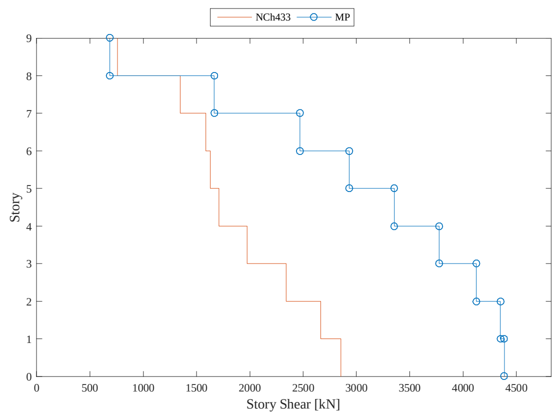
(b) Story Drifts, US design



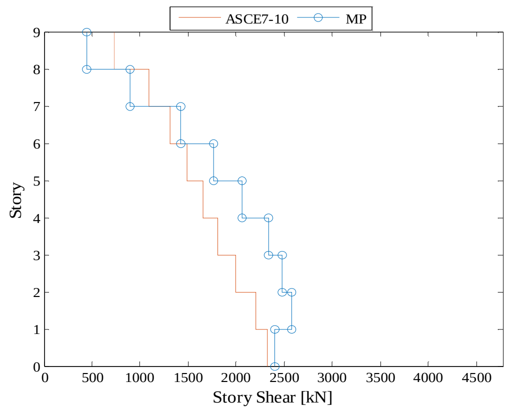
(c) Floor displacements, Chilean design



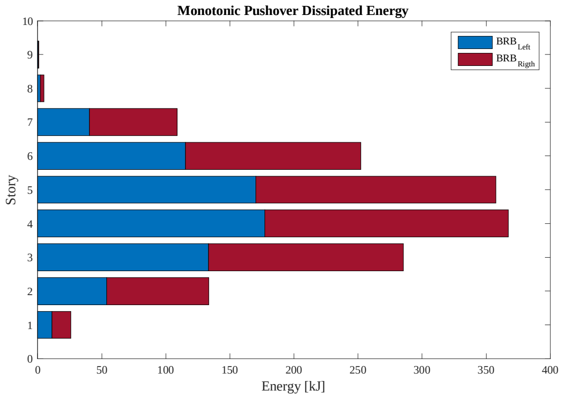
(d) Floor displacements, US design



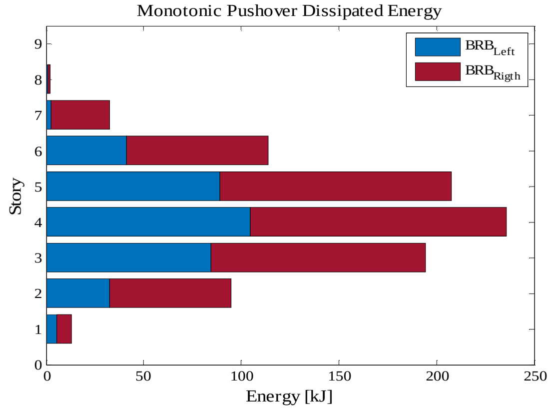
(e) Story Shear, Chilean design



(f) Story Shear, US design



(g) Dissipated energy, Chilean design



(h) Dissipated energy, US design

Fig. 4 – Pushover story results

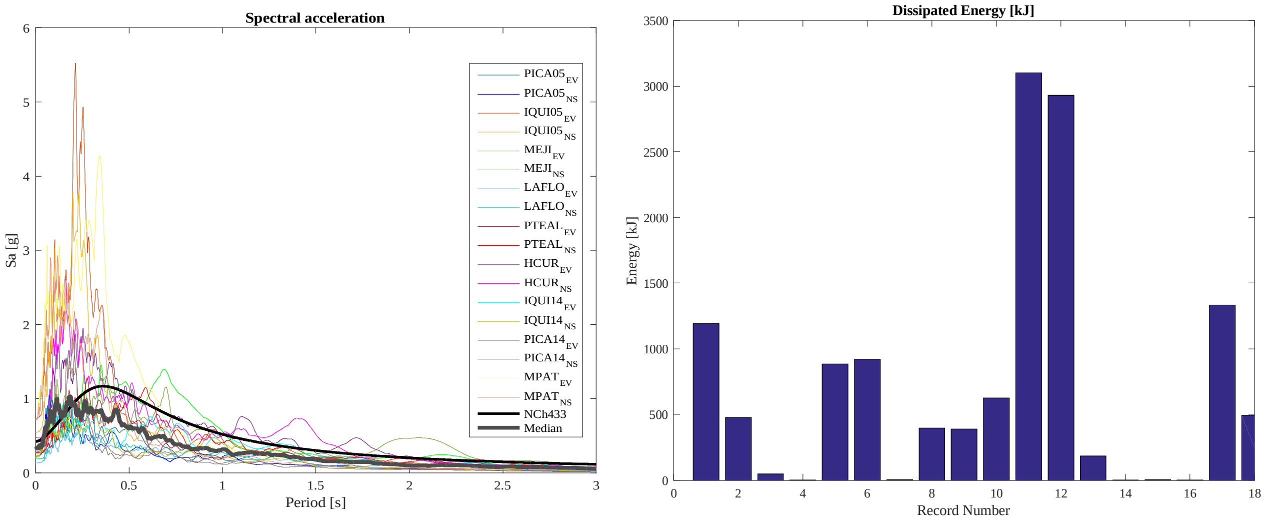
5.2 Time-history analysis results

The Chilean prototype was subjected to eighteen acceleration records, obtained from the horizontal components (east-west and north-south) measured at nine stations. The main feature why they were selected, is the similarity between soil conditions between the stations and the building foundation. Earthquakes from different zones of Chile of the last ten years were considered, including records from February 27, 2010. Table 2 shows a summary of the selected records.

Table 2 – Ground Motion Record Summary

#	Epicentre	Date	Station	Duration [s]	Mw	Component	PGA [g]
1	Tarapacá	13-06-05	Pica	252	7.8	PICA05_EW	0.735
2	Tarapacá	13-06-05	Pica	252	7.8	PICA05_NS	0.544
3	Tarapacá	13-06-05	Iquique	196	7.8	IQUI05_EW	0.227
4	Tarapacá	13-06-05	Iquique	196	7.8	IQUI05_NS	0.217
5	Tocopilla	14-11-07	Mejillones	218	7.7	MEJI_EW	0.141
6	Tocopilla	14-11-07	Mejillones	218	7.7	MEJI_NS	0.42
7	Cobquecura	27-02-10	La Florida	208	8.8	LAFLO_EW	0.133
8	Cobquecura	27-02-10	La Florida	208	8.8	LAFLO_NS	0.186
9	Cobquecura	27-02-10	Puente Alto	147	8.8	PTEAL_EW	0.268
10	Cobquecura	27-02-10	Puente Alto	147	8.8	PTEAL_NS	0.266
11	Cobquecura	27-02-10	Hospital Curicó	180	8.8	HCUR_EW	0.414
12	Cobquecura	27-02-10	Hospital Curicó	180	8.8	HCUR_NS	0.475
13	Iquique	01-04-14	Iquique	297	8.2	IQUI14_EW	0.316
14	Iquique	01-04-14	Iquique	297	8.2	IQUI14_NS	0.202
15	Iquique	01-04-14	Pica	286	8.2	PICA14_EW	0.335
16	Iquique	01-04-14	Pica	286	8.2	PICA14_NS	0.279
17	Illapel	16-09-15	Monte Patria	470	8.4	MPAT_EW	0.831
18	Illapel	16-09-15	Monte Patria	470	8.4	MPAT_NS	0.713

The elastic spectral acceleration for a 3% damping is shown in Fig. 5(a). As can be seen in this figure, the Chilean code exceeds almost all the analyzed earthquakes for the fundamental period of the building, with the exception of the Hospital Curicó station. Therefore, it is expected that the structure would not suffer significant damage. As can be seen In Fig. 5(b), the time-history analysis using the Hospital Curicó record presents the larger energy dissipation, but the structure does not reach any failure criterion, while for other records the structure remains elastic (dissipated energy is very low), which is consistent with the above information.



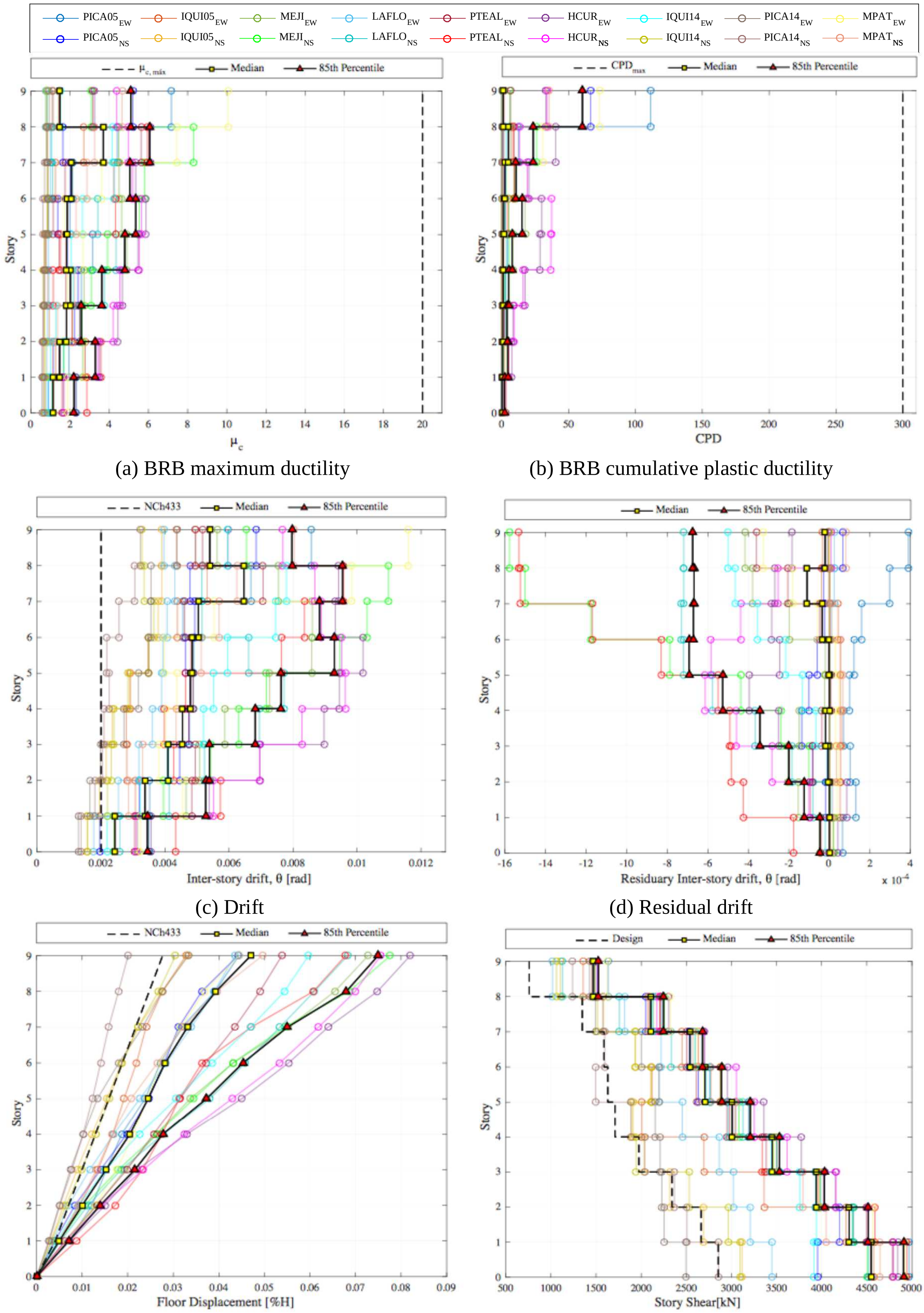
(a) Records spectral acceleration (b) Records analysis dissipated energy

Fig. 5 – Records analyses

The inelastic demand distribution along the building stories for each record time-history analysis is presented in Fig. 6. The structure does not reach any of the two structural failure criteria for any of the eighteen analyses and has a large reserve capacity, as can be seen in Figs. 6(a) and (b). Fig. 5(c) shows that the building exceed the design story drift limit when incurring in the inelastic range, as it happens with almost all earthquakes. The maximum drift is reached for the Hospital Curicó record; however for some records with lower dissipated energy, the maximum inter-story drifts in the upper levels are greater, as it happens with Pica (2005) and Monte Patria (2015). This two ground motions have in common a large PGA (peak ground acceleration) value, which appears to affect the displacement response of the roof.

As with the pushover analysis, even when the structure appears to respond in the fundamental vibration mode, there is evidence of higher vibration mode response for this building in some records, as can be seen in Figs. 6 (c) and (e), but this does not coincide necessarily with the most damaging earthquakes. All this information indicates that the response of this building during an earthquake does not depend only on the magnitude and peak ground acceleration of the record, but also on the way the structure responds to the earthquake. Residual story drifts (Fig. 6(d)) are very small, reaching maximum values of 0.17% of the story height, and do not represent a significant problem. Nevertheless, again, the earthquake characteristics play an important role.

For Hospital Curicó record, the energy dissipation is concentrated in the middle floors, and the first story remains elastic during the analysis, similarly to the pushover analysis. This again shows the effect of the continuity of the columns at the basement level in the structure's response. In Fig. 6(f), the base shear is almost two times the design base shear and even so, the structure has a significant remaining deformation capacity and does not reach the ultimate state. An interesting phenomenon is that for the Iquique 2014 earthquake records, the structure reaches the design shear and maximum floor displacements, with all its elements remaining elastic.



(a) BRB maximum ductility

(b) BRB cumulative plastic ductility

(c) Drift

(d) Residual drift

(e) Maximum floor displacement

(f) Story Shear

Fig. 6 – Time History Results

6. Conclusions

For static pushover analysis, the general behavior of the structures is very similar between the design of Chile and the United States with no strength degradation or evidence of P-Δ effects due to the gravity column. The US design is much lighter and more flexible than the Chilean design. The US design base shear is only marginally smaller than the Chilean design base shear, but the proportions of the Chilean BRBF are governed by the very strict drift limit, which causes the system to be much stiffer and stronger.

NCh433 seismic input parameters for the structure design are very strict; minimum base shear and lateral displacement requirements induce a stiff structure, having a significant impact in the final response of the building. For the time-history analyses, the structure exhibits a very good behavior, seldom reaching significant inelastic deformations and far from the ultimate state or a failure mechanism, maintaining a large reserve capacity. There is no evidence of capacity degradation or P-Δ effects on any of the analyses performed with the structure.

Even when the structure remains almost elastic for time-history analyses, there is a level of residual story drift, which could be amplified and become a serious problem in case that the structure incurs in higher levels of inelastic deformations. In this work, the first story was not affected in any of the analyses performed. The main reason for this result is the presence of a basement level. The first story columns are continuous into the basement, which makes them capable of resisting large story shears through bending, without significant inelastic deformations. For this study, failure was considered when the first BRB member reached a limit state. Therefore, it is a lower bound of the structure's capacity.

In general, the structure responds in the fundamental vibration mode for the studied ground motion records. However, the final response depends on the earthquake and record characteristics, and the boundary conditions at the ground level of the building. Along the same lines, the peak ground acceleration is not an indicator of the structure's general behavior and damage, but they depend in the way the structure interacts with the characteristics of the earthquake. Nevertheless, high PGA values do affect the maximum roof displacement. The response modification factors show that the structure has a reasonable ductility but low level of overstrength.

The same general results obtained for this 9-story structure were obtained for a 4-story and a 15-story building [13]. For pushover analyses there is no strength degradation and no evidence of P-Δ effects due to the gravitational column, and for the same records, the ultimate state is not reached in time history analyses. Also, all the inelastic deformation is concentrated in the middle floors just as the 9-story building.

7. References

[1] Black, C. J., Makris, N., and Aiken, I. D. (2002). "Component testing, stability analysis and characterization of buckling restrained 'unbonded' braces." Technical Rep. No. PEER 2002/08, Pacific Earthquake Engineering Research Center, Univ. of California at Berkeley, Berkeley, Calif.

[2] Kimura, Y., Takeda, Y., Yoshioka, K., Furuya, N., and Takemoto, Y. (1976). "An experimental study on braces encased in steel tube and mortar." Proc., Architectural Institute of Japan Annual Meeting, 1041–1042, Japan (in Japanese).

[3] Merritt, S., Uang, C. M., and Benzoni, G. (2003). "Subassemblage testing of Star Seismic buckling-restrained braces." Structural Systems Research Project, Rep. No. TR-2003/04, Univ. of California at San Diego, San Diego.

[4] Watanabe, A., Hitomi, Y., Saeki, E., Wada, A., and Fujimoto, M. (1988). "Properties of brace encased in buckling-restraining concrete and steel tube." Proc., 9th World Conf. on Earthquake Engineering, Tokyo-Kyoto, Japan.

[5] Tremblay R, Poncet L, Bolduc P, Neville R and DeVall R (2004): Testing and design of buckling restrained braces for Canadian applications. *13th World Conference on Earthquake Engineering*, Vancouver, Canada.

[6] Fermandois G. "Marcos con riostras de pandeo restringido: Comportamiento y factores de modificación de la respuesta sísmica". Memoria para optar al grado de Magister en Ciencias de la Ingeniería Civil. *Universidad Técnica Federico Santa María*, Valparaíso, Chile, 2009.

[7] Tremblay R, Dehghani M, Fahnestock L,Herrera L, Canales M, Clifton C, Hamide Z (2016): Comparison of seismic design provisions for buckling restrained braced frames in Canada, United States, Chile, and New Zealand. Structures. In press.

[8] INN (2009). NCh433.Of1996, Diseño sísmico de edificios, including 2009 modifications and 2011 Decreto 61. *Norma Chilena Oficial*, Santiago, CH.

[9] AISC (2010a). AISC-341-10, Seismic Provisions for Structural Steel Buildings. *American Institute of Steel Construction*, Chicago, IL, USA.

[10] AISC (2010b). AISC-360-10, Specification for Structural Steel Buildings. *American Institute of Steel Construction*, Chicago, IL, USA.

[11] Ariyaratana C and Fahnestock L (2010): Evaluation of buckling-restrained braced frame seismic performance considering reserve strength. *Engineering Structures*, **33** (1), 77-89.

[12] FEMA. 2009. FEMA P695 Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings, *Federal Emergency Management Agency*, Washington DC, USA.

[13] Canales M. "Diseño y estudio numérico del desempeño sísmico de marcos con riostras de pandeo restringido". Memoria para optar al título de Ingeniera Civil. *Departamento de Ingeniería Civil, Universidad de Chile,* Santiago, Chile, 2016.