Steel frame with buckling-restrained braces subject to near and far-fault inputs

G. L. Palazzo, P. Martín, F. Calderón, V. Roldán, & G. Maldonado
National Technological University, Mendoza Faculty, Ceredetec, Argentina.

F. López Almansa
Technical University of Catalonia, Barcelona, Spain.

X. Cahís
University of Girona, Girona, Spain.

SUMMARY:
A great variety of passive energy dissipation systems have been proposed in the last decades to improve the seismic resistance of new and existing constructions. However, new numerical and experimental researches are needed to evaluate the efficiency of such systems under different types of earthquakes. In particular, it is necessary to know if a building frame protected with buckling restrained braces can achieve a good performance when undergoes far-field and near-field inputs. In this paper, a steel frame that was previously tested in a shaking table is considered. The main objective is to study numerically the seismic performance of this structure with and without buckling restrained braces. The structural analysis considers the nonlinear behavior of the structure and of the protecting devices. The numerical results show the efficiency of the devices under both types of inputs.

Keywords: near and far fault inputs, steel frame, buckling-restrained braces, non-linear analysis

1. INTRODUCTION

In the last decades numerous investigations have been done about new technologies for seismic protection of building (Soong & Dargush, 1997). These technologies can be applied either to new structures or to constructions needing retrofitted. Figure 1shows a building with buckling-restrained braces (BRB).

Figure 1. Building protected with BRB

The BRB are one of these new technologies (AISC 341, 2005). This type of passive energy dissipation system has a ductile steel core, which is designed to yield both in tension and compression. To preclude the buckling of the core in compression, it can be placed inside a steel casing and then can be filled with mortar or concrete. Prior to casting mortar, the steel core can be cover with an unbonded material in order to minimize the transfer of axial force from steel core to the mortar. Experimental and analytical results have shown that the BRB is a very effective device for the dissipation of energy by yielding of the steel core (Black et al., 2001).

But more research is necessary to verify if structures with BRB have a good performance when they are subject to different kind of earthquakes. Attending to this issue, the object of this paper is to study numerically the performance of a steel frame with/without BRB, subject to two kind of earthquake.

The steel frame is a full-scale four-story building (without BRB), which was tested to collapse in September 2007 on the world's largest three-dimensional shaking table located at Japan (E-Defense Laboratory). The test was conducted by applying a scaled version of near-fault motion recorded during the 1995 Kobe earthquake. The characteristics of this specimen are given in section 2.

A first set of BRB was designed according to a methodology development by one of the authors (Palazzo, 2009).Also a second set of BRB was proposed. The dimensions in this case were given according of the drift angle at each story in the frame with the first BRB designed. The properties of these two sets of BRB are presented in section 2.

Far-field and near-field earthquakes were considered in this study, with a selection of accelerograms based on the ground motion record sets of the FEMA P695 (2009). In section 3 are identified the accelerograms selected.

The nonlinear dynamic analysis was carried out with the software SeismoStruct (SeismoSoft, 2011), a computer program for static and dynamic nonlinear analysis of framed structures. The structural modeling is described in section 4.Then the numerical results are presented in the section 5, and also these results are analyzed in this section. Finally, in the conclusion (section 6) the performance of the model with/without BRB, subject to far and near-field inputs is summarized.

2. OUTLINE ABOUT THE SPECIMEN

2.1. Specimen tested (without BRB)

The main characteristic of the specimen presented in this section are based in the date given by Pavan (2008), Yamada *et al.* (2008), and Ohsaki*et al.* (2008). More details of the specimen can be found in these papers.

The four-story building is made by steel moment resisting frames. Figure 2 shows a draw and a photo of the specimen.

a)Draw b)Specimen without walls c)Specimen tested

Figure 2. Steel frame tested in the E-Defense

Along x-direction there are two frames composed by two bays 5m long, while in y-direction the frames are three, with one bay 6m long. Interstory height is 3.5m for the story 2 to 4, and 3.875 m for the story 1.

Slabs at second to forth level consist in composite deck floor, with 175 mm height. Instead, roof floor is a reinforced concrete slab, with a flat steel deck at its bottom, with a thickness of 150mm.

All connections were made using details and fabrication practice developed following the 1995 Hyogoken-Nanbu earthquake. They force the eventual formation of the plastic hinge away from the weaker joint between the column and the beam. Column bases are connected to concrete blocks (with 1.5m height), these blocks create the connection with the shaking table.

External walls consist in ALC (autoclaved, aerated concrete) panels, with a thick of 0.125m. Internal partitions are made using LGS (light-gauge steel) backing board installed on aluminum frames. Glass windows with aluminum sash were installed within external ALC panels openings. A light-gauge steel was used to hanging ceiling at each floor.

The weight of each part of the specimen is given in Table 2.1.

Table 2.1. Weight of each part of the specimen [kN]

	Floor	Steel fr.	Ex. wall	Int. wall	Ceilling	Parapet	S. Syst.	C. Floor	Total
Roof floor	459	20			12	71		2	565
4-th Story		19	79	35					133
4-th Story	270	24			3		47	4	349
3-rd Story		18	73	30					122
3-rd Story	260	32			3		47	4	347
2-nd Story		18	73	30			8		130
2-nd Story	260	41					47	4	352
1-st story		27	76				12		115
Total	1248	200	302	95	19	71	162	15	2113

2.2. Bucking-Restrained Braces considered in the structural modeling

The study in this paper was done only in the longitudinal direction of the frame. For this reason, only BRB was considered in this direction.

The design of the BRB is defined for the follow parameters of the steel core: yielding tension of the steel, length of the core, and the area of its transversal section. The yielding tension of the steel, of 240 MPa, was defined according to the steels that can be found in Argentina. The length of the core was fixed in function of the length of the diagonals in the frame. Two sets of BRB (BRB1 y BRB2) were considered, the difference between them was only in the area of its transversal section.

The area of each transversal section in the BRB1 sets, that was different in each story, was determinate according to the design methodology proposed by Palazzo (2009). Following this procedure, and assuming circular section, the diameter of the steel core adopted in the model was: 30 mm (story 1 and 2), 20 mm (story 3), and 10 mm (story 4).

The area of each transversal section in the BRB2 sets, were not determinate with a special procedure. Only was considered the drift angle at each story of the model with BRB1. A bigger diameter was assigned when the drift angle was larger. With this assumption, the diameter of the steel core adopted in the model was: 25 mm (story 1 and 2) and 20 mm (story 3 and 4). The axial stiffness of the steel cores was: reduced 30.6 % in the story 1 and 2, maintained in the story 3, and increased 300% in the story 4. The total volume of steel cores were reduced 11.15 %, respect the BPR1.

3. GROUND MOTION

3.1. Ground Motion for the Specimen tested in the E-Defense

The building was shaken and collapsed by applying a scaled version of the near-fault motion recorded in Takatori during the 1995 Kobe earthquake**.** More 3D shaking table tests were performed consecutively with increasing levels of seismic motion to evaluate the effect of plastic deformation: i - Takatori scaled to 40% (elastic level); ii - Takatori scaled to 60% (incipient collapse seismic level, elasto-plastic); and iii - Takatori in full scale (collapse seismic level).

All result data have to be submitted for incipient collapse level while the collapse level was used to evaluate the time of collapse.

3.2. Ground Motion for the Structural Analysis

3.2.1. Ground motion for verifying the model
The model considered in this paper (described in section 4) was subject to the three components of the Takatori ground motion time history, scaled to 60%.The numerical result data were not compared with the experimental result data, because the ground motion time history used in the shaking table were a little different of the Takatori record. But it was possible to compare these results with the numerical result data obtained by Pavan (2008).

3.2.2. Ground motions for studying the far-field vs. near-field responses
The ground motion record sets of the FEMA P695 - Appendix A - (2009) were considered in this study to select accelerograms. This appendix describes the selection of ground motion record sets for collapse assessment of building structures using nonlinear dynamic analysis methods.

The ground motion record sets include a set of ground motions recorded at sites located greater than or equal to 10 km from fault rupture, referred to as the “far-field” record set, and a set of ground motions recorded at sites less than 10 km from fault rupture, referred to as the “near-field” record set. The near-field record set includes two subsets: i - ground motions with strong pulses, and ii - ground motions without such pulses.

In this paper ten accelerograms were selected of the Appendix A, Table A-4A (far-field record set), and other ten accelerograms were selected of Appendix A, Table A-6A (near-field record set, with strong pulses). Like the first ten accelerograms were cortical, ten subductive far-field record set were added.

All the ground motion records considered in this study were scaled. The scale factor of each record was selected such as its spectral acceleration at the main elastic period of the frame was similar to the spectral acceleration in this period, according to the Argentinian code (INPRES-CIRSOC 103, 2008).

Table 3.1 presents the data of each ground motion record set.

Table 3.1.(a) Ground motion record data (far-field record set, cortical)

Name	Date	Magnit.	Name	S. factor	Recording St.	Com.	PGA [g]	PGV[cm/s]
Duzque	11/12/99	7.1	1 ffc	0.637	Bolu	90	0.822	62.1
Duzque	11/12/99	7.1	2 ffc	1.0.	Bolu	0	0.728	56.44
Cape Mend.	04/25/92	7.0	3 ffc	2.16	Rio Dell Ov.	360	0.549	41.87
Kobe	01/16/95	6.9	4 ffc	1.49	Nishi Akashi	0	0.509	37.28
Kobe	01/16/95	6.9	5 ffc	1.43	Nishi Akashi	90	0.503	36.62
Northridge	01/17/94	6.7	6 ffc	1.31	Canyon Ctry.	270	0.482	44.91
Chi Chi	09/20/99	7.6	7 ffc	0.93	CHY 101	N-S	0.440	115.03
Northridge	01/17/94	6.7	8 ffc	1.65	Canyon Ctry.	0	0.410	42.97
Cape Mend.	04/25/92	7.0	9 ffc	1.61	Rio Dell Ov.	270	0.385	43.8
Chi Chi	09/20/99	7.6	10 ffc	1.56	CHY 101	E-W	0.535	70.65

Table 3.1.(b) Ground motion record data (far-field record set, subductive)

Name	Date	Magnit.	Name	S. factor	Recording St.	Com.	PGA [g]	PGV[cm/s]
Japan 2011	11/03/11	9.0	1 ffs	1.E-4	MYG004	N-S	2.58	5807
Japan 2011	11/03/11	9.0	2 ffs	1.E-4	MYG004	E-W	1.24	4796
Chile 2010	08/04/10	8.8	3 ffs	1.09	Maule	Ch. 1	0.401	69.28
Chile 2010	08/04/10	8.8	4 ffs	1.84	Maule	Ch. 2	0.286	52.58
Mex. 1985	19/09/85	8.0	5 ffs	4.95	CDAF	N90W	0.096	37.74
Peru 2007	15/08/07	8.0	6 ffs	9.35	La Molina	E-W	0.08	11.64
Peru 2007	15/08/07	8.0	7 ffs	11.29	La Molina	N-S	0.07	159.3
Mex. 1985	19/09/85	8.0	8 ffs	5.3	CDAO	N00E	0.07	35.98
Mex. 1985	19/09/85	8.0	9 ffs	7.66	CU01	N90W	0.034	9.27
Mex. 1985	19/09/85	8.0	10 fcs	10.54	CU01	S00E	0.029	10.16

Table 3.1.(c) Ground motion record data (near-field record set, with strong pulses)

Name	Date	Magnit.	Name	S. factor	Recording St.	Com.	PGA [g]	PGV[cm/s]
Northridge	17/01/94	6.7	1 nfp	0.78	Sylmar-Hosp.	360	0.843	129.4
Chi Chi	09/20/99	7.6	2 nfp	0.933	Chi Chi	E-W	0.814	126.2
Cape Mend.	04/25/92	7.0	3 nfp	0.59	Petrolia	90	0.662	89.68
Northridge	01/17/94	6.7	4 nfp	0.75	Sylmar-Hosp.	90	0.604	78.10
Chi Chi	09/20/99	7.6	5 nfp	0.78	Chi Chi	N-S	0.603	78.82
Cape Mend.	04/25/92	7.0	6 nfp	0.68	Petrolia	0	0.590	48.14
Erzikan	03/13/92	6.7	7 nfp	1.16	Erzikan	N-S	0.515	83.96
Erzikan	03/13/92	6.7	8 nfp	1.13	Erzikan	E-W	0.496	64.28
Imp. Valley	10/15/79	6.5	9 nfp	1.48	El Centro	230	0.439	109.8
Imp. Valley	10/15/79	6.5	10 nfp	1.05	El Centro	140	0.410	64.86

The far-field and near-field ground motion record sets were applied only in the longitudinal direction of the model.

4. STRUCTURAL MODELING

The structural modeling was performed according with the date given by Pavan (2008).

All analysis have been carried out using SeismoStruct (SeismoSoft, 2011), a finite element analysis program, used for seismic analysis of framed building. The software is fibre element-based, able to predict accurately the distribution of damage because it spreads material inelasticity both along the element length and across its section depth.

To determine the amount of mass to apply on each beam, its tributary area was defined on the geometry of the specimen, assuming a distributed mass over the diaphragm. The tributary area for each beam is given in section 3.1 of Pavan (2008). Also in this section, the values of additional mass per unit length applied to the beams in the model are given.

The properties of concrete are given in section 3.2.1 of Pavan (2008). A nonlinear constant confinement concrete model was adopted for the concrete of slabs. Five parameters that characterized the model are given in Table 10 of Pavan (2008).

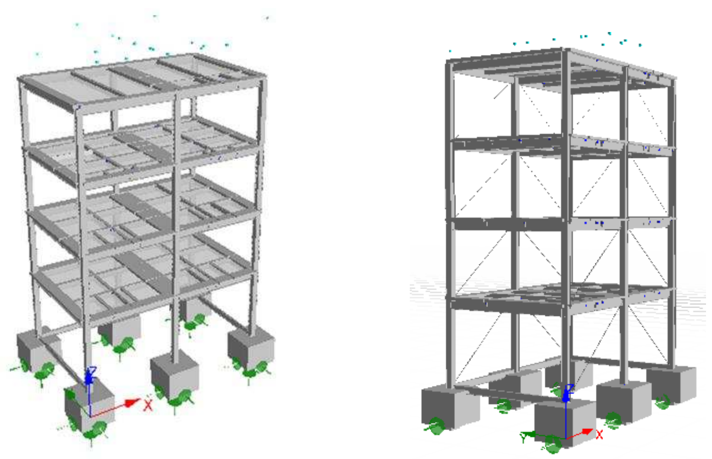
The properties of steel are given in sections 3.2.2 (Table 11) and 6 of Pavan (2008).Two different kinds of steel were used in the specimen. Beams were connected directly to columns edge points (the panel zone was not treated in any detailed way). It was supposed that the lack of contribution to global deformation coming from panel zone yielding could be compensated by the absence in the model of non structural elements that add stiffness and reduce structural deflection. A bi-linear curve was adopted to represent the constitutive law for the steel. The data utilized to characterize the elastic frame elements of the model are given in Table 13 of Pavan (2008), and the steel bi-linear constitutive model characteristic parameters are given in Table 18. Also a bi-linear curve was adopted to represent the constitutive law for the BRB (its parameters were given in section 2.2. of this paper).

Columns were modeled using "Rectangular Hollow section". Girders were modeled using "Composite I section" for which three materials had to be defined: the steel for the profile, the concrete for the cover and the confined concrete. The effective slab width and thickness for different beams are given in Table 19 of Pavan (2008). BPR were modeled using circular solid section (its parameters were given in section 2.2. of this paper).

Beam and columns were modeled as "Inelastic frame elements – infrm -", that are 3-dimensional elements characterized both by geometric and material inelasticity. Bi-linear material uniaxial response is the particular response of every individual fiber in which elements cross section is divided. In the model for this paper, a number of 100 fibers were used for every cross section (Pavan, 2008, used 200 fibers). BPP were modeled as inelastic truss element (it is particularly handy in those cases where there is a need to introduce members that work in their axial direction only).

Slabs were modelled as rigid diaphragms with no bending stiffness out of plane. At every level, nodes were linked between them with rigid links in XY plane.

In infrm elements formulation, hysteretic damping is already taken in consideration by material inelasticity that characterizes fibres. Another kind of damping was considered in the analysis: this is non-hysteretic damping, able to represent the dissipation of a small amount of energy mobilized during dynamic response of structures through phenomena such friction between structural and non-structural members, friction in opened concrete cracks, energy radiation through foundation. This kind of damping was defined globally for the whole structure and was assumed to be tangent stiffness-proportional. Considering that the analysis that had to be performed was expected to be highly inelastic, for its numerical stability a small amount of equivalent viscous damping, set equal to 0.5%, was introduced in the model. The stiffness parameter was calculated like the product of the first-mode period of vibration of the structure and the equivalent viscous damping, divided by π. The Hilber-Hughes-Taylor algorithm was used was used as method of time integration. The model considered in this paper, without BPB, is shown in Figure 3.



a)Without BRB b)With BRB

Figure 3. 3D steel frame models

5. NUMERICAL RESULTS AND DISCUSSION

5.1. Analysis of the model without BRB

The period of the first modes in longitudinal and transversal direction of the model was obtained by eigenvalue analysis in SeismoStruct (SeismoSoft, 2011). In Table 5.1 these values are compared with the experimental measured during the test of the specimen (Yamada et al., 2008). It was obtained a good agreement.

Table 5.1. Experimental and numerical first model period (s)

Direction	Model	Experimental	Dif. [%]
Longitudinal	0.82	0.76	7.6
Transversal	0.83	0.80	4.3

The result data of the model in this paper was compared with the calibrated numerical model of Pavan (2008). In these models the three component of the Takatori ground motion time history, scaled to 60%, was considered. The Table 5.2 shows the main result data.

Table 5.2. Comparison of two models excited with the Takatori ground motion

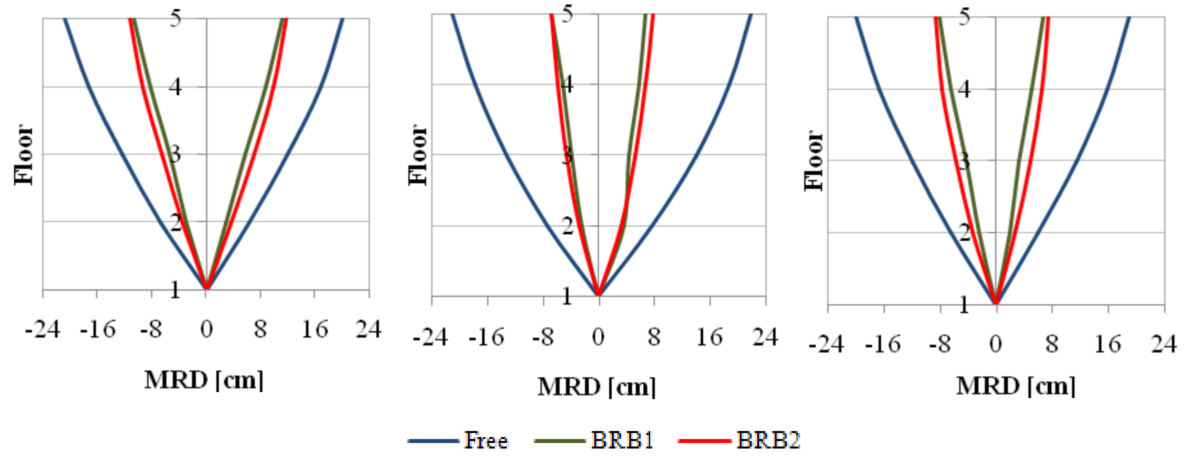
Maximum value of:	Model in this paper	Pavan's Model	Dif. [%]
relative displacement in floor 5 [mm]	221.00	222.00	0.45
absolute acceleration in floor 5 [m/s^2]	8.35	8.99	7.12
story shear in story 2 [kN]	1165.00	1670.00	30.24

Also in this case, if relative displacement and absolute acceleration is considered, a good agreement between the two models is obtained. The difference in the story shear values shows that the distribution of the mass in both models was different. Pavan (2008) also considered her model with the ground motion really applied to the shaking table. The author obtained a good approximation with the experimental results. This analysis was not considered in this paper because this last ground motion could not be gotten.

5.2. Result data of the model with/without BRB subject to far and near-field input

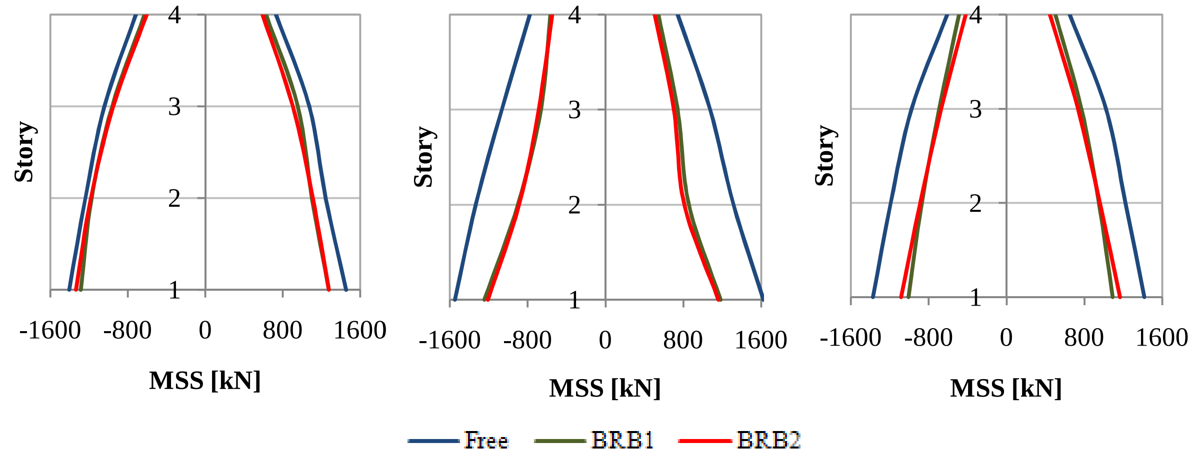
The model in SeismoStruct (SeismoSoft, 2011) was excited by 90 ground motions (applied only in the longitudinal direction): 30 for the free frame, 30 for the frame with BRB1 and 30 for the frame with BRB2. In each sets of 30 ground motions was considered: 10 from cortical far-field records, 10 from subductive far-field records, and 10 from near-field records.

The maximum value of: i - relative displacement from the base at each floor; ii - drift angle at each story; iii - absolute acceleration at each floor; iv - story shear at each story; and v - overturning moment at each floor, were considered in this paper. But only the mean values of relative displacement and story shear are presented in this section (see Figure 4 and 5).



a) Cortical far-field records b) Subductive far-field records c) Near-field records

Figure 4. Maximum floor relative displacement - MRD (mean values)



a) Cortical far-field records b) Subductive far-field records c) Near-field records

Figure 5. Maximum story shear – MSS (mean values)

The percentage of reduction of the steel with BRB1 and BRB1 response respect to the free steel response, if the maximum mean values are considered, are given in Table 5.3.and 5.4.

Table 5.2. Reduction of the maximum relative displacement (steel frame with BRB vs. steel frame without BRB).

Earthquake	Floor	BPR1 vs. free frame [%]		BPR2 vs. free frame [%]	
		MRD+	MRD+	MRD+	MRD+
Cortical far field records	5	44.7	48.8	41.7	46.3
	2	53.9	57.1	41.6	47.9
Subductive far field records	5	69.2	67.9	64.8	68.0
	2	53.7	66.1	59.1	60.9
Near field records	5	65.0	59.4	60.8	56.5
	2	68.3	62.5	53.2	49.3

Table 5.3. Reduction of the maximum story shear (steel frame with BRB vs. steel frame without BRB)

Earthquake	Floor	BPR1 vs. free frame [%]		BPR2 vs. free frame [%]	
		MRD+	MRD+	MRD+	MRD+
Cortical far field records	5	14.1	12.4	19.0	15.9
	2	12.4	8.6	12.4	5.1
Subductive far field records	5	26.5	26.9	31.7	29.5
	2	27.3	19.4	28.5	21.9
Near field records	5	22.9	20.3	31.4	31.2
	2	23.1	26.7	17.8	21.2

According with the Figures 4 and 5, and the Table 5.2 and 5.3:

i) The maximum relative displacements in the frame with BRB are smaller that in the frame without BRB (the reduction is bigger than 40%).

ii) The frame with BRB1 presented relative displacements little smaller than the frame with BRB2.

iii) The maximum story shears in the frame with BRB are smaller that in the frame without BRB (the reduction is bigger than 11%, but smaller that in the case of the relative displacements).

iv) The frame with BRB1 and BRB2 presented story shears almost equal.

v) There was not found a relation statically significant between the relative displacements (or the story shears) and the kind of input considered.

6. CONCLUSION

The numerical result data of a building (with/without buckling-restrained braces and excited with far-field and near-field ground motion records), was studying in this paper.

A model of a full-scale four-story steel frame, witch was tested in the E-Defense’s shaking table, was considered like a building. Also in the model were included two designs of BRB (BRB1 and BRB2). The first set of BRB was designed according to a methodology proposed by the authors. The other set of BRB was designed according to result data of the frame with the first set of BRB.

The model without BRB was done following the data of a model calibrated according to measured data in the test. Both models were analyzed with the same software. The nonlinear dynamic analysis was carried out with this software.

A total of 30 accelerograms were included in the model: 10 for cortical far-field records, 10 for subductive far-field records, and 10 near-field records. The model was run 90 times to consider the frame without BRB, with BRB1 and with BRB2.

The relative displacement and the story shear were presented in this paper, but also the drift angle, the absolute acceleration, and the overturning moment were studied.

The numerical result data show that with the two kinds of ground motion records, the response is reduced when BRB are included in the frame. This reduction is bigger when the parameters relative to the displacement are considered.

The methodology considered to design BRB1 resulted suitable.

Important changes in the response for the frame excited with the two king of ground motion were no find.

New experimental studies of BRB and building with BRB excited with far-field and near-field ground motion will permit to confirm the numerical result or to improve the structural modeling.

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REFERENCES

AISC 341 (2005). Seismic provisions for structural steel building. *American Institute of Steel Construction, Inc.*

Black, C., Makris, N., Aiken, L. (2004). Component testing, seismic evaluation and characterization of buckling restrained braces. *Journal of Structural Engineering*,**ASCE 130**, 329-337.

FEMA P695 (2009).Quantification of Building Seismic Performance Factors.Federal Emergency Management Agency.

INPRES-CIRSOC 103 (2008). Normas Argentinas para Construcciones Sismorresistentes – Parte I Construcciones en general. Instituto Nacional de Tecnología Industrial, Argentina.

Ohsaki, M., Kasai, K., Thiagarajan, G., Yang, Y. and Komiya, Y.(2008).3-D analysis methods for 2007 blind analysis contest.*14th World Conference on Earthquake Engineering*, Beijing, China.

Ohsaki, M., Kasai, K.,Hikino, T., and Matsuoka, Y. (2008).Overview of 2007 E-defense blind analysiscontest results.*14th World Conference on Earthquake Engineering*, Beijing, China.

Palazzo, G. (2009). Rehabilitation of seismic RC frames thru buckling restrained braces. PhD Dissertation,National Technological University, Mendoza, Argentina.

Pavan A. (2008). Blind prediction of a full-scale 3d steel frame tested under dynamic conditions. MSc Dissertation, Centre for Post-Graduate Training and Research in Earthquake Engineering and Engineering Seismology (ROSE School), Pavia, Italy.

SeismoSoft (2011).SeismoStruct - A computer program for static and dynamic nonlinear analysis of framed structures, V. 5.2.2. Available from URL: www.seismosoft.com.

Soong T. andDargush G. (1997).Passive energy Dissipation Systems in Structural Engineering, John Wiley.

Yamada, S.; Suita, K.; Tada; K., Kasai, K; Matsuoka, Y., and Shimada; Y. (2008).Collapse experiment on 4-story steel moment frame: Part 1 outline of test results. *14th World Conference on Earthquake Engineering*, Beijing, China.