Improving the Seismic Response of Tall Reinforced Concrete Buildings using Buckling Restrained Braces

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SUMMARY:
In the last years BRB were used in existing structures and in new ones as primary lateral force resisting elements. Retrofit of existing buildings in seismic areas can be made by using buckling restrained braces (BRBs), because they have the ability to sustain large inelastic deformations without important loss of strength. This study analyzes the possibility of using BRBs to improve the seismic response of tall reinforced concrete buildings. Therefore dynamic nonlinear analysis was performed on two tall structures: first, one structure with reinforced concrete walls (that has been designed according to the Romanian Seismic Design Code) and then the same structure with buckling restrained braces included. Computation was made for three recorded accelerograms. The aim of the study was to highlight the advantages and disadvantages of using BRBs together with reinforced concrete walls for improving the seismic response of tall buildings. Parameters like natural period, deformations and stresses were carefully evaluated and then some comparatively studies were made in order to establish the efficiency of the buckling restrained braces.

Keywords: buckling restrained braces, tall buildings, concrete walls

INTRODUCTION

The reinforced concrete building that is studied in this paper was initially designed following the Romanian design code P100-92. This code was updated and published as P100-2006 and it contains most of the Eurocodes guidelines. The building was designed to be built in Bucharest and its initial destination was for multiple apartments. The owner stopped the project after the initial design and started it again now, after more than 10 years. For that, the initial design must be checked according to the requirements of the new codes. He also changed the destination of the building, instead of apartments there will be only offices. This modification leads to higher loads, the increase of the importance factor and new performance level. A special require from the owner is to user an innovative system in order to improve the seismic response and the value of the building on the local market.

1. BUILDING DESCRIPTION

1.1 The geometry

The building that is analyzed has 20 stories, 17 stories above ground and 3 basements. Each story has 3 meters height and the plan dimensions of the building are 30 x 29 m, with a total height of 51 m. In this paper only the suprastructure was analyzed and it was considered fixed at the ground floor level. Because the torsion effects are insignificant only a 2D model was analyzed. The selected inner frame from **Fig. 1.1.1 a)** was studied.

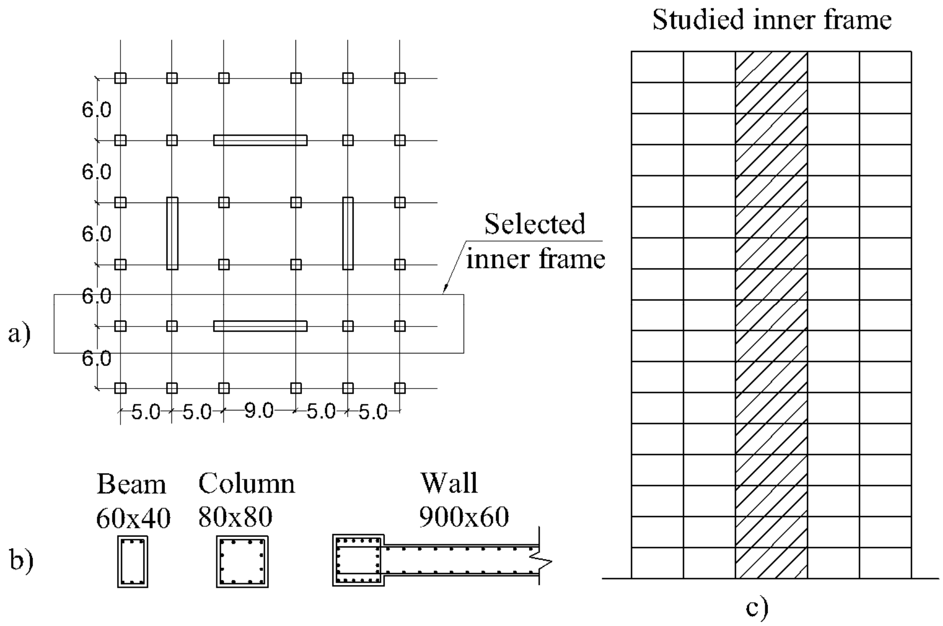

Figure 1.1.1. Building overview: a) Plane view; b) Cross-sections of the elements; c) Elevation of the studied frame

The initial design of the structure was done considering 2 reinforced concrete walls in each direction, concrete columns and beams, with the cross section shown in **Fig. 1.1.1 b)**.

1.2.Loads

The initial reinforced concrete building was designed at smaller loads than the requirements for the office building and the seismic design has changed too. The following loads have increased:

Table 1.2.1 Loads

Load	Initial Design	Actual Design	%
Partitioned walls	0.5 KN/m^2	3.0 KN/m^2	600%
Live Loads	2.0 KN/m^2	3.0 KN/m^2	50%

The self weight of the interior walls was taken initially 0.5 KN/m^2 and because of the new destination it must be at least 3.0 KN/m^2. Due to the office and technical areas, according to Eurocode 1 the building is considered in category B, which means a live load of 3.0 KN/m^2. The roof of the office building is now accessible and according to Eurocode 1 the live load is 3.0 KN/m^2. The characteristic snow load is considered 2.0 KN/m^2.
The loads from the structural elements in transverse direction were assigned to the inner frame beams.

1.3 Seismic load

The accelerograms used for time history analysis were recorded during the earthquakes from Vrancea in 1977, 1986 and 1990. They are presented in the **Fig. 1.3.1**. Because the accelerogram from 1977, component N-S, recorded at INCERC, Bucharest, is the most severe, with PGA=0.2g, in this paper are presented only the results for this seismic input. In **Fig. 1.3.2** is presented a comparison between design spectrum according to P100-1/2006 and the response spectrum of Vrancea 77 N-S accelerogram.

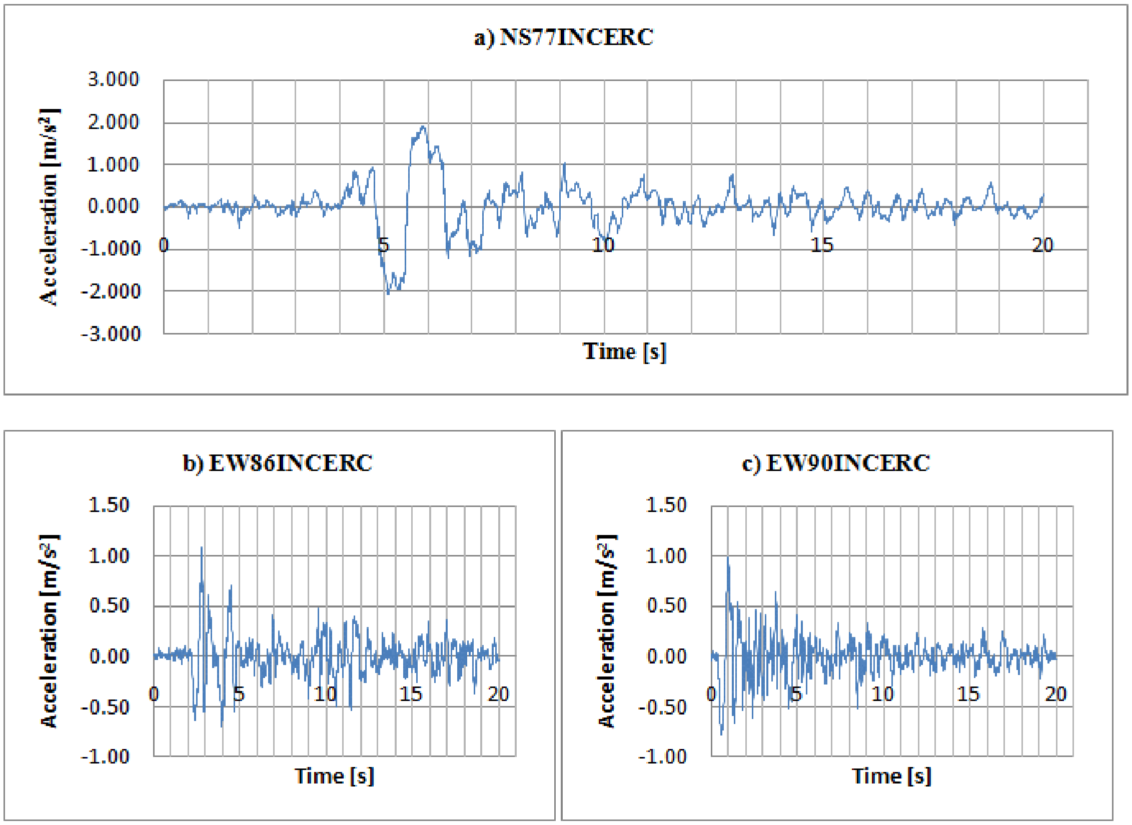

Figure 1.3.1. Recorded Accelerograms from Vrancea seismic area: a) 1977, N-S component, PGA=2.069 m/s^2a); b) 1986, E-W component, PGA= 1.091 m/s^2; c) 1990, E-W component, PGA= 0.989 m/s^2

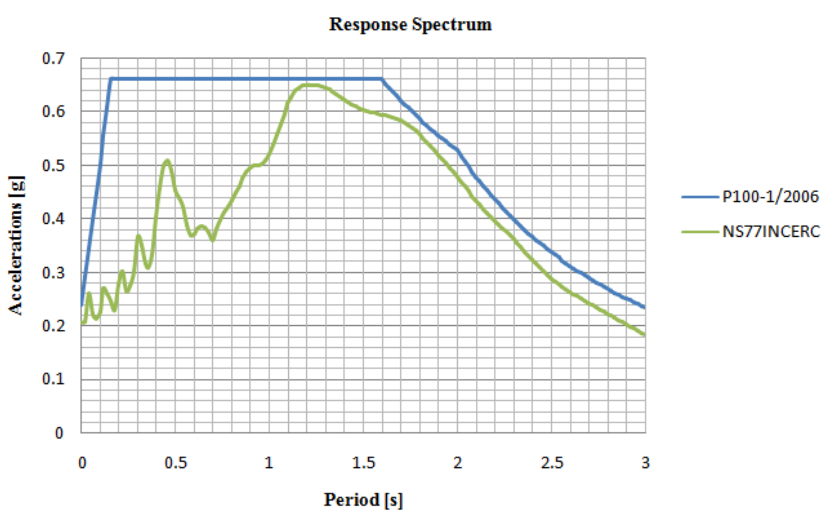

Figure 1.3.2 – Accelerations Response Spectrum

2. STRUCTURE MODELING

2.1. Materials

The materials used for reinforced concrete elements were concrete C30/37 and steel PC52 (fyd=300 N/mm^2, ultimate strain capacity 0.10). The stiffness of all the concrete elements was reduced to 50% by decreasing the value of the modulus of elasticity. Characteristics of C30/37 are according to Eurocode 2. For reinforcements the material was modeled as user defined with post elastic behavior like in **Fig.2.1.1**. For buckling restrained braces the steel used is S235.

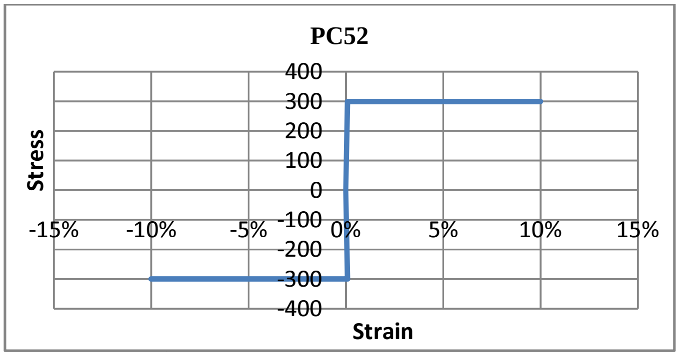

Figure 2.1.1 – PC52 Stress – Strain

2.2. Elements

Concrete columns and beams were modeled as frame elements. At the end of each structural element there were assigned plastic hinges that were deformation controlled, moment hinges and interacting axial-moment hinges modeled according to FEMA 356 and ACI 318-02 provisions. Walls were modeled as frame elements: mid-pear frame and rigid beam that allows a right connection between elements.
The buckling restrained braces (BRBs) were modeled in two ways: as jointed frames and links. For the designing of the braces there were used linear elements. When the section of the steel core was established the linear elements were replaced with multilinear plastic links. According to CSI Knowledge Base this type of element has energy-dissipation capacity, an advantage compared to plastic hinges.

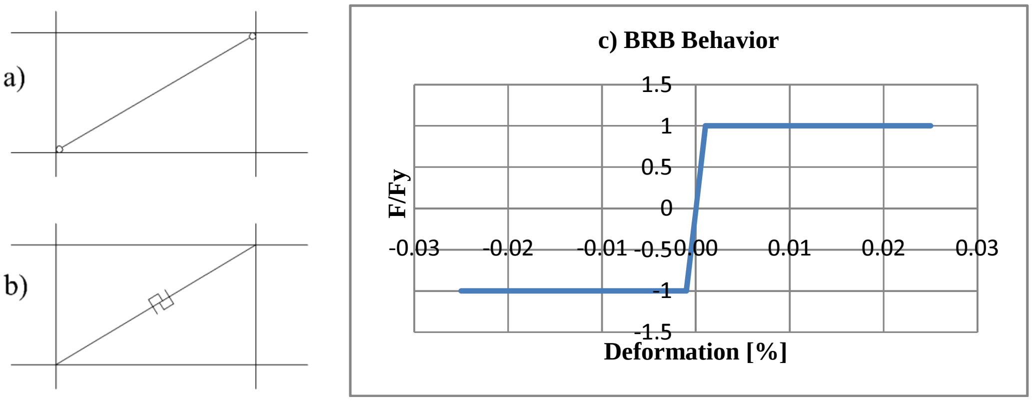

Figure 2.2.1 – Buckling restrain braces model: a) for design; b) for verification; c) BRB behavior

The most important advantage of using BRBs is that they have a very good behavior under compression, unlike the classic braces. Because of the casing that prevents buckling the compression

strength is often bigger than tension. This is reveled in published tests like “Type testing of buckling restrained braces according to EN 15129”.
In this paper the compression and tension strength are considered equal. Also the hardening of the BRBs elements is neglected in this study. The behavior of the buckling restrained frames is shown in **Fig. 2.2.1 c)**.

For the initial structure the seismic behavior factor was considered q=4.60 and for the structure with buckling restrained braces q=7. BRBs elements are around 5.50 m long and in computations the steel core that suffers plastic deformations was considered with a length of 3.0 m.

After analyzing the initial structure the places where BRBs will be placed were chosen. For design the braces it was used the equivalent static lateral forces method. The behavior factor was changed to q=7 and for this model the axial forces for the braces were computed. The designing was done using Eqn. 2.1.

$$A_{req} = \frac{N_{Ed} * \gamma_{M0}}{f_y} \tag{2.1}$$

A_{req} is the required area of the steel core of the brace, N_{Ed} is the axial force determined as told above, γ_{M0} is partial safety factor equal to 1.0 and f_y = 235 N/mm^2 is the strength capacity of steel S235. A thickness of t=20 mm was chosen for the steel core and the required width was obtained using Eqn. 2.2.

$$b_{req} = \frac{A_{req}}{t} \tag{2.2}$$

By using this method, area values from **Table 2.1** were computed.

Table 2.1. Effective area of the steel core of the buckling restrained braces.

t [mm]	20	20	20	20	20	20	20	20	20
b [mm]	100	90	80	70	60	50	40	30	20
A_{eff}[cm^2]	20	18	16	14	12	10	8	6	4

3. PERFORMANCE ASSESSMENT

In **Fig. 3.1** are presented the models for the two structures analyzed.

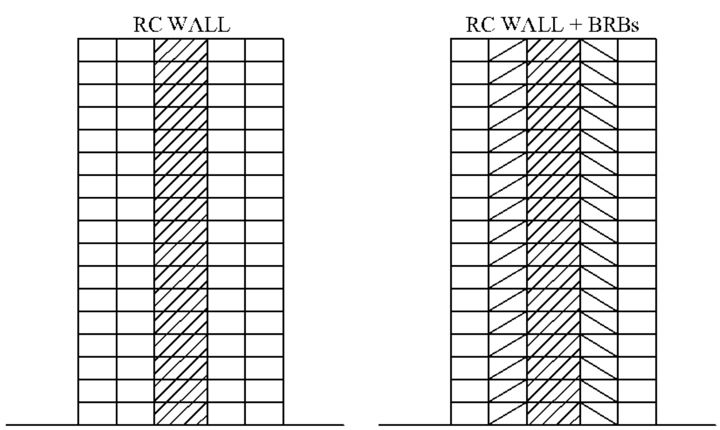

Figure 3.1 – Initial structure and retrofitted structure

In **Table 3.1** are presented the values of the first natural period of the structures. The retrofitted building has lower period due to the buckling restrained braces.

Table 3.1 First natural periods

Structure	Period [s]
RC Walls	1.12
RC Walls + BRB	0.92

The way plastic hinges appeared on the structure is different for the two models. In **Fig. 3.2** is presented the status of plastic hinges in both structures at the same time. The elements of the initial structure are starting to have plastic deformations very fast, earlier than the big peek of the accelerogram. In **Table 3.2** are presented the first plastic hinges that reached Collapse Prevention deformations. Acceptance criteria for this performance level (CP) according to FEMA 356 is 0.015 radians for beams and columns also.

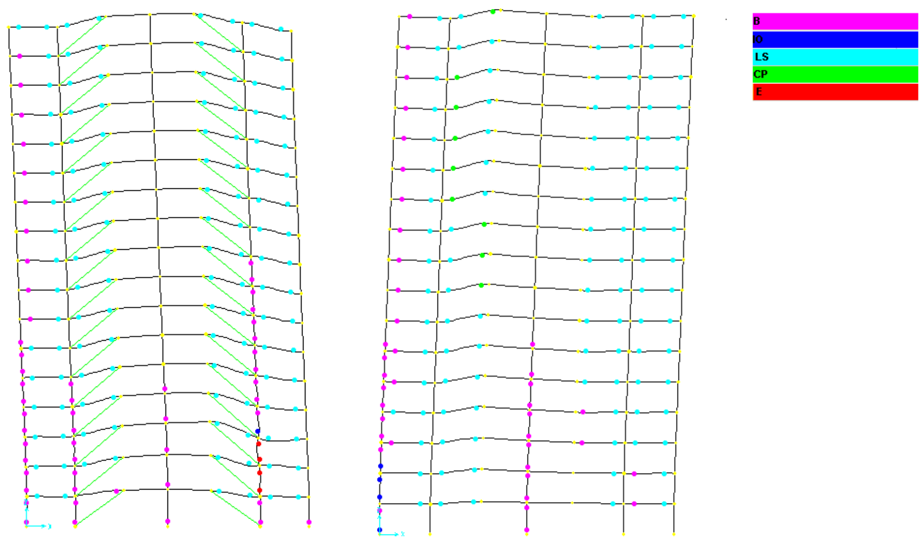

Figure 3.2 Plastic hinge status at the same time in RC WALLS + BRB and RC WALLS.

Table 3.2 First plastic hinge that reached Collapse Prevention deformation

Structure	First plastic hinge that reached CP [s]	Element
RC Walls	6.48	beam
RC Walls + BRB	7.04	column

At second 10 after the earthquake has started, more than 50% of the energy was introduced in the system. At that time the two models are compared in **Table 3.3** regarding the number of plastic hinges that reached Collapse Prevention deformation and the elements where they appeared.

Table 3.3 Number of plastic hinges that reached Collapse Prevention deformation after 10 seconds.

Structure	Number of plastic hinges that reached CP at time=10s	Most of them
RC Walls	14	beams
RC Walls + BRB	4	columns

Buckling restrained braces were added in the frames closer to the wall. In the initial model most of the beams from these frames had plastic deformations beyond their capacity and that was a criteria that lead to adding BRBs there. In the same time, the columns behavior was stable, with no plastic deformations. A comparison between the plastic deformations of the beams is presented in **Fig.3.3** where it can be seen that adding BRBs improved their behavior. Also, **Fig. 3.4** shows a decrease of the maximum moment recorded in beams.

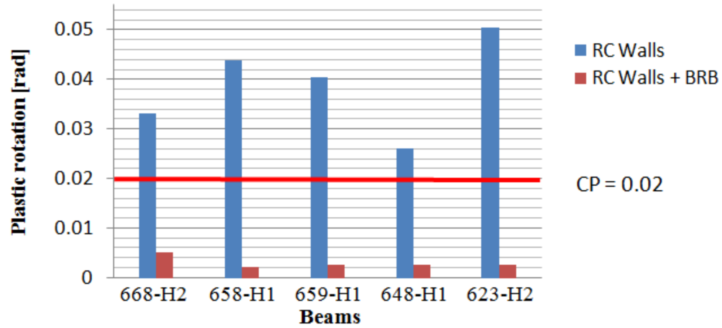

Figure 3.3 Comparison between plastic rotations before and after retrofitting with BRBs

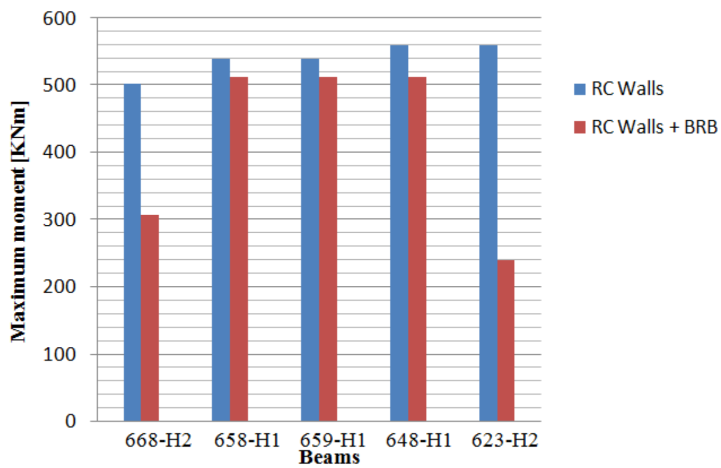

Figure 3.4 Comparison between maximum moments before and after retrofitting with BRBs

4. CONCLUDING REMARKS

In this paper an old designed building with reinforced concrete walls was studied for retrofitting using buckling restrained braces. A time history analysis was conducted both for the old and for the retrofitted structure, leading to some conclusions.

The retrofitted system had a good behavior in terms of strength and stiffness. During the analysis columns that were near the braces had plastic deformations but they didn’t reach their capacity. So a better response of the structure may be obtained if a local strengthening of the columns will be done.

In the initial system the beams had a poor behavior, especially the ones connected with the wall. After adding buckling restrained braces in that frames, beams presented a better behavior, plastic rotations decreased more than 10 times and the maximum moments recorded in plastic areas were reduced.

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