SEISMIC VULNERABILITY AND FRAGILITY OF STEEL BUILDINGS

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ABSTRACT:

The main goal of this article is to evaluate the structural behavior of steel buildings. The seismic performance of a specific steel building located in the campus of the National University of Colombia in Manizales is assessed. The seismic demand is characterized by a peak effective acceleration of 0.25 g. Pushover analysis applied to a low-rise steel moment-resitant frame building and a low-rise steel braced frame building has shown the inadequate seismic behaviour of moment-resistant frame buildings, althoug the increase of the stiffness of braced frame buildings may induce seismic damage to non-structural elements sensitive to spectral acceleration. We have obtained fragility curves and damage probability matrices for the seismic demand anticipated by the Colombian seismic code for Manizales. We obtain specific relationships between yielding and ultimate capacity spectral displacements and damage states' thresholds. These relationships can be applied to low-rise moment resistant frame and low-rise braced frame buildings. The comparison of the obtained expected mean damage states and the vulnerability classes provided by the EMS-98 macro-seismic scale leads us classifying low-rise steel braced frame buildings and low-rise steel moment resistant frame buildings respectively into the vulnerability classes C and D-E. We clearly show that, stiffening buildings with concentric braces leads to more earthquake-resistant structures, and therefore with a significantly lower seismic vulnerability.

KEYWORDS: vulnerability, fragility, moment resistant frame, braced frame, seismic damage.

1. INTRODUCTION

The main goal of this article is to evaluate the structural behavior of steel buildings designed and constructed according to the *"Load and Resistance Factor Design Specification for Structural Steel Buildings"* LRFD (AISC 1992). The seismic performance of two specific steel building is assessed. We also analyze their vulnerability. Our target building is located in the campus of the National University of Colombia in Manizales. This building was designed and built in the 2001 year, according to the 1998 Colombian Seismic Code, (NSR-98, AIS 1998). The design earthquake is characterized by a peak effective acceleration of 0.25 g. The seismic zoning of the city, used in this study, was performed after the construction of the building. The NSR-98 Code (AIS 1998) specifies two types of verifications: strength verification and deflection verification. In order to control strength, different load combinations are applied, taking into account vertical and seismic forces. In this case seismic loads must be decreased by applying a reduction factor, R. Deflection verification is performed by directly applying the seismic loads to the elastic model. In this case no reduction is applyied.

2. DESCRIPTION OF THE BUILDING

Figures 1, 2 and 3 show the geometrical features of the building. The floor plan is curved; having an average radius of about 114.5 m. Detailed cross-sections are plotted in Figures 2 and 3. The distance among cross frames ranges between 7.6 and 8.6 m. The building has 4 stories and its height is 11.94 m. The building structure is made of ASTM A-36 steel and it includes concrete slabs.

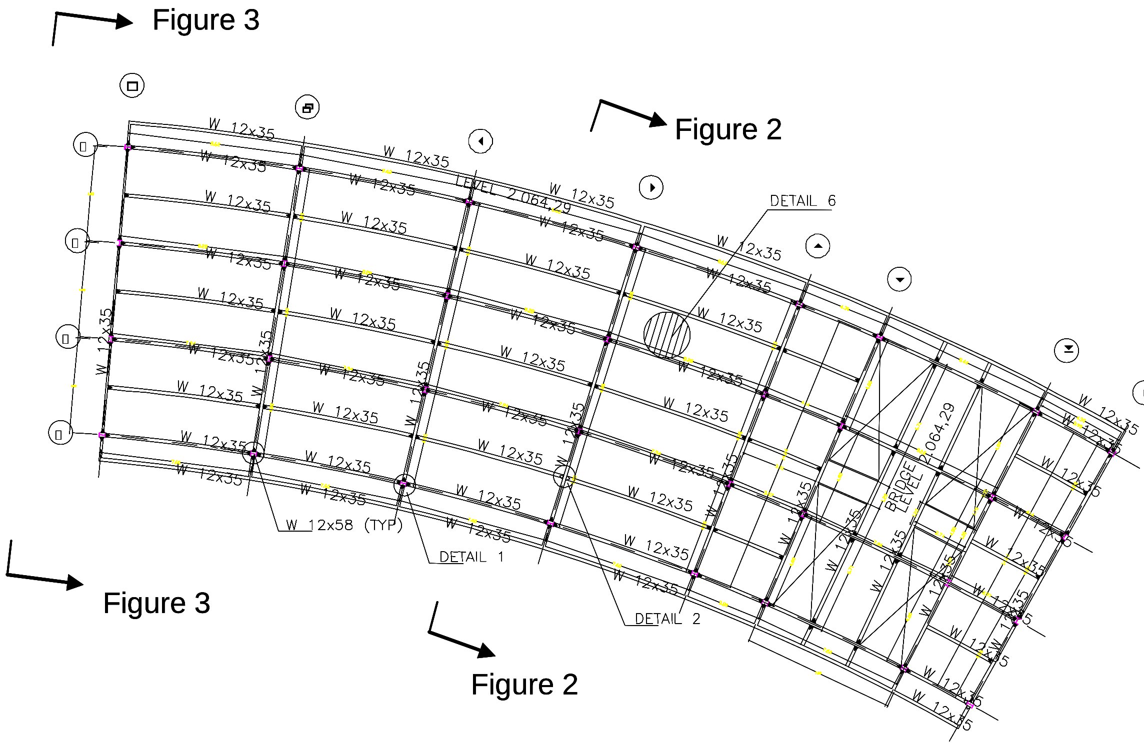


Figure 1. Floor plan of the building.

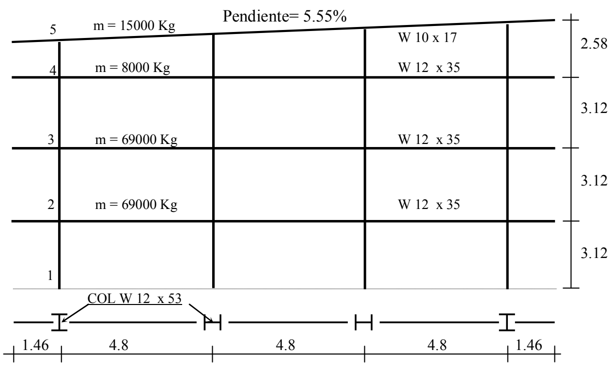


Figure 2. Moment-resisting frame.

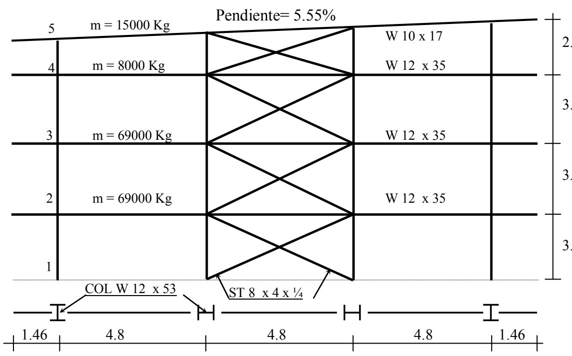


Figure 3. Braced frame

The total permanent overload per cross frame is 1569 kN. Its geometry, mass distribution and structural sections are shown in Figures 2 and 3. Structural shapes correspond to those described by the American Institute of Steel Construction (AISC). The weight of each braced frame, including joists and connection plates, is about 229 kN. The weight per constructed area is 545 N/m^2 since the building covers an area of 420 m^2. The weight per volume unit is 143 N/m^3. The gravity loads considered here are: 1) for slab beams 48.37 kN/m and 2) for roof beams 3.48 kN/m; both including permanent and occupancy loads. The vertical load applied in the pushover analysis was obtained by multiplying the dead load and the overload respectively by 1.2 and 0.5 factors.

3. STRUCTURAL BEHAVIOUR

In order to perform a non-linear study, the building was modeled by using RUAMOKO (Carr 2002) and ETABS (CSI 2005) computer programs. Plastic hinges were modeled after the *"Prestandard and Commentary for the Seismic Rehabilitation of Buildings"*, FEMA 356 (FEMA 2000). This standard includes the thresholds for the following values: "*Immediate Occupancy* " (IO), "*Life Safety*" (LS) and "*Collapse Prevention*" (CP). Figure 4 indicates the parameters defining the plastic behaviour, while Table 1 shows the values obtained for the members that are part of the frame studied, where θ_y represents the yielding rotation in radians.

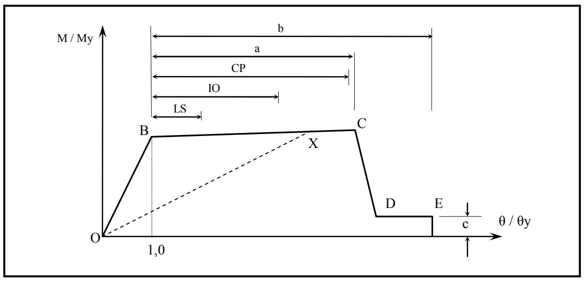


Figure 4. Layout of the moment-curvature ratio of a hinge.

Table 1. Model parameters and accepting criteria.

Profile and bending axis	a [rad]	b [rad]	c [ ]	IO [rad]	LS [rad]	CP [rad]
W 12x53, XX	**1.34 θ_y**	**2.03 θ_y**	**0.20**	**0.25 θ_y**	**0.77 θ_y**	**1.16 θ_y**
W 12x53, YY	**1.48 θ_y**	**2.25 θ_y**	**0.20**	**0.25 θ_y**	**0.87 θ_y**	**1.30 θ_y**
W 12x35, XX	**9.00 θ_y**	**11.00 θ_y**	**0.60**	**6.00 θ_y**	**8.00 θ_y**	**9.00 θ_y**

3.1. Bilinear capacity spectrum

Figure 5 shows the capacity spectrum together with its simplified bilinear representation.

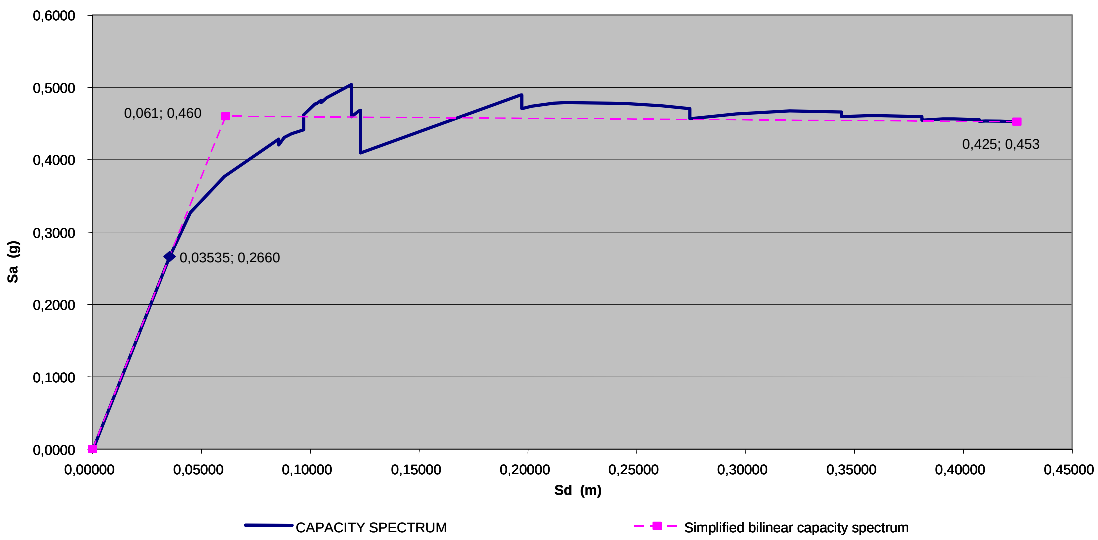


Figure 5. Capacity spectrum. The simplified bilinear representation is also shown.

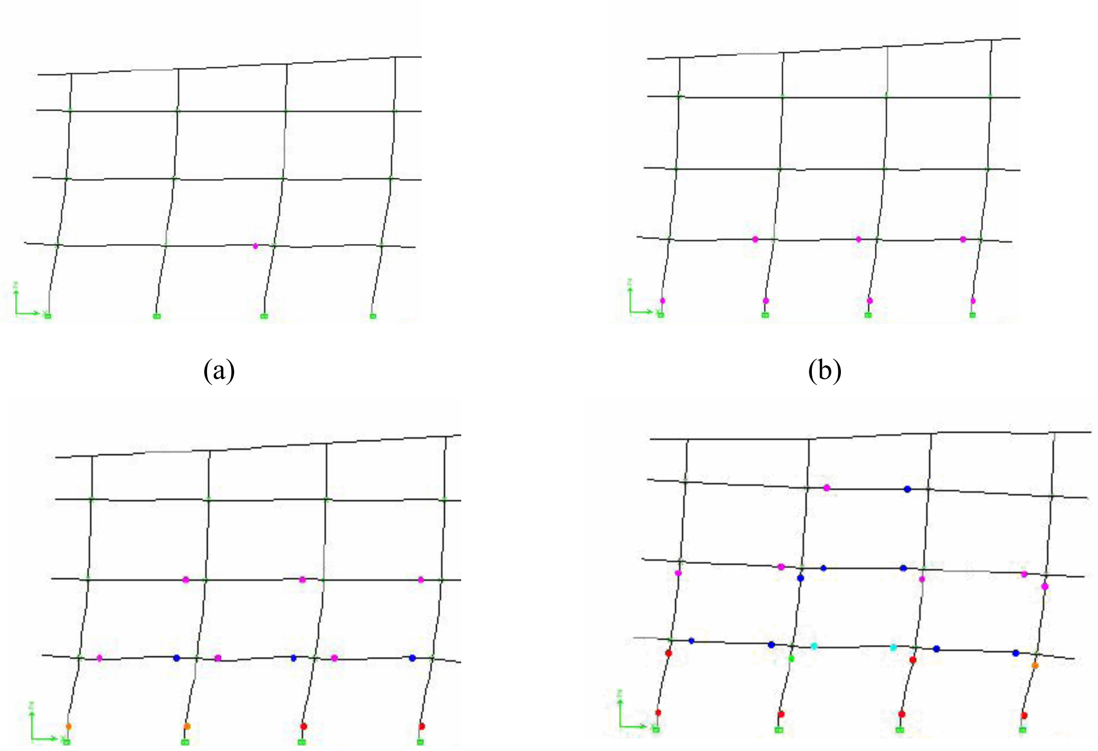
Ductility of the building is estimated as follows: $\mu = \delta_u / \delta_y = 0{,}425/0{,}061 = 6{,}97$ where δ_y and δ_u are, respectively, the yielding and ultimate spectral displacements. see, for instance, Barbat and Canet (1994). Note that ductility is higher than the energy dissipation coefficient considered in the project, R = 6.3. This coefficient is also frequently associated to the ductility.

3.2. Damage states thresholds

Four non-null damage states are considered. Taking into account that damage follows a binomial type distribution, to elaborate the fragility curves, firstly the different damage states thresholds should be determined, and then the log normal standard deviation for each of the curves. An analysis of the structure's yielding course will determine the thresholds for each damage state.

3.2.1. Slight damage state

Figure 6a shows the first yielding hinge appearing in the loading process, which results in a total drift of 0.05 m, a spectral displacement of 0.037 m and a maximum ground floor inter story drift of 0.66%. Given that, till that moment, the structure behaviour is linear, a slight damage threshold of Sd_L = 0.037 m is adopted. As shown in Figure 5, the yielding spectral displacement is Sd_Y = 0.061 m and, thus Sd_L can be expressed in terms of Sd_Y as Sd_L = 0,607 Sd_Y.



(a) (b)

(c) (d)

Figure 6. First yielding hinges appearing in the loading process.

3.2.2. Moderate damage state

Figure 6b shows a significant number of hinges. In fact, all the base columns in the ground floor yielded. This situation arises when building total drift equals 0.086 m, the spectral displacement is 0.065 m, and the maximum inter-story drift, which occurs in the ground floor, is 1.24 %. This spectral displacement is taken as the moderate damage state threshold (Sd_M).

In terms of the yielding spectral displacement, Sd_Y, Sd_M can be expressed as $Sd_M = 1.066\ Sd_Y$.

3.2.3. Severe damage state

Figure 5 shows a drastic decay of the strength of the building. For a spectral displacement of about 0.13 m, spectral acceleration decays from about 0.5 g to about 0.4 g. This situation matches the degradation observed in Figure 6c. This spectral displacement is assumed for the severe damage state threshold. The corresponding deformation parameters are as follows: building total drift is 0.17 m, spectral displacement is 0.13 m and maximum inter story drift, arising in the ground floor, is 2.67 %. In terms of yielding spectral displacement, Sd_Y, and ultimate spectral displacement, Sd_U, the severe damage state threshold, Sd_S, can be expressed as $Sd_S = Sd_Y + 0{,}184\ (Sd_U - Sd_Y)$.

3.2.4. Collapse damage state

A building displacement of 0.60 m, a spectral displacement of 0.437 m and an inter story drift of 9.19%.are related to the collapse damage state, The yielding state is shown in Figure 6d.

3.3 Fragility curves

The information obtained in section 3.2 allows plotting the fragility curves shown in Figure 7. In this Figure, the obtained fragility curves are compared with those provided by the *Earthquake Loss Estimation Methodology HAZUS 99* (FEMA 1999) for low-rise steel building with moment-resistant frames (S1L type) and medium level seismic protection codes.

3.4. Performance point

Crossing capacity and demand spectra, properly reduced to account for hysteretic damping, allows obtaining the performance point. We use the guidelines of the *Seismic Evaluation and Retrofit of Concrete Buildings ATC-40* (ATC 1996b), and the method recommended in project RISK-UE (Milutinovic & Trendafiloski 2003).The later is based on the bilinear representation of the capacity spectrum and is well described in the SISman document (ITEC 2004). The obtained performance point is (S_d=0.06638 m, S_a= 0.385 g) (see Figure 8).

3.5Damage probability Matrix

For a given seismic demand, fragility curves allows obtaining the expected damage distribution of damage, this is the so called damage probability matrix (Barbat et al. 2006). The spectral displacement for the design seismic demand is 0.07 m and the corresponding damage probability matrix is (0.12 0.33 0.34 0.17 0.04).

4. DISCUSSION

4.1 Vulnerability and fragility

The probability of exceedance of the moderate damage state is about 55%. This situation is not suitable, because as shown in Figure 6b), at the spectral displacement corresponding to moderate state threshold, the piles in the first floor yield at their bases. For the obtained performance point, the expected local inter-story drift index in the ground floor is 1.36 % and extensive damage would take place in the non-structural elements sensitive to drift. This analysis clearly indicates that this frame should be stiffened. A braced frame building has been also analyzed. We have applied the same procedures described above to obtain fragility curves and damage probability matrices. Figure 9 shows the obtained fragility curves. For the same seismic demand, the expected damage probability matrix is [0.75 0.15 0.06 0.04 0.00]. For the so stiffened building, the probability of exceedance of the moderate damage state is only 10%.

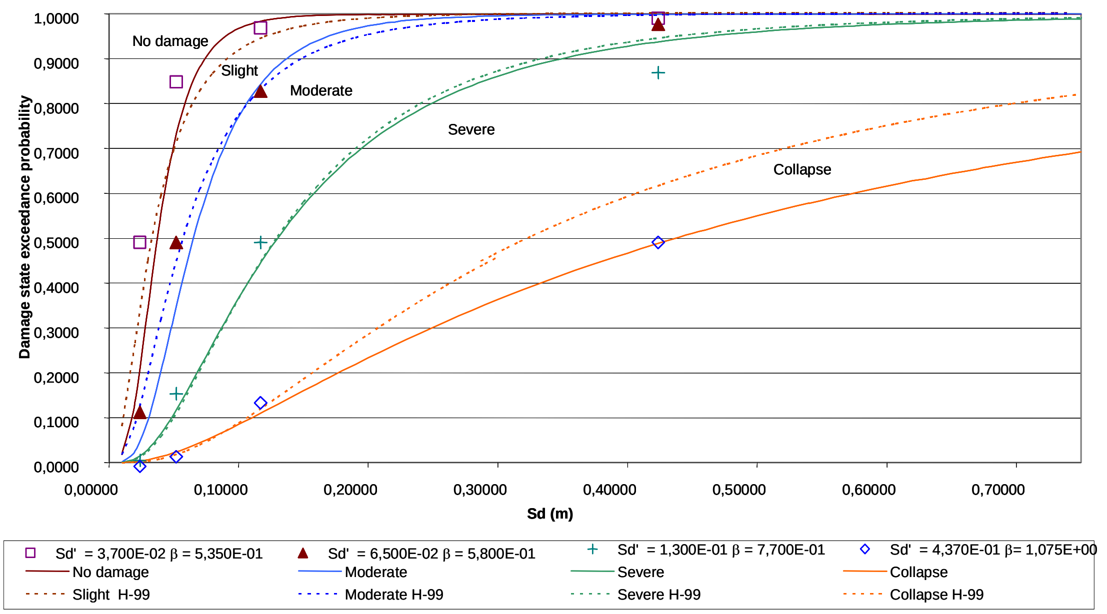


Figure 7. Fragility curves obtained for steel moment-resistant frames and from HAZUS 99

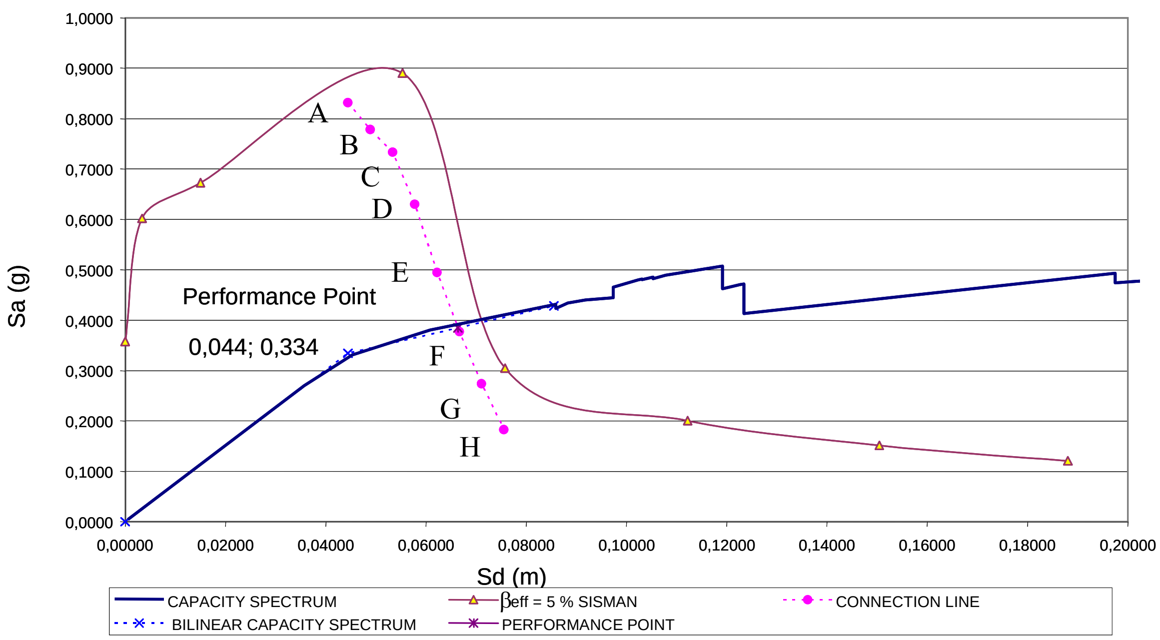


Figure 8. Obtaining of the demand capacity point of the moment-resisting frame.

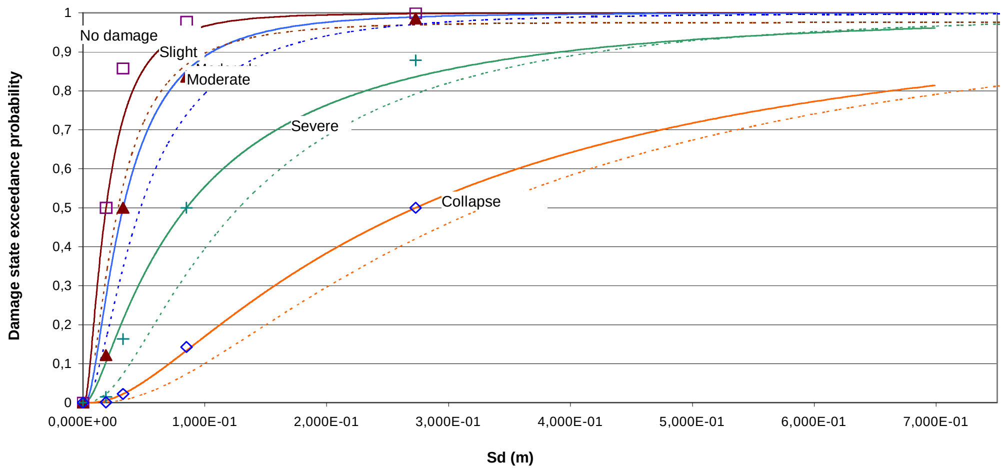


Figure 9. Fragility curves obtained for braced frames and from HAZUS 99

4.2 EMS-98 vulnerability classification

We use the obtained damage probability matrices to classify these structures within EMS-98 (Grünthal 1998). The seismic intensity is calculated by using the well-known Murphy & O'Brien equation ($\log_{10} \mathrm{PGA} = 0.25\mathrm{I} + 0.25$). A PGA value of 0.25 g is assumed and we obtain a 8.56 intensity. To estimate the corresponding vulnerability index we use the following equation (Milutinovic & Trendafiloski, 2003):

$$\mu_D = 2.5\left[1 + \tanh\left(\frac{I + 6.25V_I - 13.1}{2.3}\right)\right] \tag{4.1}$$

where I is the seismic intensity, μ_D is the expected damage state and V_I is the vulnerability index. In this case we take the following values: μ_D =1.69, I=8.56 for the moment resistant frame building and μ_D =0.56, I=8.56 for the braced frame building. We respectively obtain V_I values of 0.60 and 0.35 which correspond to the EMS'98 vulnerability classes C and D-E.

5. CONCLUSIONS

Braced frame structures fulfil the *LRFD Specification* (AISC 1992) requirements. Nevertheles, althoug these requirement limiting the elastic deformations may lead to a rigid building with low expected structural damage, the effect of this stiffening on the non-structural members' behaviour, which are sensitive to acceleration, should be analysed, because, in the cases here analyzed, the decrease of the performance spectral displacement involves an increase of its spectral acceleration.

This work has allowed obtaining specific relationship between yielding and ultimate capacity spectral displacements and damage states' thresholds. More case studies are needed to confirm the obtained relationships. For comparison purposes Table 2 shows our results. Our feeling is that these results can be applied to low-rise moment resistant frame buildings. Any way it is worth noting that the thresholds here obtained are similar to those provided by the HAZUS-99 program (FEMA 1999) for low-rise moment-resistant frame buildings (S1L) and

also, although at a lesser extent, for low-rise braced frame buildings (S2L).
The comparison of the obtained expected mean damage states and the EMS-98 macro-seismic scale leads us classifying the studied buildings into the vulnerability classes C and D-E, respectively for low-rise braced frame buildings and for low-rise moment resistant frame buildings. We have clearly shown that, stiffening buildings with concentric braces leads to more earthquake-resistant structures, and therefore with a significantly lower seismic vulnerability.

Table 2. Proposed damage state thresholds for low-rise steel buildings.

Damage state	Spectral displacement Sd
Slight	$\mathbf{Sd_L = 0.60\ Sd_Y}$
Moderate	$\mathbf{Sd_M = 1.10\ Sd_Y}$
Severe	$\mathbf{Sd_S = Sd_Y + 0{,}20\ (Sd_U - Sd_Y)}$
Collapse	$\mathbf{Sd_C = Sd_U}$

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