Fragility analysis of a soft first story building rehabilitated with buckling restrained braces

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ABSTRACT

Several soft first story buildings were damaged during the September 19, 2017, Mexico (SEPT/19/2017-MEX) earthquake. Here, both the structural behavior and the seismic fragility of a typical 5-story building with soft first story (SFS) are analyzed. The building, located in Mexico City, has regular R/C moment-resisting frames in the soft story, and confined masonry walls are found in the upper levels. Several buildings that collapsed during the SEPT/19/2017-MEX earthquake had a similar structure. The following three structural cases are analyzed: S1 corresponds to the original structure with SFS; S2 is the case in which concrete jacketing is used in the columns of the SFS and the masonry walls are retrofitted with steel horizontal reinforcement; for case S3, steel jacketing and buckling-restrained braces (BRB) are added to the SFS and the masonry walls are retrofitted with steel horizontal reinforcement. Ten seismic ground motions are used in the analysis. The maximum story drift (MSD) profiles for the three buildings subject to the ten seismic ground motions with different intensities are used to analyze the behavior of the structural systems. Fragility curves for limit values of MSD recommended by the Mexico City Building Code (2017) are obtained. It is shown that the building rehabilitated with BRBs and steel jacketing (Case S3) presents fragility curves similar to those of the building with concrete jacketing (Case S2).

Keywords: Fragility curves, soft first story buildings, structural rehabilitation, buckling restrained braces, BRB devices

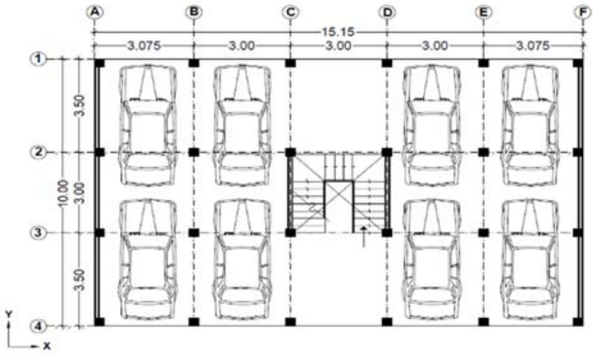
INTRODUCTION

The construction of buildings with SFS is very common, both in mega-cities and in urban areas with limited space. Mexico City is not the exception. It has many buildings whose first story is used as a lobby, commercial area, or parking lot [1-3]. During the SEPT/19/2017-MEX earthquake, the percentage of collapsed buildings due to SFS was approximately 35%. Attention is drawn to such high percentage; it generates the need to study both the behavior and the fragility of buildings with SFS. Even though many buildings with SFS did not collapse during the SEPT/19/2017-MEX earthquake, it is necessary to review the safety of this type of buildings, and rehabilitate them if necessary, in order to provide them with enough seismic structural reliability, thus assuring their good structural behavior in future seismic events.

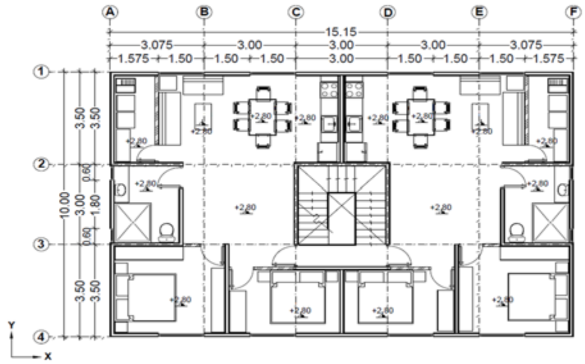
This study shows the similarity of reliability levels obtained through the fragility evaluation of a typical 5-story building with SFS, rehabilitated with the two following techniques: 1) reinforced concrete jacketing of the columns of the SFS, and 2) steel jacketing of the columns of the SFS, plus the inclusion of BRBs.

CASE STUDIES

A 5-story building rehabilitated with two different techniques is analyzed. The structure has both geometry and material properties similar to the collapsed buildings during the SEPT/19/2017-MEX earthquake. The building has a rectangular plan, and presents irregularity in its height, characterized by a change of stiffness story from the first to the second story. Figures 1 and 2 show the plan (first and second stories), and the height of the building, respectively.



a) First story (FS)



b) Second story

Figure 1. Plan distribution of structural elements.

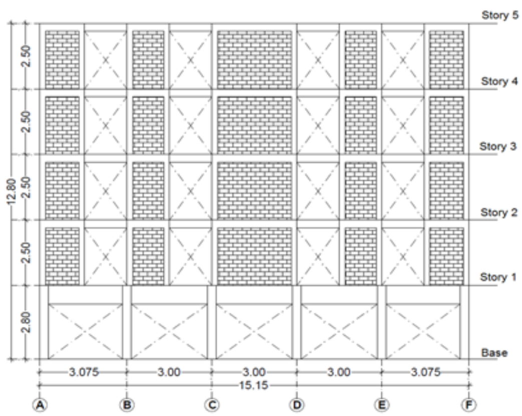


Figure 2. Height distribution of structural elements.

Table 1. Dimensions of structural elements.

Element	Dimensions (m)	Level
Columns (first story)	0.30 x 0.30	FS-S1
Beams (first story)	0.30 x 0.70	S1
Beams (upper levels)	0.12x0.30	S2-S5
Masonry walls	0.12x (1.5 to 3.5)	S2-S5

The original building was designed according to The Mexico City Building Code – Seismic Design (NTC-S2004), The Mexico City Building Code – Concrete Structures Design (NTC-C2004), and The Mexico City Building Code – Masonry Structures Design (NTC-M2004) [4]. The structure has regular R/C moment-resisting frames in the soft story and confined masonry walls in the rest of the levels. The concrete frames were modeled using bar elements, and masonry walls were modeled with the wide column method, assuming shear failure. The structure is considered to be highly irregular. The cross-sections of structural elements of the original building (S1) are shown in Table 1.

The general characteristics of the three cases of interest (original building and two rehabilitated buildings) are as follows:

1. **Case S1.** Corresponds to the original building designed according to NTC-S2004.
2. **Case S2.** Corresponds to case S1 after rehabilitation, using the technique of column jacketing. Design was performed according to NTC-S2017. The rehabilitated building is not classified as SFS when definitions of NTC-S2017 [5] are applied. The spectrum used for the design of the rehabilitation is shown with a dotted line in Figure 3. Such spectrum was obtained with the following parameters: factor of seismic behavior Q = 2, over-resistance factor R_0 = 2, and an irregularity factor of 0.8. Table 2 shows the cross-sections of columns and beams corresponding to the first story of the building.

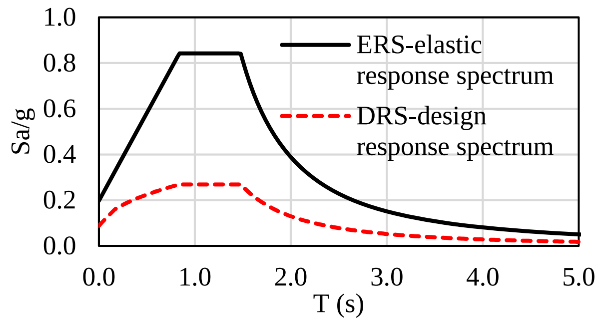


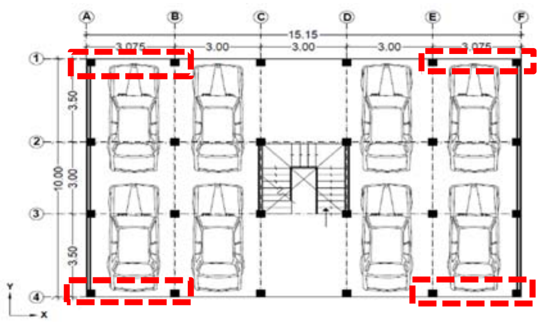
Figure 3. Design response spectrum.

Table 2. Dimensions of columns and beams of the first story for case S2.

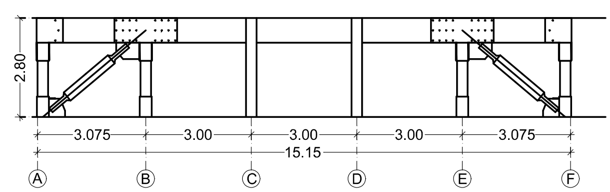
Element	Dimensions of columns (m)	Level
Columns first story	0.45 x 0.45	FS-S1
Beams first story	0.30 x 0.70	S1

3. **Case S3**. Corresponds to case S1 after including BRBs and steel jacketing of columns of the first story. The design of reinforcement was done so that both the stiffness and the lateral resistance were similar to the case S2 building. First story columns were jacketed with 2"W-3/16"T, flat ASTM A-36 structural steel bars with f_y = 2,530 kg/cm^2. The location and orientation of the BRBs are shown with dotted lines in Figures 4a and 4b. Their dimensions are shown in Table 3. The design spectrum used for the rehabilitation is similar to the spectrum shown on Figure 3, except for the fact that for case S3 an irregularity factor of 0.7 was used instead of 0.8 according to Mexico City Building Code 2017.

The fundamental periods of vibration of the cases studied are shown in Table 4. The details of the design and reinforcement of elements for cases S1, S2 and S3 can be found in [6].



a) Plan location of BRBs



b) Elevation configuration of first story

Figure 4. Location of buckling-restrained braces. Case S3.

Table 3. Dimensions of BRBs. Case S3.

Element	Area of core (m^2)	Length (m)
CPR-1	0.0018	3.36

Table 4. Periods of vibration.

Case	Period (s)
S1	0.42
S2	0.31
S3	0.32

SELECTION AND SCALING OF SEISMIC GROUND MOTIONS

Ten accelerograms corresponding to ground motions with a dominant period close to 1.0 s recorded on transition soil in Mexico City during the SEPT/19/2017-MEX earthquake [7] were selected with the objective of performing the fragility analysis of cases S1, S2 and S3. Figure 5 shows the corresponding elastic pseudo-acceleration spectra of the seismic records for a critical damping percentage of ζ = 5%. Signals were scaled with the intensity based on the average of the spectral ordinates Sa_{ave} [8] in an averaging interval between $0.2T_e$ and $1.3T_e$, according to NTC-S2017 specifications, where T_e is the fundamental period of the structure.

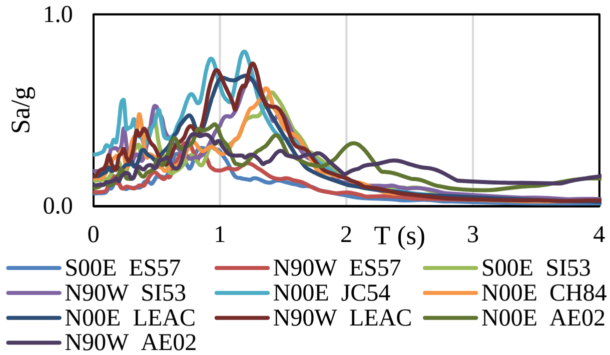


Figure 5. Elastic pseudo-acceleration response spectra.

FRAGILITY CURVES

The fragility function is the conditional probability of either equaling or exceeding a determinate value of structural demand, given an intensity value. Such function is as follows:

$$P(D > d|IM = im) = 1 - \Phi\left(\frac{\ln(y/D)}{\sigma_{lnD}}\right) \quad (1)$$

where:

$P(D > d|IM = im)$ is the conditional probability of exceeding a value *d*, given an intensity *IM* = *im*

Φ is the normalized cumulative Gaussian distribution

$\ln(y/D)$ is the natural logarithm of the result of dividing the intensity *y* by the median of the structural demand *D*

σ_{lnD} is the standard deviation of the natural logarithm of the structural demand

The results are presented in terms of fragility curves, which allow the calculation of the exceedance probability of a given value of MSD, with an average pseudo-acceleration level Sa_{ave}. The structural demand corresponds to the MSD of all stories, which is obtained from non-linear dynamic analyses, corresponding to the ten ground motions indicated in Figure 5. It is assumed that the distribution function of the structural demand is lognormal type [9].

STRUCTURE MODELING

The non-linear analyses were performed with Ruaumoko3D software [10], generating a tridimensional model of bar-type elements for beams and columns; walls were modeled using spring type elements. In order to represent the non-linear behavior, models of concentrated plasticity were used in both cases. The main considerations for the non-linear modeling are: a) interaction of axial load and flexure in both directions is considered for columns; b) for beams, only flexure on the main axis is considered; c) the modified Takeda model was used in both cases; d) the non-linear behavior of the masonry walls was located at the center of wide columns with a predominant shear failure; e) a trilinear model was used for modeling the degradation of both stiffness and resistance of the masonry walls. Such model was calibrated using information of cycling tests performed in Mexico [11]; f) a bilinear model was used for the BRBs behavior.

RESULTS

The results of the incremental dynamic analyses (IDAs) are expressed in terms of capacity [12], which are useful for determining both the intensity that generates a certain distortion level in the structural system, and the maximum capacity of it. In order to develop IDA curves, incremental dynamic analyses were performed for the ten ground motions selected, considering an interval of average spectral intensities Sa_{ave} between 0.1 and 0.8g, with increments of 0.01g.

There are several approaches to determine the capacity of ultimate deformation of structural systems through a detailed analysis of the IDA curve [12,13]; in this paper, it is assumed that the maximum deformations that structural systems can hold are the maximum permissible drift ($\gamma_{máx}$) specified in Tables 4.2.1 and 4.2.3 of NTC-S2017. An evaluation of the capacity of the three cases studied is done below.

1. Structural behavior of case S1 under different seismic intensities

Figure 6 shows the capacity curves of building S1; the thick line shows the average of the ten analyses; the gray lines show the results of each IDA. The thick line shows that maximum drift $\gamma_{máx}$ = 0.015 is reached at an approximate intensity (Sa/g) of 0.35 (dotted line). It can also be seen that after such value, maximum drifts grow significantly, but there is not a substantial increment in the value of the intensity. It is necessary to point out that, according to NTC-S2017, a structure of low-ductility reinforced concrete frames has a maximum permissible value $\gamma_{máx}$=0.015.

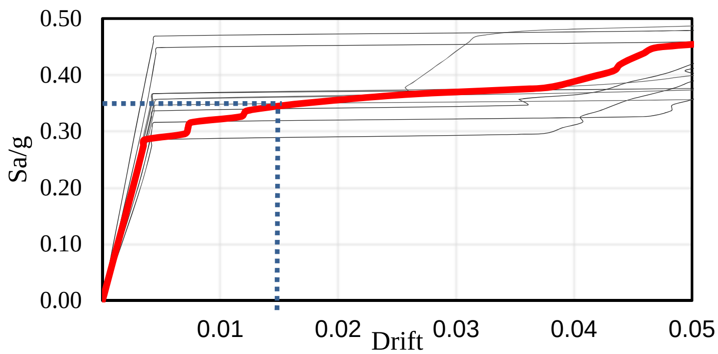


Figure 6. IDA curves corresponding to case S1.

Figures 7a and 7b show the MSDs of case S1 for intensities of 0.3g and 0.34g respectively. The red dotted line shows the average MSD corresponding to the ten dynamic analyses; the gray lines show the results of each analysis. Figures 7a shows that, for low intensities, the greatest maximum drifts are concentrated in the first story; when intensity increases, there is not a substantial increase in the maximum drifts of the upper stories. In fact, such drifts remain below 0.15%. On the other hand, Figure 7b shows that for an intensity of 0.34g, an average maximum drift γ = 0.015 is reached, which corresponds to the maximum allowable distortion for ordinary concrete moment frames according to NTC-S2017. Figure 7b shows that the maximum drift of the first story corresponds to the point in the capacity curve intersected by the dotted lines in Figure 6, which indicates the condition for the displacements to grow significantly, without a relevant increase in intensity.

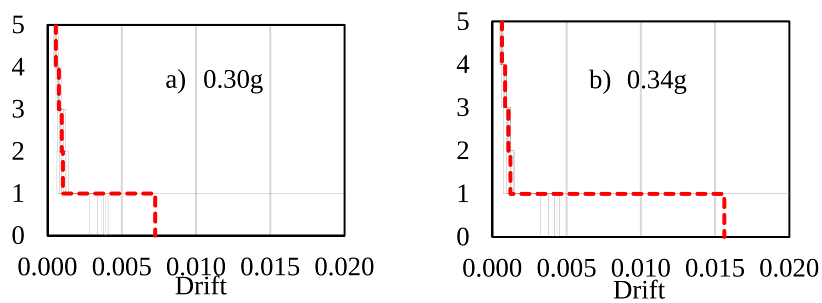


Figure 7. MSDs for case S1, for two values of intensity.

2. Structural behavior of cases S2 and S3 under different seismic intensities

IDA curves for cases S2 and S3 were generated, similarly to what was done in case S1. Figures 8 and 9 show the IDA curves for cases S2 and S3, respectively, corresponding to the maximum drift of all stories of the rehabilitated buildings. The cases S2 and S3 have a similar structural capacity, because the design was performed with the objective of getting both similar global resistance and equivalent stiffness in both cases.

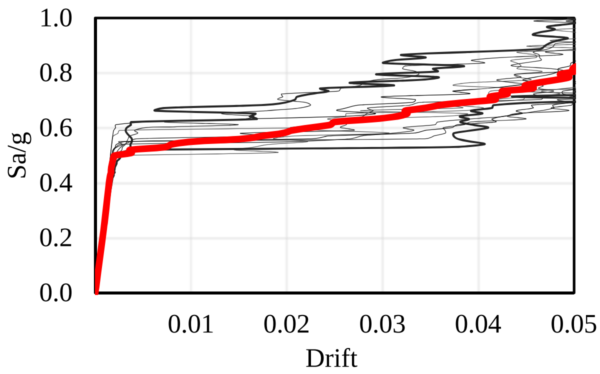


Figure 8. IDA curves corresponding to case S2.

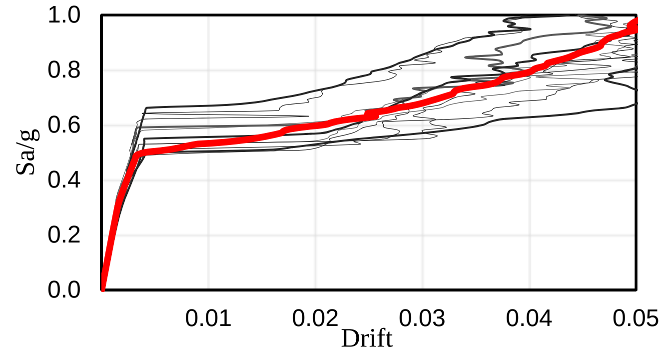


Figure 9. IDA curves corresponding to case S3.

Figures 10 to 12 show both the MSDs corresponding to different intensities and the behavior of the three buildings under study: a) case S1, b) case S2, c) case S3. Part *c* of such figures also show the hysteresis cycle of BRBs (in units of kN, and meters) associated with the seismic ground motion that originates the MSDs close to the maximum average story drift.

Figure 10, corresponding to an intensity of 0.3g shows that cases S2 and S3 have average MSDs close to 0.001; however, MSDs for case S1 (figure 10a) increase more than twice in the first story. Figure 10c shows that the inelastic behavior of the BRB just starts in this intensity (Sa/g = 0.3).

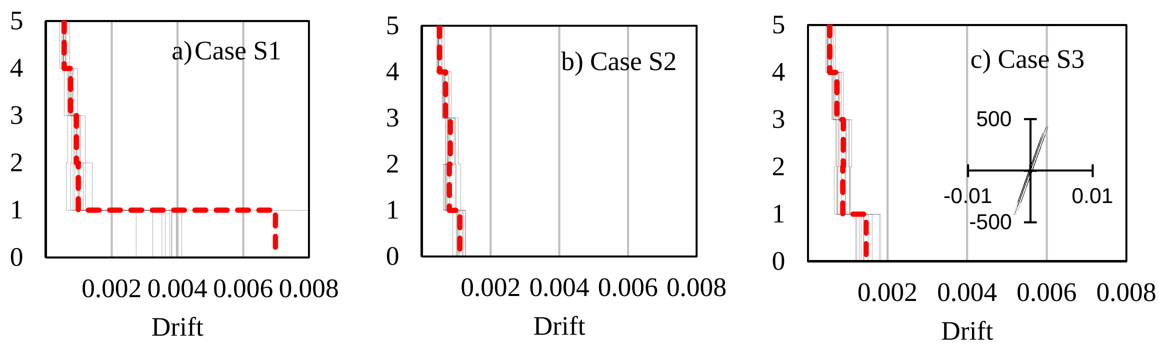


Figure 10. Drift profiles for Sa/g = 0.30; hysteresis cycle in kN and m.

Figure 11a shows that, for an intensity Sa/g = 0.41, the average MSD for case S1 is 0.043, which is 2.8 times greater than the maximum permissible limit according to NTC-S2017. On the other hand, figures 11b and 11c show that the average MSD corresponding to models S2 and S3 are below 0.003. It is observed that, for case S2 (figure 11b), the MSDs of the second story for some seismic motions are greater than the MSDs for the first story, which means that forces are re-distributed in the upper levels. Figure 11c shows the bi-linear behavior of one of the BRBs.

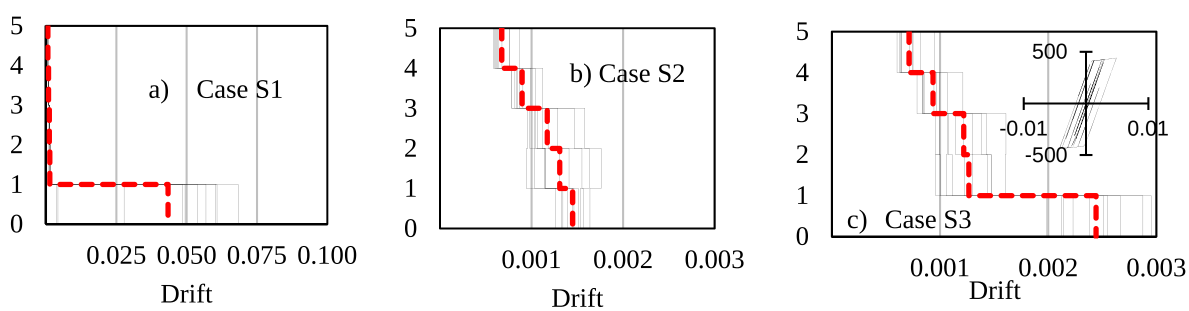


Figure 11. Drift profiles for Sa/g = 0.41; hysteresis cycle in kN and m.

Figure 12 shows the drift profiles for an intensity of 0.53g. Figure 12b shows that the average MSDs of the second story (dotted line) are below 0.01 (maximum permissible design value according to NTC-S2017), whereas for building S3 (figure 12c), the average MSD of the second story is 0.010, which means that the walls located between the first and second story developed the MSD allowed by the Mexican Code for confined masonry walls with horizontal reinforcement ($\gamma_{máx}$ = 0.010). Such behavior corresponds with what is shown by the dotted line in the IDA curve of figure 9. Also, figure 12c shows that the local ductility of BRB is approximately 3.

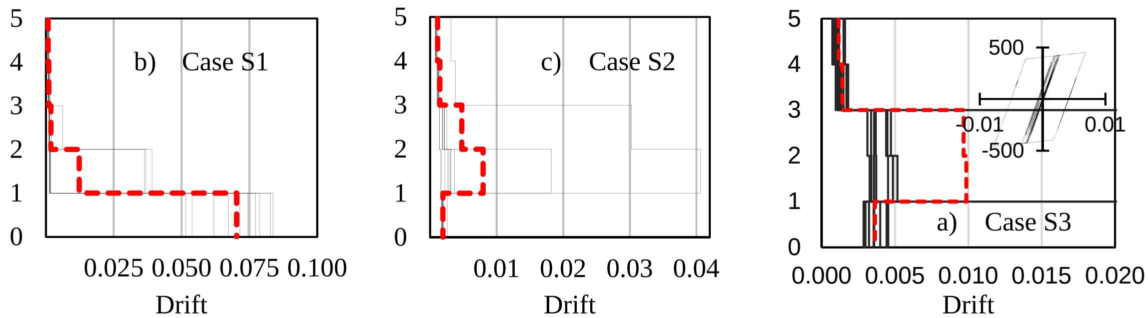


Figure 12. Drift profiles for Sa/g = 0.53; hysteresis cycle in kN and m.

Fragility curves

Figure 13 shows the fragility curves for cases S1, S2 and S3 for the allowable MSDs associated with the collapse prevention limit state according to NTC-S2017. Such values are equal to 0.015 for ordinary concrete moment frames (first story), and 0.01 for masonry walls with horizontal reinforcement.

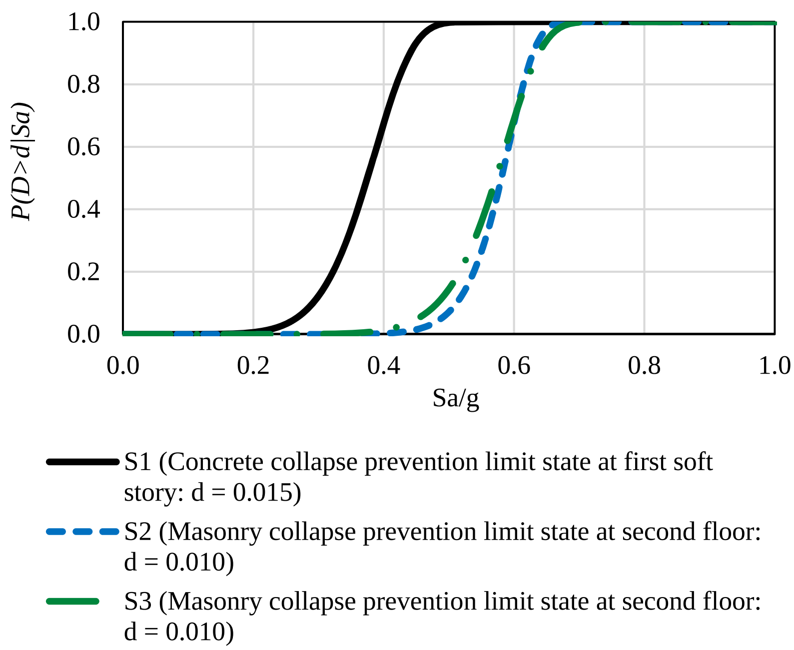


Figure 13. Fragility curves for collapse prevention limit state. Cases S1, S2 and S3.

In Figure 13 can be observed that case S3 (building with steel jacketing and BRB) presents a fragility similar to case S2 (building with concrete jacketing) corresponding to the collapse prevention limit state, when the MSD is considered as a parameter of the structural demand.

CONCLUSIONS

Both the behavior and the seismic fragility of three structural systems were analyzed: S1, original building with SFS; S2, building with rehabilitated columns of the first story with the technique of reinforced concrete jacketing; S3, building with rehabilitated columns of the first story with the technique of steel jacketing plus BRBs.

It was verified that both rehabilitation alternatives S2 (column jacketing with reinforced concrete) and S3 (column jacketing with steel plus BRBs) present similar seismic fragility curves.

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