Developing Fragility Curves for Steel Building with X-Bracing by Nonlinear Time History Analyses

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SUMMARY:
Regarding the importance of fragility curves in seismic evaluation of buildings, in this study the fragility curves were developed for some sets of steel buildings, having frames with X-bracing as their load bearing system, by performing a series of Nonlinear Time History Analyses. The buildings, which were assumed to be regular in both plan and elevation to avoid the torsion effects, include a set of 2- by 4-bay and another set of 4- by 6-bay plans having 3, 5, or 7 stories. Several recorded accelerograms, with various peak ground acceleration values and also different frequency contents, were used for each building to create a relatively large statistical data for developing reliable fragility curves. Two damage indices, including the “inter-story drifts” and the “axial plastic deformation of bracing elements” were used. The numerical results show that the second index is more reliable for developing the fragility curves of braced frames.

Keywords: Steel frames with X-bracing, Inter-story drifts, Axial plastic deformation

1. INTRODUCTION

Damage estimation is one the main prerequisites for all ‘earthquake risk reduction programs’ in any earthquake prone country. However, there are very few data with regard to the relation between level of damage in various buildings and the earthquake characteristics. Most of the developed fragility curves are based on the actual data gained from the damaged buildings in past earthquakes, which do not necessarily include all types of the existing buildings in various countries. Therefore, developing fragility curves by computational methods is of great importance, especially for those areas which suffer from the lack of rich earthquake data banks. Up to now, many researchers have tried to develop the fragility curves for the existing buildings by using the real data or various types of analyses, including Push Over or Nonlinear Time History.

Recently, Kircil and Polat (2006) have developed fragility curves for mid-rise R/C frame buildings in Istanbul, which have been designed according to the 1975 version of the Turkish seismic design code, based on numerical simulation with respect to the number of stories of the buildings. They have designed sample 3-, 5- and 7-story buildings, and then have performed incremental dynamic analyses for sample buildings using twelve artificial ground motions to determine the yielding and collapse capacity of each sample building. Based on the buildings’ capacities, fragility curves have been developed in terms of elastic pseudo spectral acceleration, peak ground acceleration (PGA) and elastic spectral displacement for yielding and collapse damage levels with lognormal distribution assumption. To investigate the effect of number of stories of the building on fragility parameters, regression analysis has been carried out. They have observed that fragility parameters change significantly with the number of stories of the building. Finally, using constructed fragility curves and statistical methods, they have estimated the maximum allowable inter-story drift ratio and spectral displacement values that satisfy the 'immediate occupancy' and 'collapse prevention' performance level requirements.

Also Arizaga (2006) have presented the fragility curves of steel moment frames of various numbers of stories up to ten just based on the inter-story drifts by using the RAM PERFORM software for nonlinear dynamic analyses. Very recently, Aziminejad and Moghadam (2007) have developed the fragility curves for 1-story reinforced concrete building models with shear walls based on the story drift and plastic hinge rotation.

It is seen that although there are several publication with regard to fragility curves, there are just few ones, focused on steel buildings, and particularly braced frames. However, the buildings with braced frames are among the most common types of steel buildings in Iran, and it is necessary to develop fragility curves for this type of existing building to help the risk evaluation and mitigation program of the country. In a very recent study, the authors of this paper have used an approach similar to that of Kircil and Polat, and also Aziminejad and Moghadam to develop the fragility curves for a group of steel buildings with x-bracings, with special attention to the location of braced bays (Majd, 2008). A summary of that study is presented in this paper.

2. DEVELOPING FRAGILITY CURVES BY NONLINEAR TIME HISTORY ANALYSES

To develop the fragility curves for any type of buildings based on Nonlinear Time History Analyses (NLTHA) the following steps should be followed:

1- Considering some typical samples of the concerned building type, and assuming some specific soil type for their site
2- Modeling the considered buildings based on the nonlinear behavior of materials and its damping characteristics
3- Selecting some recorded accelerograms of past earthquakes based on their frequency content to be compatible with the site soil, and scaling them for various PGA values
4- Considering some appropriate failure criteria for building structural members or its stories, such as Inter-Story Drift (ISD), Plastic Hinge Rotations (PHR), and Axial Plastic Deformation (APD) of bracing elements
5- Considering some suitable acceptance criteria for failure limits, based on codes or regulations
6- Performing NLTHA for each building by considering various levels of PGA
7- Selecting an appropriate statistical probability density function
8- Producing the fragility tables and curves

In the next section of the paper the above steps have been used for the case of steel building with X-bracing and various number of stories to obtain firstly the fragility curves for this type of buildings, and secondly to find out which damage criteria results in lower level of seismic fragility for this type of buildings.

3. INTRODUCING THE CONSIDERED MODELS OF BUILDINGS

Regarding that steel buildings with X-bracing are the most common type of multistory buildings in Iran, and that developing of fragility curves is almost new in this country, for developing the fragility curves, this building type was considered first. For this purpose two sets of buildings, including a set of 2- by 4-bay and another set of 4- by 6-bay plans having 3, 5, or 7 stories have been considered. Regarding that the majority of existing buildings in Iran have been designed for earthquake loadings by using version II of the Iranian Standard No. 2800 (2002) (Iranian Code for Seismic Resistant Design of Buildings), which is very similar to UBC-97, this version of that standard has been used for design of sample buildings. For calculation of the lateral loads factor in all cases the soil condition B has been used, since most of the existing constructions in Tehran are on this type of soil. Furthermore, the AISC-ASD89 code has been used for design of steel sections of the considered buildings, which has been in concurrent use with UBC-97. Plans of the considered buildings and the selected frames for analyses are shown in Figures 1 and 2.

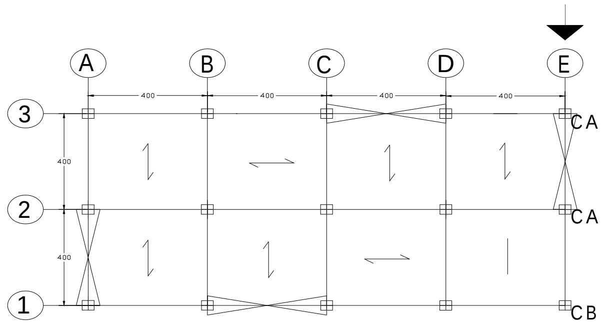


Figure 1. Plan of 2- by 4-bay buildings of which the 2-bay frames have been extracted for NLTHA

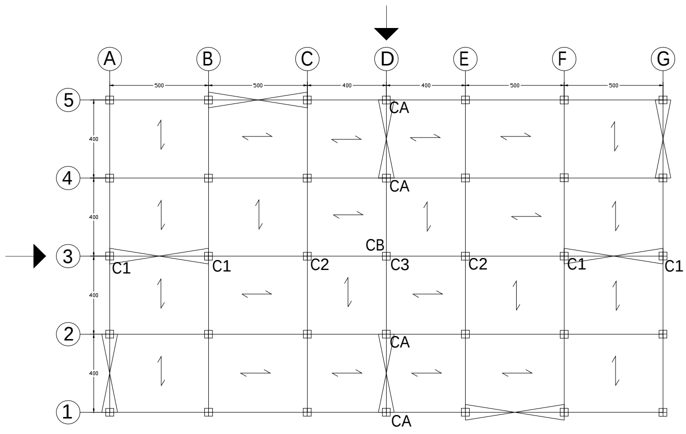


Figure 2. Plan of 4- by 6-bay buildings with bracings in non-adjacent bays

A simple rule has been used for naming the analyzed frames based on the number of stories, number of bays as follow. A name consisted of the letters FO followed by the number of stories and finally number of bays completely introduces each frame. For example, FO54 means a 5-story frame with 4 bays.

4. MODELING

To perform the NLTHA and evaluate the vulnerability of considered buildings the building frames have been modeled by Ram-Perform 3D software. The nonlinear or inelastic behavior of various structural members, including beams, columns and bracing elements has been introduced to the software based on the FEMA 356 (2000) guidelines. For beams and columns the general behavior, shown in Figure 3, and for bracing elements the one shown in Figure 4 has been used.

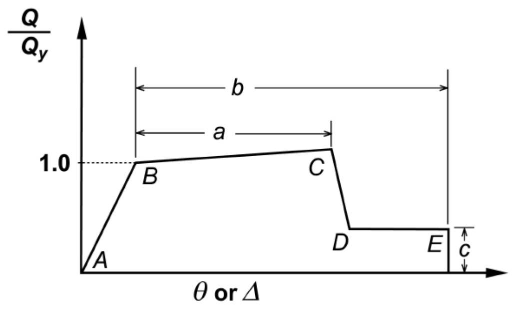


Figure 3. Inelastic model used for beams and columns

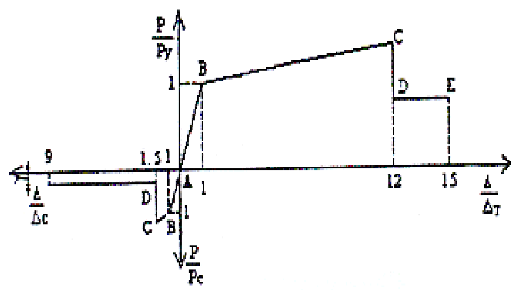


Figure 4. Inelastic model used for bracing elements

Parameters used in Figure 4, which define the inelastic behavior of bracing elements, are calculated by following formulas.

$$P_C = 1.7 F_a A \quad (1)$$

$$\Delta_C = \frac{P_C L}{AE} \quad (2)$$

$$P_y = F_Y A \quad (3)$$

$$\Delta_T = \frac{P_y L}{AE} \quad (4)$$

In these formulas A is the cross-sectional area and L is the length of element. The required parameters indicated in Figures 3 and 4 as well as those used in Equations (1) to (4) have been calculated based on the cross-sectional properties of various elements by using Tables 5-6 and 5-7 of FEMA356 guidelines, and have been introduced to the software accordingly. In Figure 5 a sample of inelastic behavior graph related to a bracing element made of two UNP120 is shown.

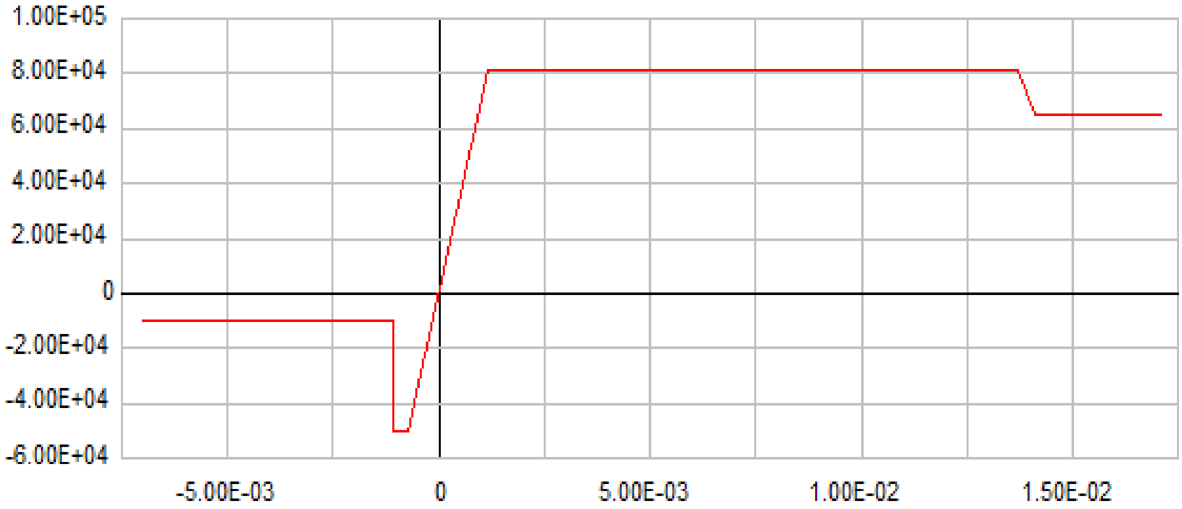


Figure 5. A sample of inelastic behavior graph related to a bracing element made of two UNP120 introduced to nonlinear dynamic analysis software

5. THE USED DAMAGE INDICES AND PERFORMANCE LEVELS

For developing the fragility curves it is necessary to use some reasonable "damage index" for each of the structural elements. In case of beams and columns the rotation of plastic hinges has been used widely by researchers, however, in case of bracing elements the axial relative deformation is an appropriate index. The inter-story drift has been also used as a damage index for fragility calculations in this study, and the results have been compared to realize which index is better for the case of braced frames.

Furthermore, three levels of minor, moderate, and extensive can be considered for the overall damage of an ordinary building, which is usually looked at as the Performance Level (PL) of the building subjected to a given earthquake of a specified hazard level. In FEMA 356 these three levels are called "Immediate Occupancy" (IO), "Life Safety" (LS), and "Collapse Prevention" (CP) performance levels, which have been used in this study. For this purpose exceedence of the selected damage index from the corresponding value associated with each of these performance levels means fragility of the system in that specific performance level. For axial plastic deformations of bracing elements three levels have been obtained based on the values given in Table 5-7 of FEMA 356, depending on the cross-section and acceptable value of axial plastic deformation. For inter-story drifts the values given in Table C1-3 of FEMA 356 have been used.

6. NONLINEAR TIME HISTORY ANALYSES (NLTHA)

For NLTHA of various building models six accelerograms recorded on soil type B, all having the PGA level around 0.35g, which is maximum PGA value in the code, have been used. The used accelerograms have been scaled to 7 various PGA levels of 0.1g to 0.7g to create totally 42 cases of time history analysis for each of the building models. The specifications of these accelerograms are given in Table 1.

Table 1. The specifications of used accelerograms

No.	Event	PGA (g)
1	Kocaeli, Turkey, 1999.8.17	0.375
2	Loma Prieta 1989.10.18	0.357
3	El Centro 1950.5.18	0.319
4	Northridge1995.01.17	0.363
5	San Fernando 1971.2.9	0.365
6	Duzce, Turkey1999.11.12	0.426

In fact, 42 cases of NLTHA have been performed for each frames model. Samples of response time histories can not be presented here because of lack of space, and can be found in the main report of the study (Majd 2008).

7. FRAGILITY CALCULATIONS

As it is common, the fragility curves have been developed by using PGA values as the variable parameter. Based on the numerical results of NLTHA the maximum values of inter-story drifts as well as the maximum axial plastic deformations of bracing elements have been obtained. These two parameters have been used as the damage indices for developing the fragility functions. On this basis the fragility function can be defined as:

$$Fragility = P[EDP > AC \mid IM] \tag{5}$$

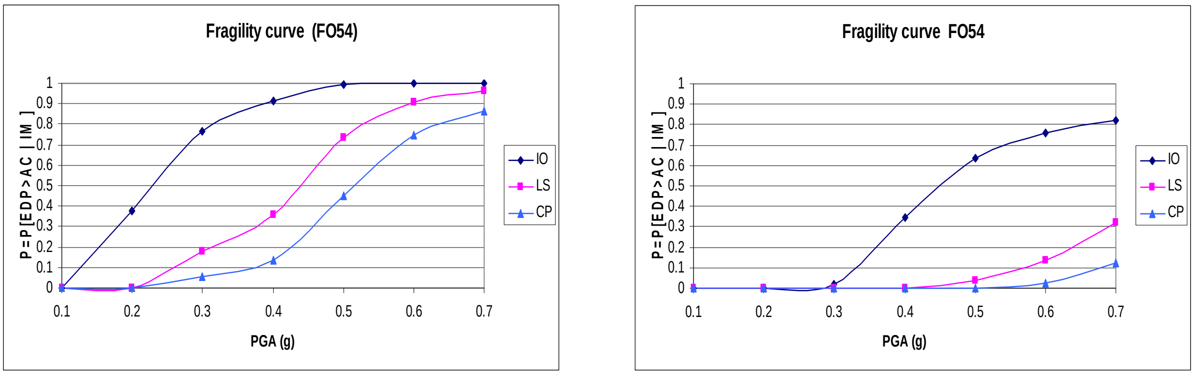
In Equation (5) IM is the Intensity Measure, which is the PGA value, and EDP is the Engineering Demand Parameter, which has been considered to be the same as either of Damage Indices in this study, and AC is the Acceptance Criterion, which has been considered to be Performance Level mentioned in section 5. The probability function, given in equation (5), can be calculated as:

$$P = P[EDP > AC] = 1 - P[EDP < AC] = 1 - \phi\left(\frac{AC - \mu}{\sigma}\right) \tag{6}$$

Although some researchers have proposed to use a log-normal probability density function, many other researchers have claimed that using the normal density function gives satisfactory results as well (Altug Erberik 2008; Suppasri et al. 2011). On this basis, a normal or Gaussian probability density function is assumed for the used EDP. To evaluate the probability of exceedence from a specific limit state, the average and standard deviation of each EDP is calculated for the ensemble of six earthquake records. Then using cumulative distribution function of normal distribution the exceeding probability of each EDP from the given limit state is calculated.

8. PRODUCING THE FRAGILITY TABLES AND CURVES

To develop the fragility curve for each of the considered frames, the numerical results obtained from the NLTHA and Equation (6) have been used. By using the obtained fragility data the fragility curves can be plotted. Figures 6 shows two samples of the developed fragility curves developed for 5-sotry frames in various performance levels using the two considered EDPs.



(a) (b)

Figure 6. Fragility curves for frame FO54 using APD of bracing elements, (a), and ISD, (b), as EDP

It can be seen in Figure 6 that for higher performance level the fragility values are higher as it is expected. However, the differences between the fragility curves in Figures 6 (a) and 6 (b) show that the two used EDPs do not yield to the same fragility levels. In fact, in all cases, using inter-story drift as EDP leads to lower fragility values comparing with using axial plastic deformation, and this difference in fragility values is more for lower performance level. On this basis, it can be claimed that using ISD as EDP is not reliable for fragility analysis of braced frames. This can be supported by the fact that in FEMA guidelines the calculations for plastic deformations have been presented in more details comparing with those for inter-story drifts. It seems that the values given in Table C1-3 for the ultimate values of drifts in the case of braced frames, which are 0.5, 1.5 and 2.0 for IO, LS, and CP performance levels, respectively, should be decreased to be compatible with results obtained by using axial plastic deformations of bracing elements as EDP. On this basis, just the results corresponding to the cases of APD index are presented and focused on for discussion. The fragility curves for 3-story frames with various numbers of bays, developed for the three PLs are shown in Figure 7.

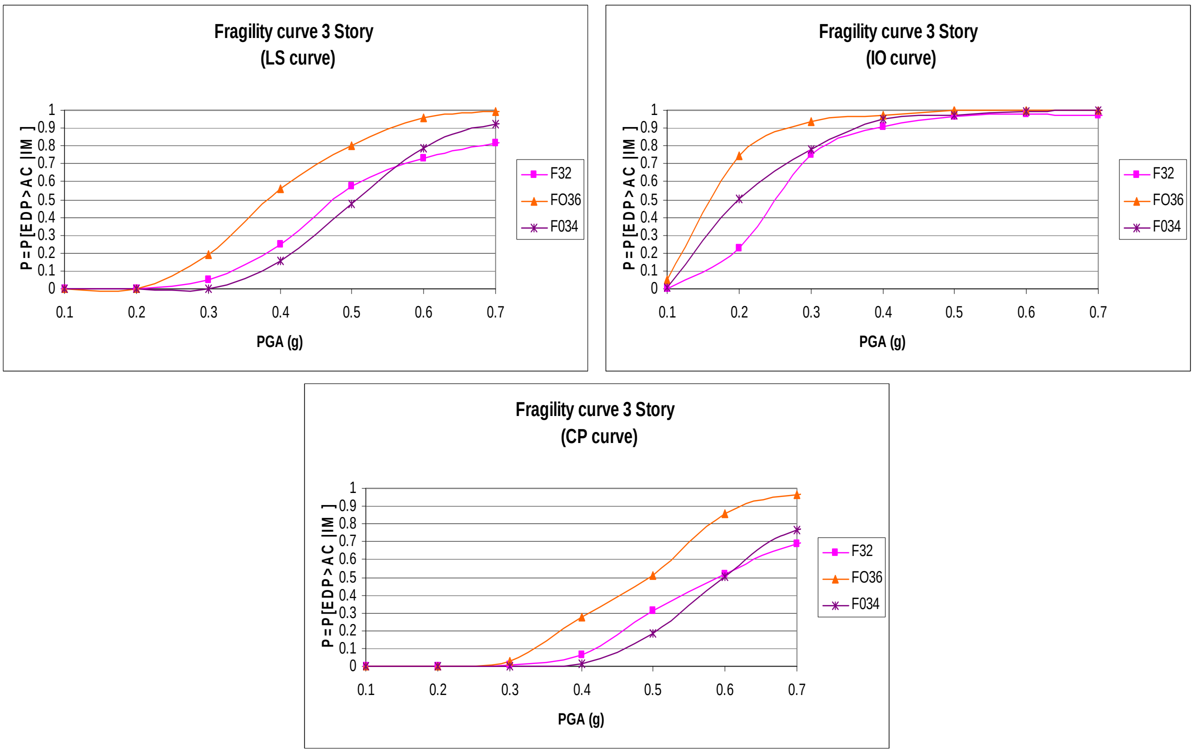


Figure 7. Fragility curves for 3-story frames

Looking at Figure 7, one can observe that in the case of 3-story frames:

- The fragility curves at IO performance level, depending on the frame type, starts somewhere between 0.1g and 0.2g, and reaches a value of almost 1.0 somewhere between 0.4g and 0.5g.
- The fragility curves at LS performance level, depending on the frame type, starts somewhere between 0.2g and 0.35g, and reaches a value of 0.8 or more in around the PGA value of 0.7g.
- The fragility curves at CP performance level, depending on the frame type, starts somewhere between 0.3g and 0.4g, and reaches a value of 0.7 or more in around the PGA value of 0.7g, however the fragility values in this performance level can not exceed 0.95 for the highest considered PGA value of 0.7g.

The fragility curves for 5-story frames with various numbers of bays, developed for the three PLs are shown in Figure 8.

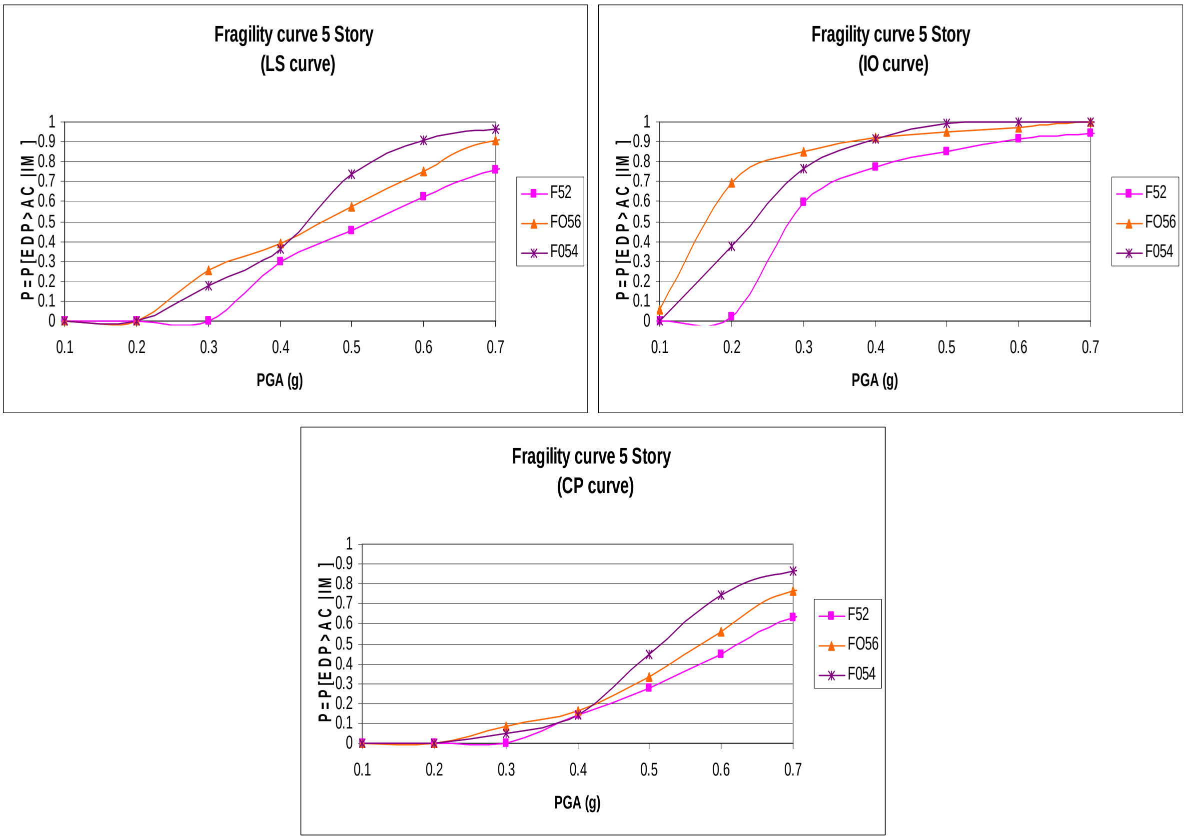


Figure 8. Fragility curves for 5-story frames

Based on the fragility curves shown in Figure 8 it can be said that in the case of 5-story frames:

- The fragility curves at IO performance level, depending on the frame type, starts somewhere between 0.1g and 0.2g, and reaches a value of almost 1.0 somewhere between 0.4g and 0.5g.
- The fragility curves at LS performance level, depending on the frame type, starts somewhere between 0.2g and 0.35g, and reaches a value of 0.7 or more in around the PGA value of 0.7g, however, the fragility values in this performance level can not exceed 0.95 for the highest considered PGA value of 0.7g.
- The fragility curves at CP performance level, depending on the frame type, starts somewhere between 0.25g and 0.4g, and reaches a value of 0.5 or more in around the PGA value of 0.7g, however, the fragility values in this performance level can not exceed 0.85 for the highest considered PGA value of 0.7g.

The fragility curves for 7-story frames with various numbers of bays, developed for the three PLs are shown in Figure 9.

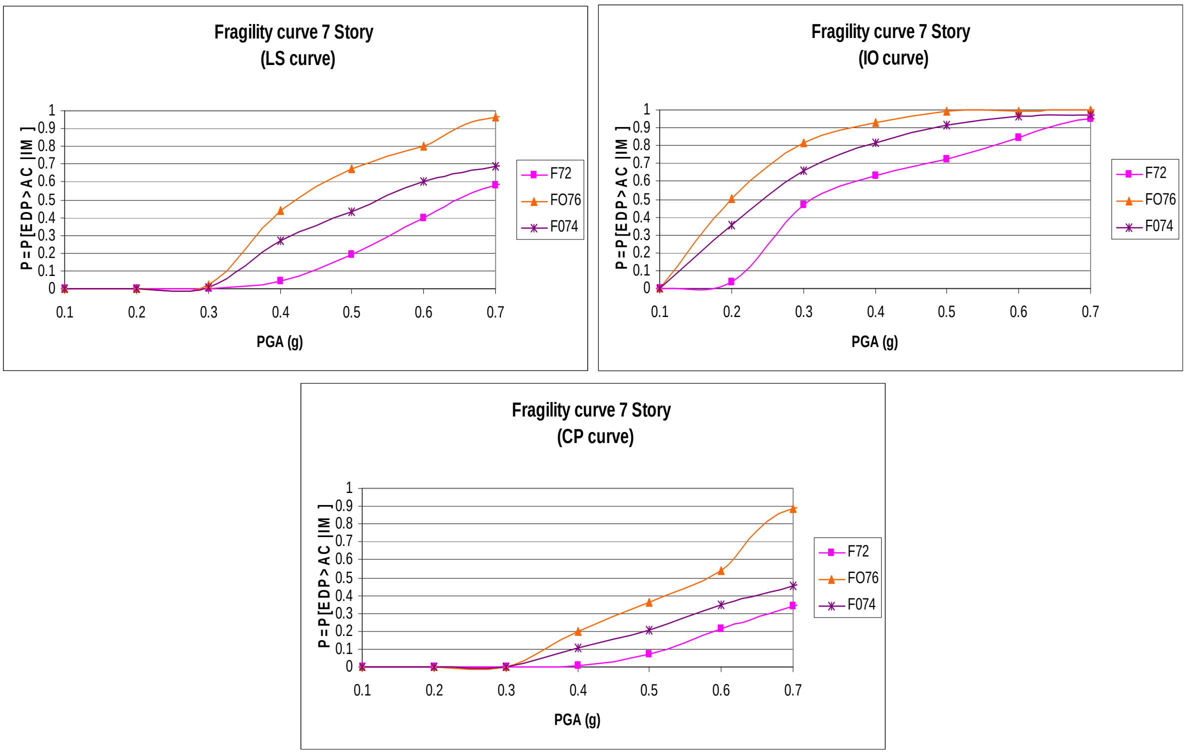


Figure 9. Fragility curves for 7-story frames

It can be seen in Figure 9 that for the case of 7-story frames:

- The fragility curves at IO performance level, depending on the frame type, starts somewhere between 0.1g and 0.2g, and reaches a value of 0.95 or more somewhere between 0.4g and 0.5g.
- The fragility curves at LS performance level, depending on the frame type, starts somewhere between 0.25g and 0.4g, and reaches a value of 0.6 or more in around the PGA value of 0.7g, however, the fragility values in this performance level can not exceed 0.95 for the highest considered PGA value of 0.7g.
- The fragility curves at CP performance level, depending on the frame type, starts somewhere between 0.3g and 0.4g, and reaches a value of 0.45 or more in around the PGA value of 0.7g, however, the fragility values in this performance level can not exceed 0.85 for the highest considered PGA value of 0.7g.

9. CONCLUSIONS

Based on the fragility curves, developed for steel buildings with various numbers of stories and X-bracings, by using nonlinear time history analyses, and considering once the “inter-story drift” and once more the “axial plastic deformation of bracing elements” as the damage index, the following conclusions can be stated:

- The number of stories of frames does not have a remarkable effect on their fragility values in various performance levels.
- Number of bays is an important factor in the fragility values of frames, and as this number increases the fragility values increase as well.
- In general for PGA values of 0.5g or more the variation of fragility values less than the variation corresponding to PGA values below 0.5g.
- Of the two damage indices of ‘inter-story drift’ (ISD) and ‘axial plastic deformation’ (APD) of bracing elements” the second index is more reliable for developing the fragility curves for steel frames with X-bracing. Considering that ISD is usually looked at as a system-level demand parameter, while the APD as a member-level parameter, APD is not expected to represent the

system fragility like the ISD, however, since the X-bracing elements start yielding in relatively small values of inter-story drift, and after yielding, usually, localization of plastic deformation happens, the ADP seems to be a better index in case of frames with X-bracing.

Finally, it should be noted that this study was on regular buildings. To get more general conclusions the irregular buildings should be studied as well.

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