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OPTIMIZING THE BRACING PATTERN OF STEEL BUILDINGS WITH CONCENTRICALLY BRACED FRAMES BASED ON FRAGILITY CONCEPTS

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ABSTRACT

In almost all seismic design codes the 'Response Modification Factors' (RMFs) are used to take into account the inelastic behavior of the building structures. However, suggested RMF by codes is a single value for each type of structure, which means that for case of CBFs, as an example, it is independent of the bracing pattern, namely the number of braced bays, their relative locations, or their distribution in the building's plan or elevation. This means that the code considers the ultimate load bearing capacity, or more specifically speaking, the seismic performance of a CBF, independent of its bracing pattern. However, several studies have shown that this is not true. This paper tries to show the dependency of the seismic performance of CBFs to the bracing pattern by using the fragility concepts. For this purpose some sets of steel buildings with 4- by 6-bay plans having 3, 5, and 7 stories have been considered, with various bracing patterns, including bracing in adjacent bays and bracing in non-adjacent bays. Then, the fragility curves have been developed for these buildings by performing a series of Nonlinear Time History Analyses (NLTHA). The buildings have been assumed to be regular in both plan and elevation to avoid the torsion effects. In NLTHA several recorded accelerograms, with various Peak Ground Acceleration (PGA) values and also different frequency contents, have been used for each building to create a relatively large statistical data for developing the fragility curves with high reliability. Based on the numerical results it can be concluded that for moderate to high PGA values (between 0.3g and 0.6g) the effect of bracing pattern in remarkable, so that the fragility values for the case of bracing in adjacent bays is 10% to 50% lower than those related to the pattern in which the bracing elements are location in non-adjacent bays.

Introduction

Almost all seismic design codes suggest the use of a so called 'Response Modification Factors' (RMFs) in calculation of seismic forces acting on buildings for taking into account the inelastic behavior of the building structure. However, suggested RMF by codes is a single value

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for each type of structure, regardless of, for example the location of braced bays in the CBFs, which means that for case of CBFs the RMF value is independent of the number of braced bays, their relative locations, or their distribution in the building's plan or elevation. In fact, this means that the code considers the ultimate load bearing capacity, or more specifically speaking, the seismic performance of a CBF, independent of its bracing pattern. This is while the structural mechanics knowledge suggests that the location of braced bays in a multi-bay frame does have effect on it linear and particularly nonlinear behavior subjected to lateral loads. However, just a few studies have been specifically performed on this issue (Baldock et al. 2005). In recent years the first author of this paper and his colleagues have conducted some studies on the effect of bracing pattern on the ultimate load bearing capacity of frames with X-bracing both analytically (Shadman 2006) and experimentally (Hosseini et al. 2008). In a recent study the fragility curves for a group of steel buildings with x-bracings have been developed, with special attention to the location of braced bays (Majd, 2008). A summary of that study is presented in this paper.

Steps of Developing Fragility Curves by Time History Analyses

To develop the fragility curves for any desired group of buildings the following steps are required:

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

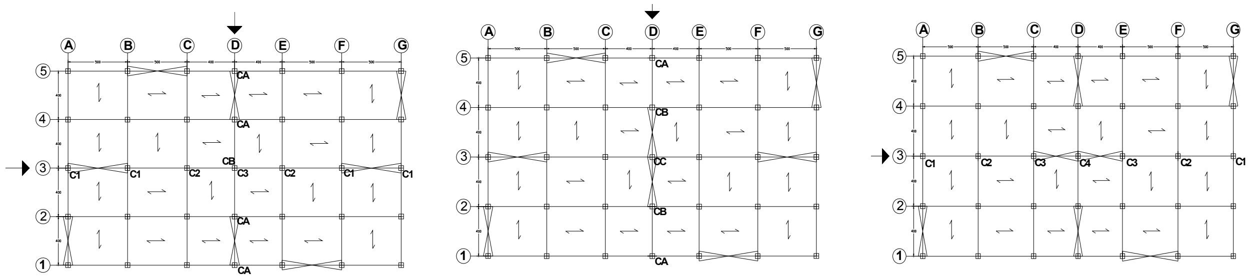
8- Producing the fragility tables and curves

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

Introducing the Considered Buildings of the Study

A set of X-braced steel buildings with 4- by 6-bay plans, having 3, 5, or 7 stories, have been considered for the study. Two different patterns for location of braced bays have been used, including one with adjacent braced bays and the other with non-adjacent braced bays. Regarding that the majority of existing buildings in Iran have been designed for earthquake loadings by using the Iranian Standard No. 2800 (Iranian Code for Seismic Resistant Design of Buildings), which is very similar to UBC-97. 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 Figure 1.



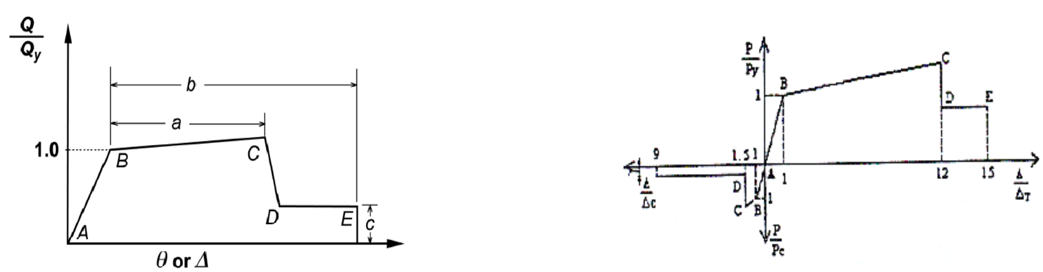
(a) bracings in non-adjacent bays (b) bracings in adjacent bays of 4-bay frames (c) bracings in adjacent bays of 6-bay frames

Figure 1. Plans of buildings with various bracing patterns

A simple rule has been used for naming the analyzed frames based on the number of stories, number of bays, and the location of braced bays as follow. A name consisted of the letter F followed by another letter T for bracings in two adjacent bays (Together), or O for bracings in non-adjacent bays (Open bay in between) followed by the number of stories and finally number of bays completely introduces each frame. For example, FT76 means a 7-story frame with 6 bays having adjacent bracings, and FO54 means a 5-story frame with 4 bays and non-adjacent bracings.

Modeling of Buildings' Structure

To perform the NLTHA and evaluate the vulnerability of considered buildings the building frames have been modeled by Ram-Perform 3D software (Powell 2000). 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 guidelines (ASCE 2000), and are shown in Figure 2.



(a) (b)

Figure 2. Inelastic model used for beams and columns, (a), and for bracing elements, (b)

Parameters used in Figure 2, which define the inelastic behavior of bracing elements, are calculated by following formulas, based on the limit state stress values of steel in compression and tension, F_a and F_y, respectively.

$$P_C = 1.7 F_a A \tag{1}$$

$$\Delta_C = \frac{P_C L}{AE} \tag{2}$$

$$P_y = F_Y A \tag{3}$$

$$\Delta_T = \frac{P_y L}{AE} \tag{4}$$

In these formulas A is the cross-sectional area and L is the length of element. In Figure 3 a sample of inelastic behavior graph related to a bracing element made of two UNP120 is shown.

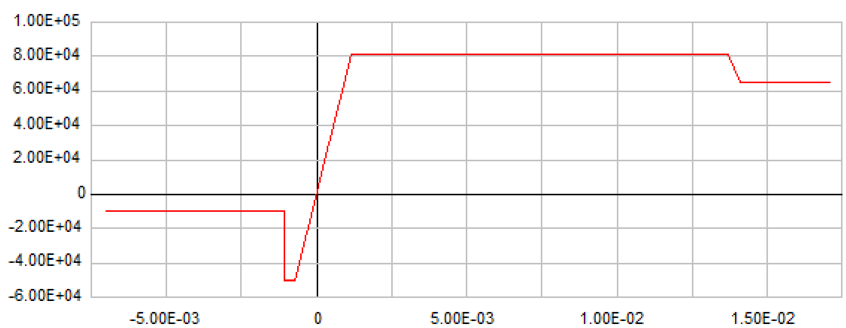


Figure 3. A sample of inelastic behavior graph related to a bracing element made of two UNP120 introduced to Ram-Perform 3D software

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 both of these indices have been used and compared to realize which one is a better index 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 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 form 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.

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, whose specifications are given in Table 1.

Table 1. The specifications of applied 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	Northridge 1995.01.17	0.363
5	San Fernando 1971.2.9	0.365
6	Duzce, Turkey 1999.11.12	0.426

The used accelerograms have been scaled to 7 various PGA levels of 0.1g to 0.7g to develop the fragility curves. On this basis, 42 cases of NLTHA have been performed for each frames model.

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 for fragility calculations. 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 had given in equation (5) can be calculated as:

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

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.

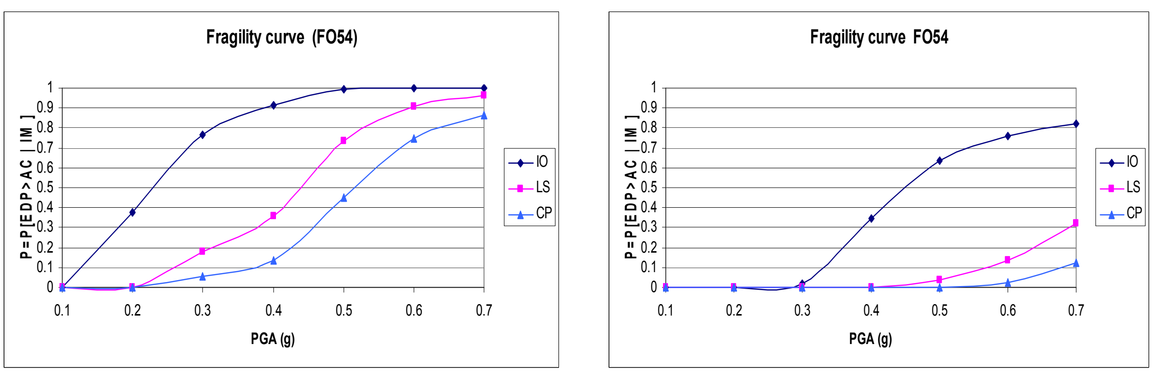
Producing the Fragility Tables and Curves.

To develop the fragility curve for each of the considered frames, at first the numerical results obtained from the NLTHA and Equation (6) have been summarized in some tables, like Tables 2. More results can be found in the main report of the study (Majd 2008).

Table 2. Fragility data for FO54 using APD of bracing elements as EDP

FO54	Axial plastic deformation - Intensity Measure						
	0.1g	**0.2g**	**0.3g**	**0.4g**	**0.5g**	**0.6g**	**0.7g**
DUZ	0.00049	0.00122	0.00448	0.005207	0.0061	0.0074	0.0081
ELS	0.0001	0.000433	0.000877	0.001901	0.00409	0.00577	0.00603
KOC	0.000103	0.00036	0.000843	0.00243	0.003916	0.00507	0.00568
LOMA	0.00009	0.000475	0.00194	0.002125	0.00325	0.00435	0.00461
NOR	0.00009	0.000389	0.000982	0.00206	0.00356	0.00446	0.00504
SAN	0.00008	0.000239	0.000885	0.001292	0.00209	0.00296	0.00401
Arithmetic mean	0.000159	0.000519	0.001668	0.002503	0.003834	0.005002	0.005578
Var	2.64E-08	1.24E-07	2.07E-06	1.9E-06	1.73E-06	2.25E-06	2.05E-06
Standard Deviation	0.000162	0.000353	0.00144	0.001377	0.001316	0.001499	0.001433
P(X>XI)=IO	0.835906	0.929643	0.876534	0.965387	0.998209	0.999576	0.999951
P(X>XI)=LS	0.835906	0.929643	0.876534	0.965387	0.998209	0.999576	0.999951
P(X>XI)=CP	0.835906	0.929643	0.876534	0.965387	0.998209	0.999576	0.999951

Tables of this type show the values of mean, variance, and standard deviation of the considered EDPs, obtained from NLTHA by using the six employed earthquake records, as well as the exceedence probabilities calculated by Equation (6) for various performance levels. By using the fragility data given in such tables 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 4. Fragility curves for FO54 using APD of bracing elements, (a), and ISD (b) as EDP

It can be seen in Figure 4 that for higher performance level the fragility values are higher as it is expected. However, the differences between the fragility curves in Figures 4-a and 4-b show that the two used EDPs do not yield to the same fragility levels.

The Effect of Bracing Pattern in Fragility Values

In almost all seismic design codes the "response modification factor" (RMF) is used to take into account the plastic deformation of the building structure and its energy dissipation capacity in the seismic design. However, suggested RMF by codes is a single value for each type of structure, which means that for case of CBFs, as an example, it is independent of the bracing pattern, namely the number of braced bays and their relative locations. This means that the code considers the ultimate load bearing capacity, or more specifically speaking, the seismic performance of a CBF, independent of its bracing pattern. However, several studies have shown that this is not true (Hosseini and Esmaeili, 2006; Hosseini et al., 2008). In this section of the paper it is tried to show the dependency of the seismic performance of CBFs to the bracing pattern by using the fragility concepts. Figures 5 to 7 show the fragility curves developed for 3-, 5-, and 7-story frames with 4 or 6 bays in two cases of bracings in adjacent and non-adjacent bays in three performance levels.

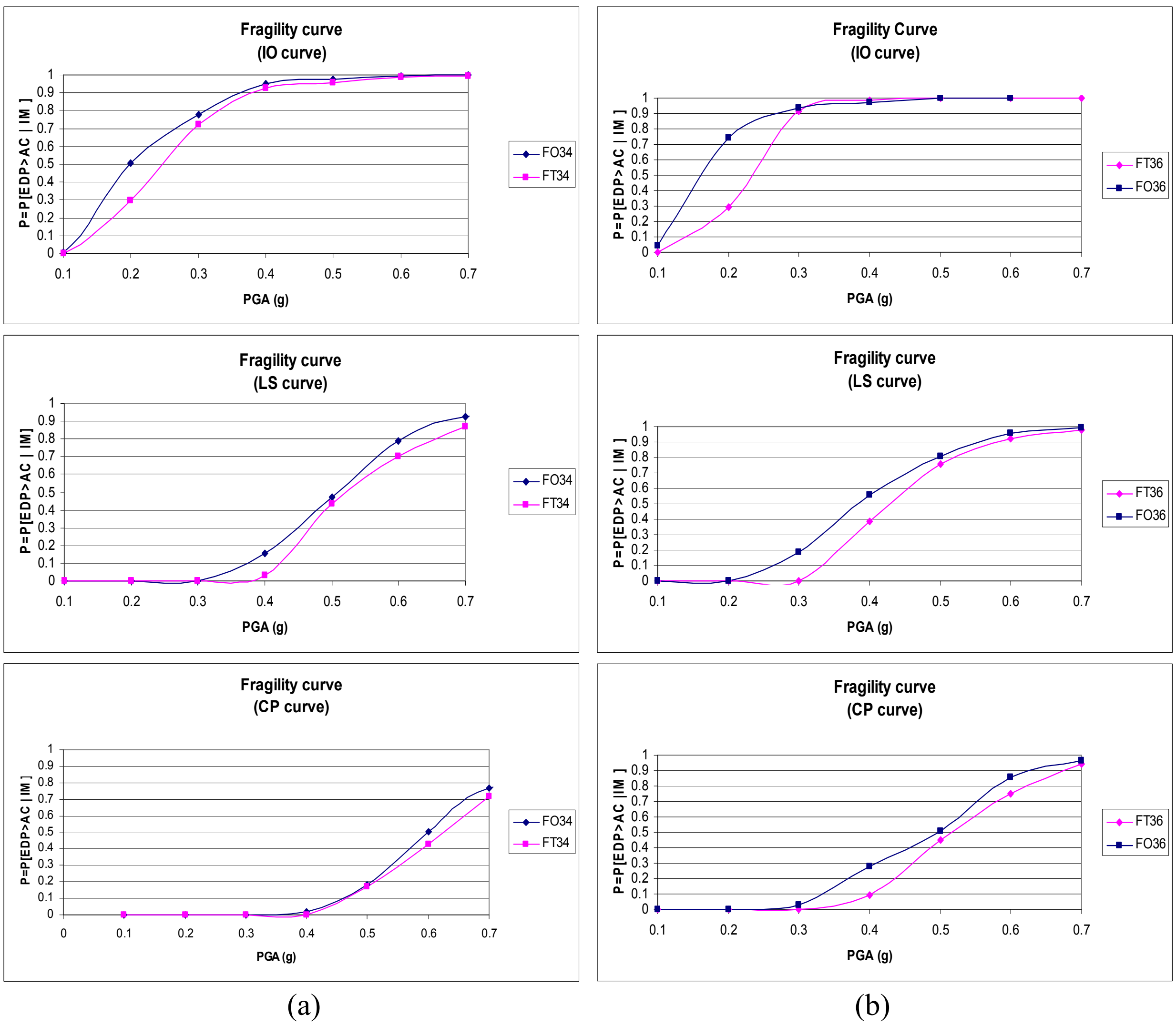
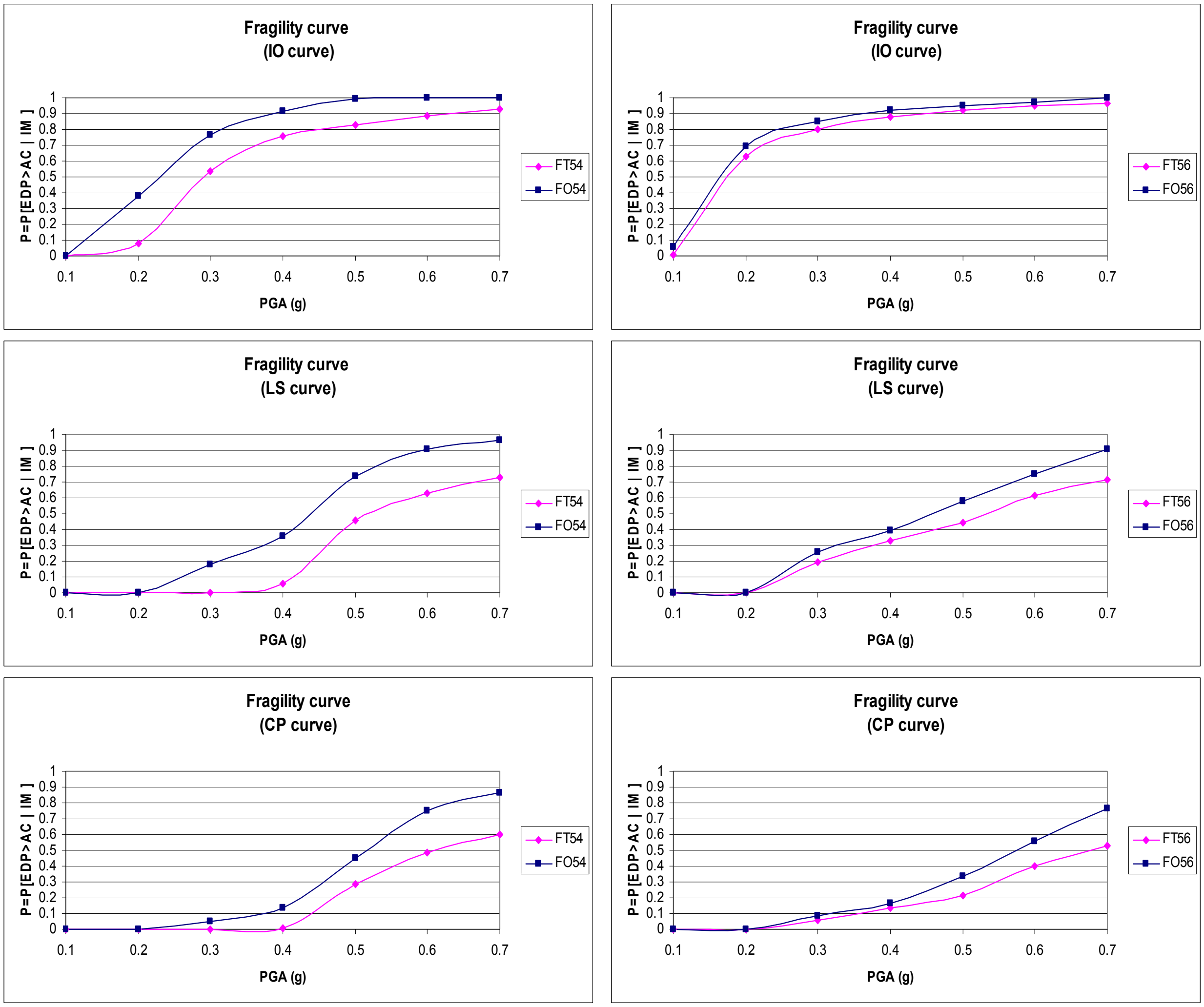


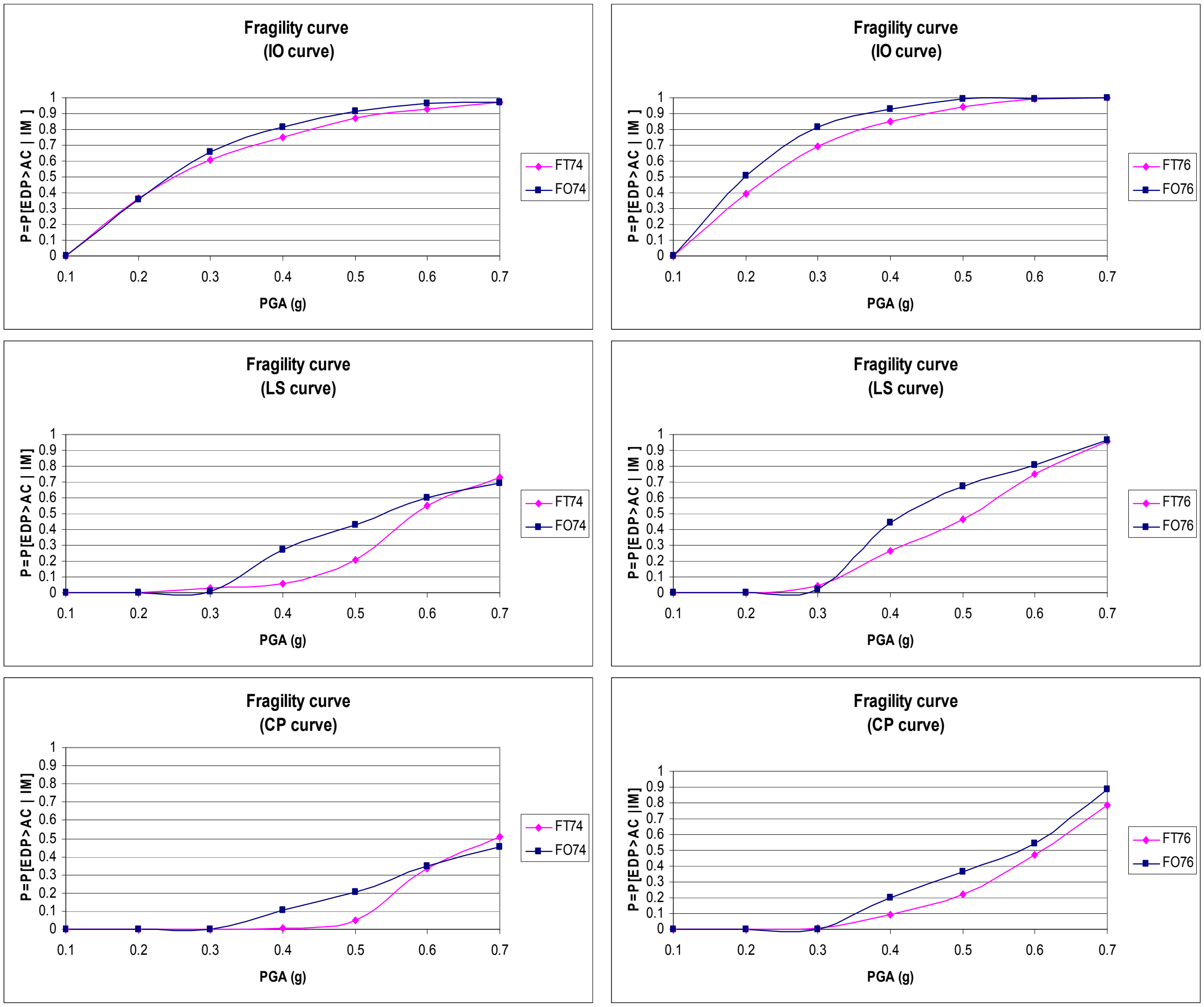
Figure 5. Comparison of the fragility curves of 3-story frames with 4 bays, (a), and 6 bays, (b), in two cases of bracings in adjacent and non-adjacent bays in IO, LS, and CP performance levels



(a) (b)

Figure 6. Comparison of the fragility curves of 5-story frames with 4 bays, (a), and 6 bays, (b), in two cases of bracings in adjacent and non-adjacent bays in IO, LS, and CP performance levels

Looking at Figures 5 to 7, one can realize that the fragility of frames with bracing in adjacent bays are generally lower that that of frames with bracing in non-adjacent bays. This means that using bracing elements in adjacent bays leads to more reliable seismic design. On the other hand, it can be also suggested to use higher RMF values for the case of bracing in adjacent bays, which will result in more economical seismic design. On this basis, it is possible to optimize the seismic design of steel buildings with CBFs based on choosing more appropriate configuration of braced bays, by using their fragility values. In other words, the minimum fragility values can be used as optimization criteria for achieving the optimum bracing pattern. By comparing figures 5, 6 and 7 it can be seen that in case of 3- and 7-story buildings the fragility of 6-bay frames is generally higher than that of 4-bay frames, while in case of 5-story buildings the situation is vice versa. The reason behind this difference can be the various stiffness and resulting fundamental periods of the buildings with the same number of stories but different number of bays, and also their different ultimate strengths.



(a) (b)

Figure 7. Comparison of the fragility curves of 7-story frames with 4 bays, (a), and 6 bays, (b), in two cases of bracings in adjacent and non-adjacent bays in IO, LS, and CP performance levels

Conclusions

In this paper, by using nonlinear time history analyses, the fragility curves were developed for steel buildings with X-bracing in two bracing pattern: 1) bracing in non-adjacent bays, and 2) bracing in adjacent bays. Based on the numerical results it can be concluded that:

- Of the two damage indices of "inter-story drift" and "axial plastic deformation of bracing elements" the second index is more reliable for developing the fragility curves.
- The fragility of frames with bracing in adjacent bays are generally lower that that of frames with bracing in non-adjacent bays, therefore, the minimum fragility values can be used as optimization criteria for achieving the optimum bracing pattern.
- For moderate to high PGA values (between 0.3g and 0.6g) the effect of bracing pattern in remarkable, so that the fragility values for the case of bracing in adjacent bays is 10% to 50% lower than those related to the other pattern, depending on the number of bays and number of

stories of the frames. This is particularly important for design purposes.

- For very high PGA values (more than 0.6g) again the fragility values get close for the two bracing patterns.

Finally, based on the results of this study, it can be suggested that the effect of bracing pattern is taken into account in determining the values of "response modification factor" of building systems in seismic design codes.

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