

Article

A Comparison of the Targeted and Analytic Performance of a Post-Tensioned Frame Structure Through Fragility Analysis

Murat Serdar Kirçil *  and Zeynep Kirici

Department of Civil Engineering, Yildiz Technical University, Esenler, Istanbul 34220, Turkey; zeynepkirici@hotmail.com

* Correspondence: kircil@yildiz.edu.tr or mskircil@gmail.com

Abstract: Buildings designed in accordance with Earthquake Codes are expected to survive slight earthquakes without damage, moderate earthquakes with limited and repairable damage, and severe earthquakes without collapse and loss of life. These definitions can be considered as target performance levels; however, whether the performance targets have been achieved is not verified using an engineering parameter. It is assumed that a structure designed in accordance with the code regulations will meet the prescribed controlled damage behavior and that the risk to life safety is minimum. However, studies examining the accuracy of this assumption in terms of the probability of exceeding the targeted performance levels are not widely available in the literature. The purpose of this study is to investigate to what extent this assumption is realistic. The targeted and analytically determined performance of the examined post-tensioned reinforced concrete frame building was compared through the generated fragility curves. The results show that the analytical performance of the examined building does not completely comply with the targeted performance defined by the Turkish Seismic Design Code; therefore, the assumption that a structure designed in accordance with the code will exhibit the targeted performance is not always realistic for each type of building.

Keywords: fragility curves; post-tensioned frame; damage estimation; performance; incremental dynamic analysis



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1. Introduction

The current seismic design philosophy requires ductile design of reinforced concrete structures so that the prescribed controlled damage behavior and life safety can be achieved. These definitions may qualify as target performance levels, but they are not defined in terms of an engineering parameter, and it is assumed that a building designed in accordance with current codes will exhibit the target performance. Over the last two decades, methods, including nonlinear analysis techniques, have become widely used for the damage estimation of the existing building stock. However, except for some special and critical structures that will be used immediately after an earthquake, the design procedure does not require one to check whether the targeted performance level requirements are satisfied or not for newly designed buildings. In other words, buildings designed in accordance with codes are assumed to meet the prescribed controlled damage behavior, and life safety can be achieved. However, it is well known that structures, designed according to current regulations, may not always fully meet the targeted performance requirements. On the other hand, there is no information regarding to what extent this situation may occur. Although there are many studies in the literature examining the probability of damage of different types of structures [1–22], studies examining the accuracy of this assumption are not widely found in the literature. In addition to this source of uncertainty, each newer version of design codes targets safer design and higher performance for new buildings. Thus, buildings that were designed according to the former version of seismic design codes may not completely comply with the performance criteria defined by new version of codes.

Fragility curves have been used in numerous studies to estimate the potential seismic damage of various types of structures. Casotteo et al. [1] investigated the effects of prefabricated beam–column and roof element–beam connections on the seismic performance of prefabricated industrial structures through fragility curves. They obtained a set of fragility functions for Italian industrial precast buildings and observed that the collapse fragility curves are more sensitive to the friction coefficient used to compute the connection capacity rather than to the ground motion intensity. Elligwood et al. [2] investigated the seismic fragilities of several typical low-to-mid-rise steel and reinforced concrete buildings representative of design and construction practices in the Central and Eastern United States. They concluded that gravity-designed concrete frames may not survive current design-basis earthquakes without severe damage or collapse. Hueste and Bai [3] made an assessment for a reinforced concrete frame structure representative of 1980s construction in the Central United States. They obtained the fragility curves of the existing building. Moreover, seismic fragility relationships were developed for the retrofitted structure based on three possible retrofit techniques and several performance levels. Inel and Ozmen [4] studied seismic performance of six school buildings, reflecting the structural properties of major percentage of school buildings located in several regions of Turkey whose seismic risk is high, through fragility curves. They observed that insufficient shear reinforcement and lack of shear walls of the examined buildings had increased the probability of damage. Jeong et al. [5] investigated 12 RC buildings with different structural system to evaluate the relative seismic safety margins of multi-story code-compliant RC structures and proposed a relationship to enable the quantifying of the Life Safety limit state probabilities of code-compliant mid-rise RC buildings. Kazantzi et al. [6] proposed a simplified fragility methodology for Regular Steel MRFs. Langa and Bachmann [7] obtained fragility curves showing the seismic performance of 87 existing buildings, one third of which are partially reinforced concrete, in a district of Basel. Martins et al. [8] presented an analytical methodology for the development of damage-to-loss functions for reinforced concrete buildings. They conducted nonlinear dynamic analysis and derived sets of fragility functions so that realistic estimates of economic losses due to seismic action can be provided. They highlighted some issues in the derivation of vulnerability functions as a fundamental component of a seismic risk assessment. Pasticier et al. [9] presented fragility curves of an existing two-story masonry building, which is representative of the existing building type in the Northeast of Italy. The equivalent frame modeling was employed in all analyses they carried out. They concluded that a proper seismic retrofit was required to reduce the seismic vulnerability of the studied building. Senel and Kayhan [10] examined 65 precast industrial buildings located in Denizli, Turkey. They obtained PGV-based fragility curves using strain-based displacement limits and corresponding damage states. They concluded that the most effective parameters that affect the damage probability of precast buildings are stiffness and ductility rather than strength. Silva et al. [11] developed a set of vulnerability functions of 48 reinforced concrete building typologies, based on nonlinear dynamic analyses of 2D moment-resisting frames with masonry infills, using one hundred ground motion records to derive a vulnerability model.

More recently, Rashid et al. [12] carried out a seismic assessment of two typical reinforced concrete moment frame structures constructed during the Soviet Union in Almaty, Kazakhstan. The investigated buildings represent 812 buildings with almost identical construction materials, geometries, and structural details. They proposed a new approach to determine structural global interstory drift limits at different damage states and obtained fragility curves that show damage probability of investigated structures based on the aforementioned interstory drift limits. They concluded that the investigated buildings are vulnerable to future strong ground motions because of their inadequate structural details. Kim and Kim [13] proposed an algorithm that is adaptable to traditional vulnerability analysis procedures to take into account both aleatoric and epistemic uncertainties. The algorithm uses Gaussian process-based metamodels to take epistemic uncertainties into consideration with a low number of time history analyses. They showed the ap-

plicability of the proposed algorithm on steel moment-resisting frame structures and a reinforced concrete building. Upadhyay and Pantelides [14] studied fragility curves of bridge bents with post-tensioned concrete columns for accelerated bridge construction to investigate the effect of rocking behavior of precast unbonded post-tensioned concrete columns. They compared the performance of post-tensioned bridge piers with conventional cast-in-place RC columns based on the results obtained from a nonlinear time-history analysis. They found that the performance of the investigated bridge satisfied performance criteria similar to a monolithic cast-in-place concrete bend designed according to the latest seismic design codes. Furthermore, they presented a method that is called the fragility-informed approach to design accelerated bridge construction bridge bents constructed with unbonded post-tensioned concrete columns. They showed that the residual slip in the post-tensioned bent was practically zero and post-tensioned concrete columns are expected to exhibit post-seismic serviceability and quick functional recovery after a major earthquake. Li et al. [15] investigated the effect of connection characteristics on the seismic performance of post-tensioned self-centering concrete frames through fragility curves. The results show a reduction in collapse probability provided by post-tensioned self-centering concrete frames connections with large energy dissipation. They also stated that the enhancement of beam end confinement is more effective for post-tensioned self-centering concrete frames with small energy dissipation capacities. Li et al. [16] explored the effect of long-term prestress loss on lifetime seismic performance of post-tensioned self-centering concrete frames in terms of damage probability via time-varying fragility curves. They presented an approach to assess the impact of prestress loss on the long-term seismic performance of post-tensioned self-centering concrete frames. They observed that a decrease in performance of upper floors is more significant after long-term service. The location of the maximum structural response changes with redistribution of story drift since the displacement profile changes with time. They also stated that the change in fragility over time is not always consistent for all stories for a post-tensioned self-centering concrete frame. In some cases, although the maximum fragility of the whole structure decreases with time, the vulnerabilities of higher stories increase. The most fragile story would also be changed with time. Ahmadi et al. [17] investigated the seismic performance of the precast post-tensioned segmental bridge piers under the effect of pulse-like nearfield earthquakes through fragility curves. They analyzed three piers under the original pulse-like near-field records, their extracted pulse ground motions, and their corresponding non-pulse records. It is observed that the effect of the pulse ground motion is important if the pulse period is close to the pier's natural period. It is also seen that pulse-like near-field and far-field ground motions generally have similar effects on the behavior of precast post-tensioned segmental bridge piers. Genikomsou et al. [18] obtained fragility curves that relate the probability of a post-tensioned slab-column connection exceeding a performance level or damage state in terms of interstory drift ratio and gravity shear ratio to be used as a basis for generating reliability-based formulations to determine the drift limits. It is shown that connections with continuous bottom reinforcement, shear reinforcement, or prestressing have increased drift capacities while higher gravity shear ratios significantly lower the limit drift ratio of both reinforced concrete and post-tensioned slab-column connections without shear reinforcement. Li et al. [19] carried out an assessment on the seismic performance of post-tensioned self-centering frames under the effect of near-fault pulse-like ground motions. They developed fragility surfaces for different damage states with the help of incremental dynamic analysis. The results show that the impacts of pulse period on the post-tensioned self-centering frames of different stories are different. Also, the most unfavorable pulse period changes with the severity of structural damage. Geng et al. [20] investigated the structural performance of a novel self-centering precast concrete frame with hysteretic dampers under the effect of near-fault earthquakes. By the help of fragility curves, they compared the seismic performance of the aforementioned structure with a reinforced concrete frame and a traditional self-centering precast concrete frame whose structural system characteristics are consistent with the novel self-centering

precast concrete frame with hysteretic dampers. The dynamic time–history analysis at the two seismic levels is also carried out for two types of frames. Results demonstrate that the novel self-centering precast concrete frame has great seismic performance under the effect of near-fault earthquakes loading. Granello et al. [21] investigated the seismic performance of a post-tensioned timber frame building designed with and without supplemental damping devices by carrying out fragility curves analysis. Fragility curves are developed to take the effect of post-tensioning loss expected in fifty years into account. Results show that the additional dissipators decrease the effect of post-tensioning loss for high-intensity earthquakes. However, performance under low-intensity earthquakes is strongly dependent on the amount of post-tensioning. In the last few years new methods have been developed based on the opportunities provided by computer technologies, such as optimization, machine-learning, genetic algorithms, etc., to be used for seismic risk assessment. Kazemi et al. [22,23] proposed and used machine-learning methods for structural response estimation and seismic risk assessment.

This paper focuses on the comparison of targeted performance levels and analytically determined the performance level of a hypothetical post-tensioned RC frame building through fragility analysis. The investigated building was designed according to provisions of the Turkish Seismic Design Code 2007 [24] given for cast-in-place ductile moment-resisting monolithic RC frames, since there are several buildings designed and built following this approach. Elastic earthquake load is determined by using parameters defined in the seismic design code. The strength reduction factor of cast-in-place ductile moment-resisting monolithic RC frames, which reflects redundancy, overstrength, and ductility, was used to obtain the Design Earthquake load. Furthermore, additional provisions must be satisfied so that the ductile behavior can be guaranteed such as reinforcement limits of structural system elements, strong column–weak beam design, a limitation on transverse reinforcement spacing, a limitation on axial force level of columns, etc. All those provisions are given for cast-in-place ductile moment-resisting monolithic RC frames by the Turkish Seismic Design Code 2007 [24]. However, a chapter was introduced for the special rules for the design of new post-tensioned precast reinforced concrete frame buildings and rules to be followed for the evaluation of existing counterparts with the 2018 version of the Turkish Seismic Design Code [25]. Thus, investigation and comparison of the real and targeted performance of such existing buildings is an essential requirement and worth investigating. Incremental dynamic analyses (IDA) were performed for the considered sample building to determine the damage threshold of corresponding performance levels in terms of the interstory drift ratio, and fragility curves have been obtained based on the determined threshold values.

2. Materials and Methods

2.1. Sample Building

Prestressing has been widely used in reinforced concrete structures and their components such as bridges, water tanks, slabs, etc. It provides some advantages compared to cast-in-place concrete structures such as decrease in deflection, increase in structural stiffness, supplying lighter structural system to span longer distance, etc. Prestressing technology is an evolving concept. For instance, the use of prestressing only in combination with pure biobased composite materials is being studied and developed by researchers. A study has been recently made to investigate the possibility of using prestressing technology with bamboo–timber composite beams by Zang et al. [26] called bio-based prestress technology. Zang et al. [26] investigate the failure modes, load capacity, and corresponding deflections of bamboo–timber composite beams at different stages of behavior such as serviceability, yielding, and ultimate. Based on the experimental results, they confirmed the feasibility, effectiveness, and industrial scalability of the proposed technology.

The hypothetical sample building, investigated within the scope of this study, is a precast post-tensioned reinforced concrete building. The frames are expected to behave as full-prestressing structures under vertical load and moderate earthquakes. Under the effect

of a Design Earthquake and a Maximum Earthquake, the connections rotate elastically to prevent damage to beams and columns. Moreover, since the beams and columns are built with high-quality industrial production, the relatively better quality control of concrete and reinforcement can be provided during the production of structural elements. This high-quality production provides durability, and applied prestress prevents cracks. This type of construction allows joint rotation during earthquakes and that behavior improves the capacity to absorb input energy during an earthquake. Post-tensioned self-centering beam–column connections can be beneficial from the point of an earthquake-resistant design requirement view since they exhibit relatively low residual displacement compared to conventional monolithic cast-in-place connections. However, it is evident that the beam–column connections of these structures are critical and can cause a negative effect on the seismic performance if not properly designed. Tendons are used to provide self-centering capacity to connections, and mild steel bars are used to increase the section ductility. Repair costs after an earthquake may be lower because of the low residual displacement. The effectiveness of such types of connections of various structural systems were investigated by several researchers through analytic studies [14–21,27–29]. The effectiveness of such types of connections were also investigated by experimental tests [28,30,31]. The obtained results demonstrate the good properties of the connections in terms of strength and deformation capacity, control of story drift, easy reparability, and preventing beams and columns from damage. The rapid erection of prefabricated columns is another advantage of such types of structures. Prefabricated beams are installed following the erection of columns and then prefabricated slab panels are installed. This procedure is repeated for each story. After completing the procedure just explained above, post-tensioning is applied at each story level.

The investigated building is a 7-story R/C residential building with 2 spans, and its structural system consists of ductile post-tensioned R/C frames in both directions. The plan dimensions are 24.6 m $\times$ 16 m, and the story height is 3.13 m. Slabs are composed of prefabricated R/C plates with 12 cm thickness and cast-in-place topping concrete with 7 cm thickness. Columns are continuous throughout the height of the building and beam–column connections are post-tensioned. The sample building has been designed according to Turkish Seismic Design Code 2006 [24] provisions as a Ductile Moment-resisting Frame. Thus, it satisfies the requirements of a typical Ductile Moment-resisting Frame such as strong column–weak beam design, capacity design for shear reinforcement, limitation on axial force level of columns, minimum compressive reinforcement for beam sections, limitation on spacing of shear reinforcement, and so on. Figure 1 shows the typical plan view of the sample building, and Figure 2 shows a schematic representation of a typical column–beam connection.

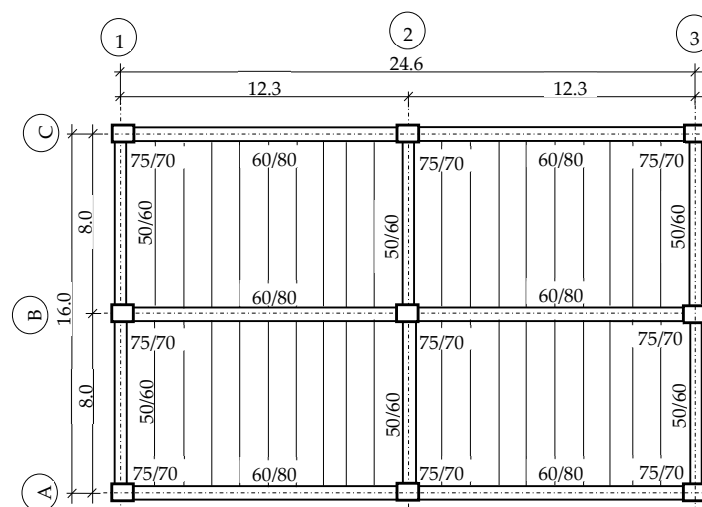


Figure 1. Typical plan view of the investigated building.

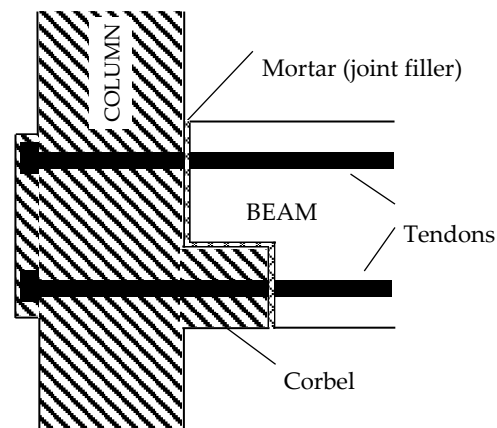


Figure 2. A typical beam–column connection.

Authors believe that showing all column and beam sections here is not meaningful since all the structural system elements are similar to each other in terms of size and reinforcement; however, a column and beam section is shown in Figure 3 as an example. Since the hypothetical post-tensioned RC frame building is under consideration, the assumed characteristic strength of concrete, reinforcement steel, and the post-tension cable are 50 Mpa, 420 Mpa, and 2700 Mpa, respectively. The Mander model [25,32] was used to describe the stress–strain relationship of both confined and unconfined concrete. For the stress–strain relationship of reinforcement steel, the model proposed by Turkish Seismic Design Code [25], including strain-hardening, is used. Bilinear stress–strain relationship is used for post-tension tendons. Figure 4 shows the stress–strain relationship of the materials explained above.

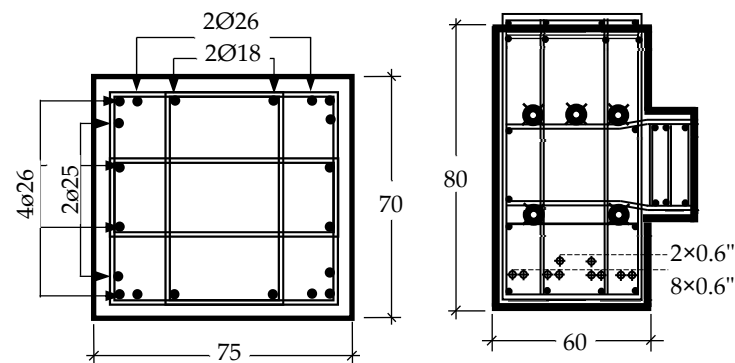


Figure 3. Typical column and beam sections.

The considered building is a hypothetical one as mentioned before. Thus, any test and statistical evaluation of test results to determine the material quality is not required. However, the analytic procedure followed within the scope of this study must be the procedure used for the performance determination of existing buildings. Thus, the mean value of material strengths must be taken into consideration. The Mean value of material strengths to be used for Nonlinear Time History Analysis were determined following the provisions of the Turkish Seismic Design Code [25] based on the assumed characteristic strengths. The Mean compressive strength of concrete is taken as equal to 1.30 times the characteristic strength, and, similarly, the mean yield strength of reinforcement steel and tendons is taken as 1.2 times the characteristic strength according to Turkish Seismic Design Code [25] provisions. However, a statistical evaluation is required in the case of a performance evaluation of a real existing building. Detailed information on the statistical evaluation can be found in [33]. The period of first and second modes of the structure, in the Y and X direction, is 0.89 s and 0.73 s, respectively.

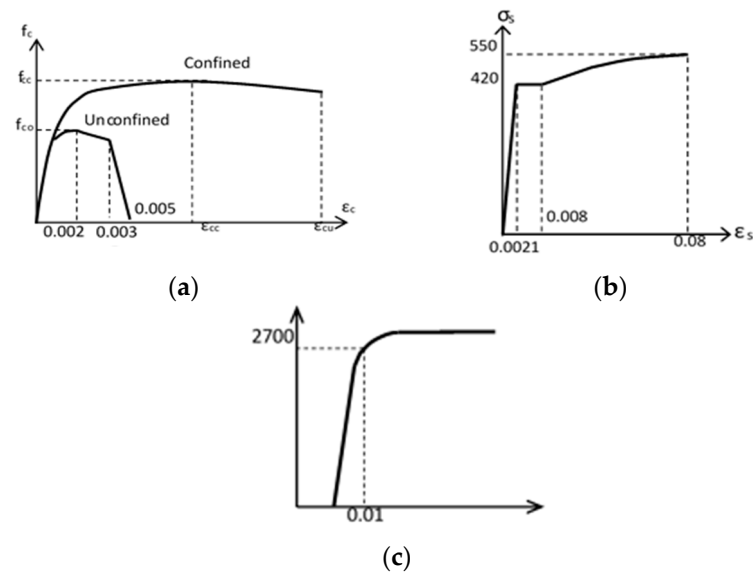


Figure 4. Stress–strain model of materials: (a) Mander Model for concrete; (b) stress–strain model with strain-hardening for reinforcement steel; (c) stress–strain model for post-tension tendons.

2.2. Ground Motions

The random nature of earthquakes makes the damage estimation problem probabilistic. Seven earthquakes with different magnitudes, PGA, and source distance have been used in this study to take the random nature of earthquakes into consideration. The soil class of the considered structure corresponds to Group C soil in the USGS classification. The USGS soil classification is based on the average shear wave velocity of all layers from the top layer of the soil to a depth of 30 m. According to this classification, the average shear wave velocity, V_{s30} , for Group C soil, up to a depth of 30 m, is between 180 m/s and 360 m/s. Table 1 shows the considered ground motions. All the selected ground motion records were recorded on Site Class C since the sample building is assumed to be constructed on this type of soil.

Table 1. Considered ground motions.

Event	Year	Station	Magnitude	Rjb (km)	Rrup (km)	V_{s30} (m/s)	Site Class
Kocaeli, Turkey	1999	Atakoy	7.51	56.5	58.3	274.5	C
Coalinga	1983	Parkfield—Cholame 5 W	6.36	47.9	48.7	289.6	C
San Fernando	1971	Gormon—Oso Pump Plant	6.61	44	46.8	308.4	C
Friuli, Italy	1976	Conegliano	6.5	80.4	80.4	274.5	C
Imperial Valley	1979	Coachella Canal #4	6.53	49.1	50.1	345.4	C
Victoria, Mexico	1980	SAHOP Casa Flores	6.33	39.1	39.3	338.6	C
Van, Turkey	2011	Muradiye	7.2	19.5	28.5	293	C

Seven earthquake records were selected for use in the analyses. Attention was paid to ensure that the magnitudes of the selected earthquakes were at least 6 and that there was a minimum distance of 40 km between the epicenter of the earthquake and the station where the record was taken. Thus, the near-fault effect was partially eliminated. The spectral match was carried out using the software of Seismomatch [34] by taking the requirement given in the Turkish Seismic Design Code [25]; the amplitudes of the average of the spectra of all selected records between the periods of $0.2T_1$ and $1.5T_1$ must not be smaller than

the amplitude of the design spectrum defined in TBDY in the same period range. T_1 is the period of the dominant mode. Figure 5 shows all the matched ground motion record spectra, design spectrum, and average spectrum.

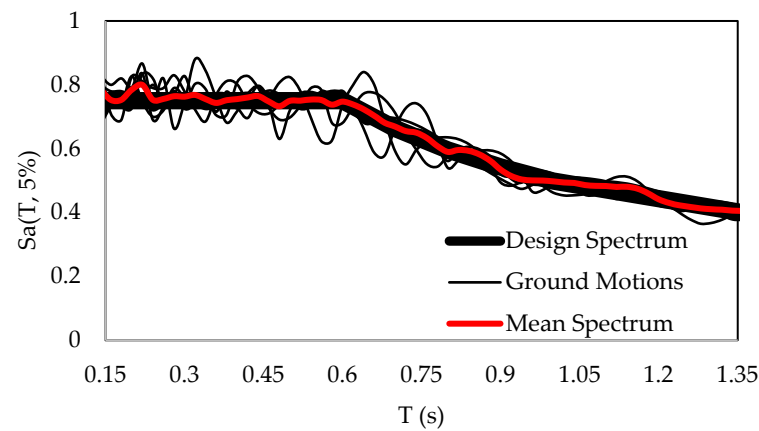


Figure 5. Design, mean, and matched ground motion spectra.

2.3. Damage Levels

There are 3 different levels of section damage—in other words, local damage criterion—defined by the 2018 version of the Turkish Seismic Design Code [25] for RC sections: Minimum Damage, Controlled Damage, and Collapse Prevention. Each one has a strain limit for concrete in compression and reinforcement steel in tension. Table 2 shows the corresponding limits of each sectional damage level defined by the aforementioned code, used for the determination of the damage level of columns. ω_{we} is the transverse reinforcement ratio in Table 2. There are also 3 section damage zones limited by the aforementioned damage limits. Those are Limited damage Zone, Significant Damage Zone, and Advanced Damage Zone.

Table 2. Strain limits of each section damage level for columns.

Section Damage	Concrete in Compression (ϵ_c)	Reinforcement (ϵ_s)
Limited damage	0.0025	0.0075
Controlled damage	$0.75(0.0035 + 0.04\sqrt{\omega_{we}})$	$0.75(0.4 \epsilon_{su})$
Collapse prevention	$0.0035 + 0.04\sqrt{\omega_{we}} \leq 0.018$	$0.4 \epsilon_{su}$

Figure 6 shows a schematic representation of the considered moment–curvature relationship of a post-tensioned beam section given by the Turkish Seismic Design Code [25] and corresponding deformation limits. M_1 is the moment where the prestress force is exceeded. M_2 is the moment capacity corresponding to 90% of the yield stress of post tension cables. θ_1 and θ_2 show the rotation capacity of the section at M_1 and M_2 , respectively.

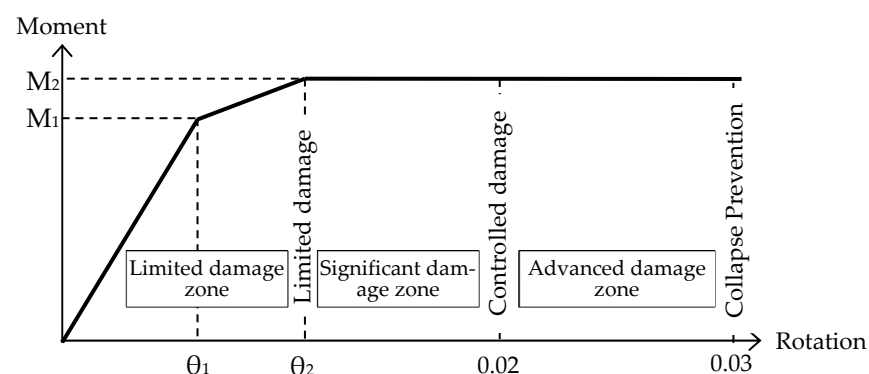


Figure 6. Typical moment–curvature curve of a post-tensioned beam section.

There are 3 different building performance levels whose qualitative definitions are given by the 2018 version of the Turkish Seismic Design Code [25] based on the percentage and distribution of the section damage levels given above. Those are Limited damage, Controlled Damage, and Collapse Prevention. A Limited Damage Performance Level is defined as limited nonlinear behavior in structural elements without requiring significant repair. A Controlled Damage Performance Level describes the significant damage without loss of life; however, there is still substantial margin against collapse and there is no decrease in base shear capacity at this performance level. Finally, a Collapse Prevention Performance Level shows incipient collapse. Quantitative definitions of the aforementioned building performance levels are based on the percentage of the structural element whose damage level exceeds the limits given in Table 2 and Figure 6 and distribution of damage along the height of the building. However, in this study, the minimum spectral acceleration level, which causes one of the specific section damages given above, as a result of Incremental Dynamic Analysis, is related to building performance levels and is assumed as the lower limit of the corresponding building performance level in terms of spectral acceleration level. Similarly, the corresponding interstory drift ratio obtained from the same analysis is accepted as the interstory drift ratio limit of the building performance level. Based on this approach; the interstory drift ratio limit of the Limited Damage Performance Level is assumed to be reached when the first beam or column section deformation exceeds the minimum damage limit given in Table 2. Similarly, when the Incremental Dynamic Analysis gives a section damage level of controlled damage at a column or beam section then the corresponding interstory drift ratio is assumed as limit of the Controlled Damage Performance Level. The Collapse Prevention Performance limit is the maximum interstory drift ratio corresponding to the first column section damage of collapse.

2.4. Incremental Dynamic Analysis

As mentioned before, the interstory drift ratio limits of the corresponding building performance levels were determined by incremental dynamic analysis using the SAP2000 computer package [35] and performance definitions given by the seismic code [25]. In addition to local damages explained in previous section, the interstory drift ratio of yielding and collapse limits as global damage criteria of the considered building have also been determined using incremental dynamic analysis so that upper and lower limit of damage can be determined. Incremental dynamic analysis (IDA) is a parametric analysis method that is useful for estimating the structural performance under several ground motions. The method is discussed comprehensively by Vamvatsikos and Cornell [36]. It is mainly based on producing one or more curves of damage measure versus intensity measure under the effect of scaled ground motions as a result of several nonlinear dynamic analyses. 5% damped elastic spectral acceleration is selected as the ground motion intensity measure since it is the main parameter that is used for the design of structures. Each ground motion is scaled up with respect to the individual spectral acceleration based on the corresponding fundamental period of the considered building. The Takeda hysteresis model was used for modeling the load–deformation relationship of columns. Figure 7 shows the origin oriented hysteretic load–deformation relationship used for the post tensioned beams with $\beta_1 = \beta_2 = 0$ based on the test results [28,30,31] and a sample hysteretic force–deformation relationship of a beam obtained with nonlinear time-history analysis.

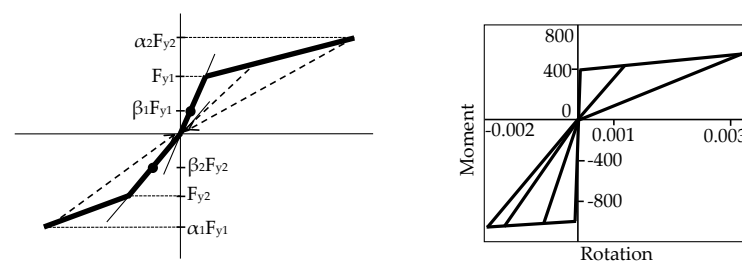


Figure 7. Origin-oriented hysteretic force–deformation relationship of a post-tensioned beam section.

Beams and columns are modeled as frame finite elements. The nonlinear behavior is modeled in the finite plastic deformation regions defined at the ends of the columns, with the lumped plastic hinge model. The length of the plastic deformation zone, called the plastic hinge length, is taken as equal to half of the section height as proposed by the Turkish Seismic Design Code [25]. The nonlinear zero length link element whose force–deformation relationship given in Figure 6 is used for beams. The schematic representation of the structural model can be seen in Figure 8.

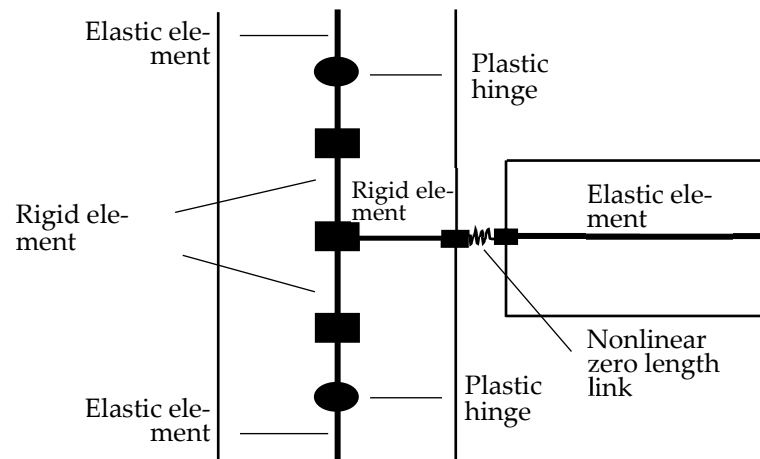


Figure 8. The schematic representation of the analytical model of a column–beam connection.

The Turkish Seismic Design Code 2006 [24] requires the use of the capacity design approach for the calculation of the required shear reinforcement of structural system elements as mentioned above. According to the capacity design principle, the design shear force is calculated based on the maximum probable moment capacities at the ends of structural system elements; thus, it is ensured that the structural system elements reach their ultimate strength through bending failure. That is why shear hinges were not taken into account. In addition, the dimensionless axial load level is limited to 0.4 f_{ck} by the code at the design stage. Therefore, it is reasonable to ignore the axial failure. Therefore, local damages such as shear or axial failure of structural system elements are not considered within the scope of this study.

The deformations induced by vertical loads were calculated by considering construction stages of the structure as given below:

- Installation of the 1st story beams following the erection of the prefabricated columns (at these stages the column–beam connections are not rigid)
- Installation of the 1st story slab panels
- Repeating the previous steps for each story (2nd to 7th)
- Post-tensioning of each story (column–beam connections are rigid)
- Casting of the topping concrete of each story
- Applying of live loads, partition walls, and finishing loads

Time history analyses have been carried out at the end of the loading stages given above. Up to the yield point, the relationship between spectral acceleration and the maximum interstory drift ratio is linear. The yielding capacity of the structure is defined as the spectral acceleration level at which the IDA curve leaves the linear path. When the structure reaches its collapse capacity, practically, an increase in intensity measure produces an infinite increase in damage measure. To determine the collapse capacity of the structure, ground motion is scaled up and several nonlinear dynamic analyses are carried out until the slope of the curve decreases to 20% of the initial slope of the IDA curve [36]. If dynamic instability as a result of a non-converging run is observed before the IDA curve slope decreases to the limit value mentioned above, the spectral acceleration level of the non-converging run is assumed as the collapse capacity. Figure 9 shows an

IDA curve obtained for Imperial Valley ground motion in the X direction of the considered building. Yielding and collapse points, obtained analytically, are indicated on the figure. The other damage levels defined by the Turkish Seismic Design Code [25] are also shown. Table 3 shows spectral acceleration of each damage level and corresponding interstory drift ratio values.

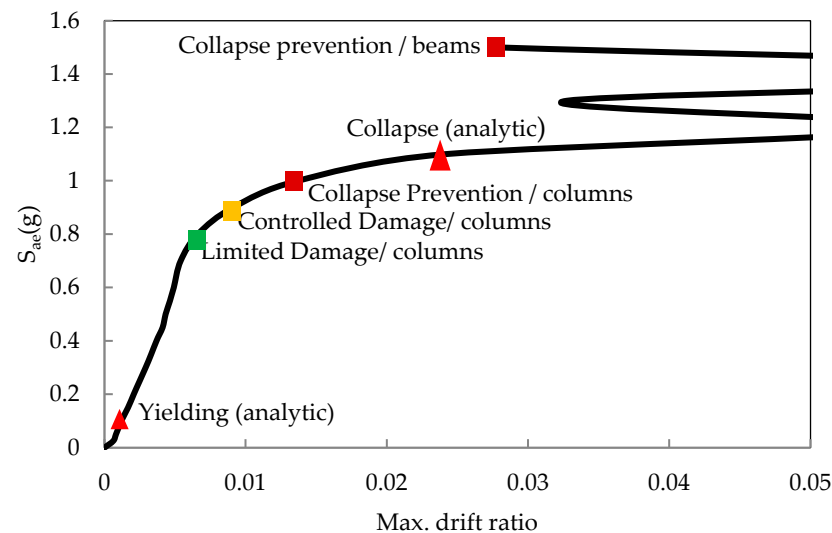


Figure 9. IDA curve obtained in the X direction for Imperial Valley ground motion.

Table 3. Spectral acceleration values of considered damage levels and corresponding interstory drift ratios for Imperial Valley ground motion in the X direction.

	Local/Global	$S_{ae}(g)$	Interstory Drift Ratio
Yielding (analytic)	Global	0.10	0.0012
Limited Damage (columns)	Local	0.80	0.0066
Controlled Damage (columns)	Local	0.90	0.0091
Collapse Prevention (columns)	Local	1.00	0.0137
Collapse (analytic)	Global	1.10	0.0240
Collapse Prevention (beams)	Local	1.50	0.0276

Figure 9 and Table 3 show that column sections reach their limits at lower level of S_{ae} compared to beam sections for the considered building. In other words, interstory drift ratio limits of the investigated building's performance levels are controlled by column strain limits, although it was designed as a ductile frame according to [24]. The interstory drift ratio of collapse is accepted as 0.024 for the ground motion of Imperial Valley ground motion in the X direction.

Figures 10 and 11 show all the IDA curves generated for two perpendicular directions of the considered building. The median interstory drift ratio of all considered building performance levels, determined following the definitions given by the Turkish Seismic Design Code [25], are given in Table 4.

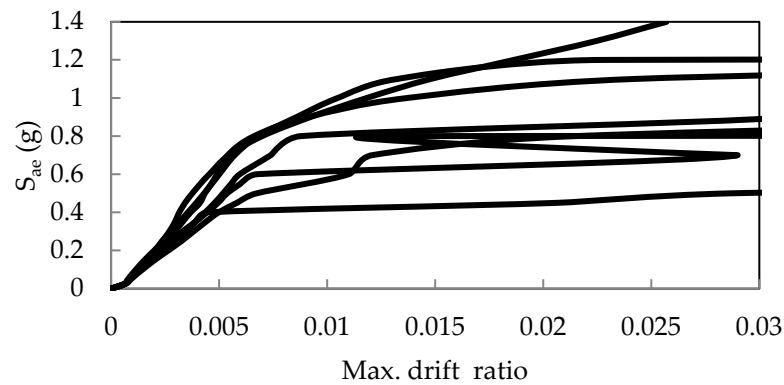


Figure 10. All IDA curves generated for X direction of the considered building.

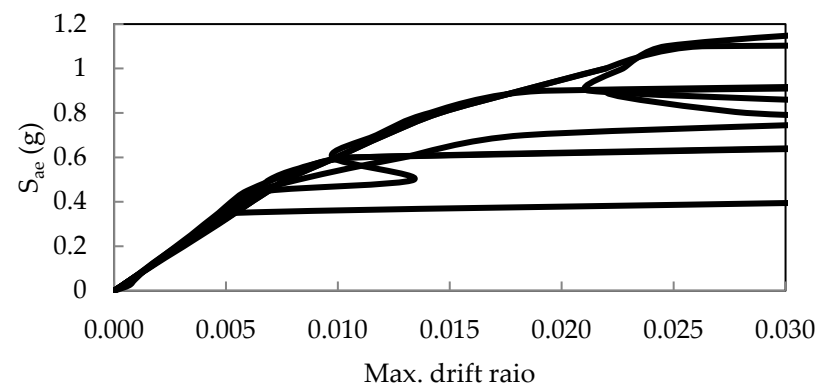


Figure 11. All IDA curves generated for Y direction of the considered building.

Table 4. Median interstory drift values of considered damage levels.

Damage Level	Local/Global	Median Interstory Drift Ratio	
		X	Y
Yielding	Global	0.0013	0.0029
Limited Damage	Local	0.0047	0.0054
Controlled Damage	Local	0.009	0.0077
Collapse prevention	Local	0.0137	0.0129
Collapse	Global	0.0228	0.0183

3. Fragility Curves

Fragility curves express the probability of structural damage due to earthquakes as a function of ground motion indices. In the present study, fragility curves are constructed in terms of elastic spectral acceleration based on the fundamental vibration period of the structure. The common assumed statistical distribution for damage probability in the literature is lognormal. Following this assumption, it is assumed that the fragility curves can be expressed in the form of two-parameter lognormal distribution functions. Based on this assumption, the probability of reaching or exceeding a limit state (LS) at a given earthquake intensity can be expressed as follows:

$$P(LS) = P(d_{LS} \leq d_{max}) = 1 - \Phi(r) \quad (1)$$

where Φ is the standard normal distribution and d_{LS} and d_{max} are limit state capacity and maximum demand, respectively. The standard normal variant r can be expressed as follows

$$r = \frac{\ln d_{LS} - \lambda_D}{\sqrt{\zeta_{LS}^2 + \zeta_D^2}} \quad (2)$$

ζ_{LS} is the lognormal standard deviation of a damage limit and is assumed to be 0.3 in this study [37]. λ_D is mean value of lognormal distribution and can be expressed in terms of the mean of the maximum response ($\bar{d}_{max}$), which is obtained from the time history analysis, and (ζ_D) is the dispersion of maximum demand. The mean of the maximum response and λ_D can be expressed as given in Equations (3) and (4)

$$\lambda_D = \ln \bar{d}_{max} - \frac{\zeta_D^2}{2} \quad (3)$$

$$\ln \bar{d}_{max} = \frac{1}{n} \sum_{i=1}^n \ln d_{max,i} \quad (4)$$

The dispersion of maximum demand, ζ_D , is obtained as the combination of uncertainties associated with demand estimation as follows:

$$\zeta_D = \sqrt{\ln \left(1 + \left(\frac{\sigma_r}{\bar{d}_{max}} \right)^2 \right) + \ln \left(1 + \left(\frac{\sigma_c}{\bar{d}_{max}} \right)^2 \right) + \ln \left(1 + \left(\frac{\sigma_D}{\bar{d}_{max}} \right)^2 \right)} \quad (5)$$

where σ_r and σ_c are the standard deviations due to randomness in earthquake records and material properties, respectively. σ_r was considered by using 7 earthquakes. Jeong and Elnashai [37] showed that the mean of the earthquake demand of 30 sample buildings, produced to consider the effect of the material uncertainty, is close to the response of the building that has the mean material strength. Thus, σ_c was ignored and mean material strength values are taken into account in this study. σ_D can be expressed as follows:

$$\sigma_D = \left[\frac{1}{n-1} \sum_{i=1}^n (\ln d_{max,i} - \ln \bar{d}_{max})^2 \right]^{0.5} \quad (6)$$

Detailed information for the construction of fragility curves can be found in Kazemi et al. [22]. Figure 12 shows the fragility curve of the Limited Damage Performance Level obtained for the X direction of the considered building.

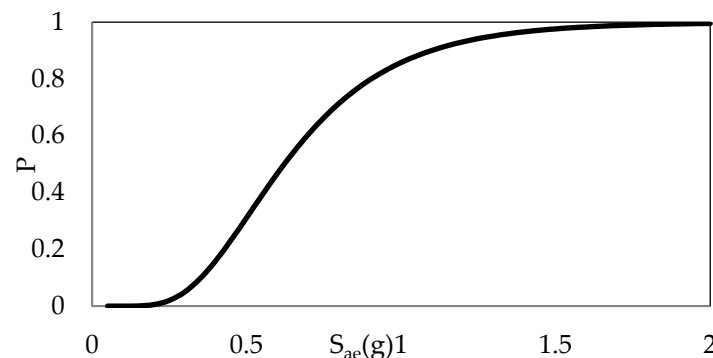


Figure 12. Fragility curve of the Limited Damage Performance Level obtained for the X direction of the considered building.

The method proposed by Shinozuka et al. [38] was used for combining fragility curves obtained in the X and Y directions of the building. The method is based on the statistical combination of the curves obtained for the X and Y directions, depending on the probability of earthquakes in the corresponding directions. In this study, earthquake probabilities are assumed to be equal and 50%. The combined fragility curve can be constructed using the following equation:

$$F_c(z) = P_1 F_1(z) + P_2 F_2(z) \quad (7)$$

where $F_c(z)$ is the combined fragility curve, and $F_1(z)$ and $F_2(z)$ are fragility curves of the considered building obtained in the X and Y directions, respectively. P_1 and P_2 are the

probability of having an earthquake in either the X or Y direction. Finally, the mean and standard deviation of the combined fragility curves can be expressed as follows:

$$\lambda_c = P_1\lambda_1 + P_2\lambda_2 \quad (8)$$

$$\zeta_c^2 = P_1\zeta_1^2 + P_2\zeta_2^2 + P_1(1 - P_1)\lambda_1^2 + P_2(1 - P_2)\lambda_2^2 - 2P_1P_2\lambda_1\lambda_2 \quad (9)$$

Figure 13 shows the combined fragility curves of the Limited Damage Performance Level. The lognormal distribution is no longer valid for the combined fragility curve. However, it is still reasonable to assume that the combined curve is lognormal with the mean and standard deviation estimated by (8) and (9), respectively [38].

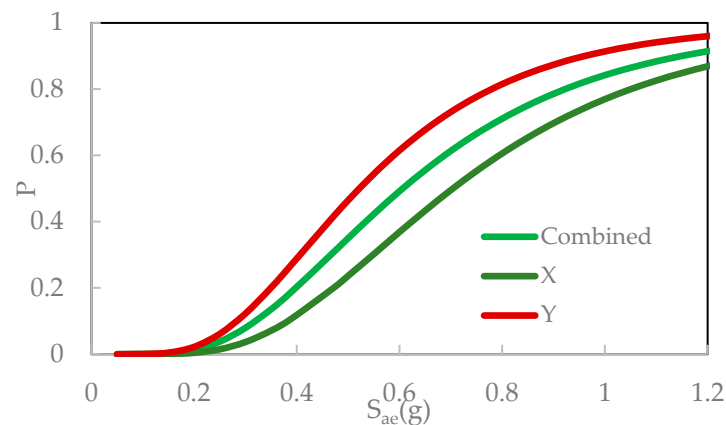


Figure 13. Combined fragility curve of the Controlled Damage Performance Level.

Combined fragility curves for yielding, collapse, and all considered damage levels are shown in Figure 14 in terms of elastic pseudo spectral acceleration.

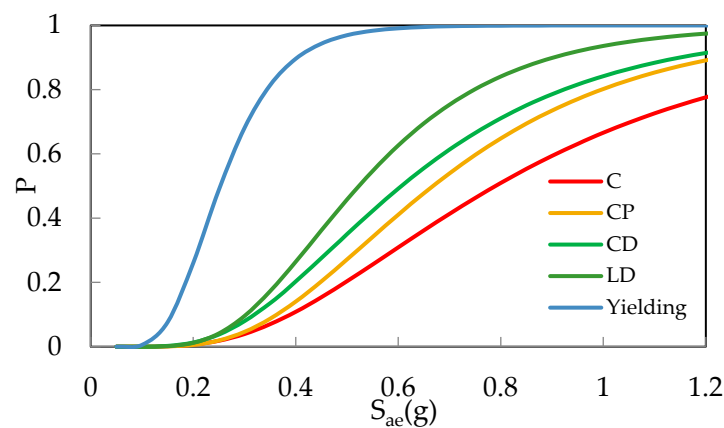


Figure 14. Combined fragility curves of all the considered performance levels.

4. Results

The aim of this study is to investigate the consistency of the analytic and expected performance of a post-tensioned frame structure through fragility curves. Figure 15 shows schematic representation of building performance levels on a schematic base shear-top displacement representation based on the definitions given by the Turkish Seismic Design Code [25].

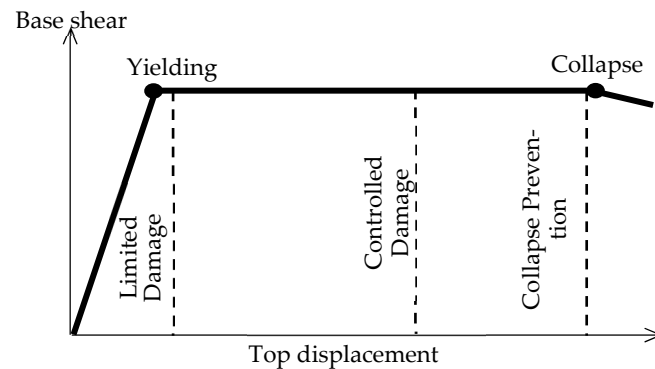


Figure 15. A Schematic representation of building performance levels based on the definitions given by the Turkish Seismic Design Code.

The Turkish Seismic Design Code [25] defines three ground motion level with 50%, 10%, and 2% probability of being exceeded in 50 years as Minimum Earthquake, Design Earthquake, and Maximum Earthquake, respectively. For a residential building, the target performance defined by the Code [25] is the Controlled Damage (CD) for the Design Earthquake. From the point of an earthquake-resistant design philosophy, a residential building should also meet the Limited Damage (LD) and Collapse Prevention (CP) Performance Level requirements for Minimum Earthquake and Maximum Earthquake, respectively.

Table 5 shows the probabilities of being exceeded for aforementioned Building Performance Levels, obtained under the effect of ground motion levels defined above. Table 5 also shows the yield and collapse probabilities—as global damage levels—to determine the upper and lower limits of damage. In Table 5, the underlined values indicate the probabilities of exceeding the previously explained target performance levels. In this study, the acceptable level of exceeding a damage level is assumed as 10%. Although it is subjective, authors believe that the assumed value is realistic.

Table 5. Probability of exceedance of considered performance levels.

	Minimum Earthquake		Design Earthquake		Maximum Earthquake	
	X	Y	X	Y	X	Y
Yielding	<u>0.92</u>	0.87	0.99	0.99	1.00	1.00
LD	<u>0.11</u>	<u>0.049</u>	0.66	0.51	0.91	0.84
CD	0.09	0.048	<u>0.52</u>	<u>0.39</u>	0.81	0.70
CP	0.054	0.026	0.44	0.31	<u>0.76</u>	<u>0.64</u>
Collapse	0.045	0.023	0.33	0.23	<u>0.62</u>	<u>0.50</u>

5. Discussion

Firstly, the consistency of the analytically obtained lower and upper limits of the damage with the damage limits given by the code will be discussed. Considering the definition of the Limited Damage Performance Level and the fact that the yield capacity determined by IDA represents the beginning of stiffness reduction, the probability of exceeding this performance level and the probability of yielding are expected to be close to each other for the Minimum Earthquake. As can be seen from Table 5, the probabilities of reaching yielding are 92% and 87% in the X and Y directions, respectively, while probabilities of exceeding the Limited Damage Performance Level are 11% and approximately 5% in the same directions. It is consistent that the probability for the yielding case is higher than the probability of exceeding the limited damage performance level. However, the difference is higher than expected under the effect of a minimum earthquake. Global and local damage criteria do not yield consistent results at this performance level. The difference is unreasonable.

There is a reasonable margin between the probability of reaching a collapse state (62% and 50%) and that of Collapse Prevention Performance Level (76% and 64%) for the

Maximum Earthquake. The global and local damage criteria give consistent results at this performance level; however, this observation must be verified by examining more sample structures with different structural system types and geometry.

As mentioned above, a structure designed according to the Turkish Seismic Design Code [24] is expected to not be damaged under the effect of the Minimum Earthquake level; even if it is damaged, the level of this damage is expected to be compatible with the Limited Damage Performance Level requirements. In other words, the building is expected to survive the minimum earthquake without damage or with minor damage in structural system elements. As can be seen in Table 5, the probability of exceeding the Limited Damage Performance Level in the X direction is 11%, which is slightly higher than the acceptable level.

At the ground motion level of a Design Earthquake, the probability of exceeding the Controlled Damage Performance Level is 52% and 39% for the X and Y directions, respectively. Those values are significantly higher than the acceptable level. Even the probability of exceeding the collapse limit is not reasonable (33% and 23%) at this ground motion level. Furthermore, the probability of exceeding the Collapse Prevention Performance Level is higher than acceptable: 76% and 64% for the Maximum Earthquake.

6. Conclusions

The following conclusions can be drawn from the results of this study.

- The probability of yielding and probability of exceeding the limited damage performance level are consistent under the effect of minimum earthquake. However, the difference is higher than expected under the effect of the minimum earthquake.
- There is a reasonable margin between the probability of reaching a collapse state and that of the Collapse Prevention Performance Level under the effect of a Maximum Earthquake. The global and local damage criteria give consistent results at this performance level.
- The probability of exceeding the Limited Damage Performance Level in the X direction for the minimum earthquake is slightly higher than the acceptable level.
- At the ground motion level of the Design Earthquake, the probability of exceeding the Controlled Damage Performance Level is significantly higher than the acceptable level. Even the probability of exceeding the Collapse Prevention performance level and probability of collapse are not reasonable under the effect of the Design Earthquake.
- The probability of exceeding the Collapse Prevention Performance Level is higher than acceptable for the Maximum Earthquake.

Those differences in damage probability indicate the importance of the damage criteria and corresponding limits used for the determination of damage probability. Damage states or performance level limits must be selected carefully and, at least, upper and lower limits (collapse and onset of yielding) must be determined analytically so that realistic estimations can be made. Finally, it should be noted that the results obtained and observations made within the scope of this study are limited to the building type taken into consideration within the scope of this study. Even a change in the number of stories of a similar type of building may yield different results than those obtained in this study. However, a significant change is not expected in the conclusions. Future studies can be made by increasing the number of ground motion and building samples. The near-fault effect can also be investigated. Conducting similar studies for other building types of the existing building stock will be useful for the realistic estimation of the earthquake risk of the existing structures.

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