Available online at www.CivileJournal.org

Civil Engineering Journal

(E-ISSN: 2476-3055; ISSN: 2676-6957)

Vol. 8, No. 03, March, 2022

Seismic Assessment of Tall Buildings Designed According to the Turkish Building Earthquake Code

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Received 14 November 2021; Revised 29 January 2022; Accepted 08 February 2022; Published 01 March 2022

Abstract

For the first time, the 2018 edition of the Turkish Building Earthquake Code has added a dedicated chapter for the design of high-rise buildings in earthquake-prone areas. Keeping in view the widely practised design option of rigid shear walls at the centre of a high-rise structure, the latest code has additionally defined limits for shear-wall axial forces in high-rise buildings. The new shear-wall axial force limits have not been independently investigated for optimal design and criticality. This calls for a detailed investigation of the newly defined axial force limits for the design of high-rise buildings in Turkey, where seismic activity has historically remained high. This study, therefore, investigates the effect of variation in limit values of shear wall axial forces on the collapse prevention of such buildings. A high-rise building designed entirely according to the code was chosen as the base model. The location of the building is in Istanbul, which has the highest number of tall buildings as compared to other cities in Turkey. A total of 7 alternative models were created by changing the concrete material class and the thickness of shear walls. This approach allowed us to quantify the effect of shear-wall thickness and its criticality against another important design consideration, i.e., the compressive strength of concrete. Forty different earthquake ground motion records were used to analyse the models to determine how critical the axial force ratio of the shear walls is in terms of collapse probability. The method proposed in the Federal Emergency Management Agency (FEMA) document FEMA P695 was followed to determine the collapse levels for the high-rise structures. A nonlinear analysis was performed to analyse the failure safety of the models. Results indicate that an increase or decrease in the axial force ratios by more than 15% renders the structure either overdesigned or deficient.

Keywords: High-Rise Buildings; Shear Walls; Nonlinear Analysis; TBDY-18; FEMA P695.

1. Introduction

In recent years, Turkey has seen a significant increase in the number of high-rise buildings constructed for commercial and residential purposes. There are many buildings with a total height of more than 200 meters in Turkey. Before 2019, there was no national guideline/code for the design of high-rise buildings. The 2018 edition of the Turkish Building Earthquake Code, i.e., TBDY-18 prepared by AFAD [1], has covered that deficiency by adding a dedicated chapter for the design of high-rise buildings. One of the main differences between the latest Turkish earthquake code and the previous codes, is that the limits for shear-wall axial forces are now specified in addition to column axial forces.

Generally, in high-rise buildings, a substantial proportion of the horizontal loads are taken by core walls. The core walls are generally composed of reinforced concrete elements surrounding elevators, stairs, and utility installation spaces—the thickness of these core walls increases due to the axial force limitations on such walls. The core walls' thickness increases as the number of floors increases, and they, therefore, consume large areas of the entire floor plan.

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http://dx.doi.org/10.28991/CEJ-2022-08-03-011

Several studies have investigated the seismic performance of shear walls in buildings with varying configurations. The primary purpose of the studies conducted in line with the Federal Emergency Management Agency – FEMA-P695 [2] is to measure structures' earthquake performance. Previously, Gogus and Wallace [3] applied the FEMA P695 methodology to evaluate the seismic performance of reinforced concrete walls in buildings designed according to the specifications of the American Concrete Institute, ACI [4]. The study, however, was limited to buildings with a maximum of 12 stories. The collapse assessment was done by evaluating the initial trial response modification factors by comparing adjusted collapse margin ratios (ACMR) and acceptable values. The FEMA P695 procedure has also been used to assess collapse in unique moment-resisting frames by Haselton et al [5]. The authors also aimed to benchmark the FEMA P695 methodology for performance assessment of code-conforming buildings built according to the specifications of the American Society of Civil Engineers, ASCE [6] and ACI [4].

Li et al. [7] used the FEMA-P695 methodology to investigate in detail the collapse risk assessment of buildings installed with shear links. A finite element model was developed to study the non-linear dynamic response of the considered structure under different earthquake shaking intensities. The results showed that the shear links have very good seismic performance. The results also showed that the performance based designed shear links have a sufficient margin against structural collapse. The seismic performance of the 3D panel systems, according to FEMA P695, has also been investigated by Mashal and Filiatrault [8]. The study concluded that the performance evaluation of such systems could be done in the same way as for reinforced concrete elements. It was reported in their study that 3D panel archetypes having more than three stories did not pass the FEMA P695 acceptance criteria for collapse assessment. Ezzeldin et al. [9] examined the collapsing risk of reinforced masonry shear walls under maximum credible earthquake. The study modeled 20 archetypes of reinforced masonry shear walls with boundary elements and was evaluated using the FEMA P695 guidelines. The experiments showed that the tested models satisfy the requirements of stiffness, strength, and inelastic deformation under cyclic loading. The seismic response factors for eccentrically braced frames were evaluated by employing FEMA P695 methodology in a study conducted by Kusyilmaz and Topkaya [10]. Different archetypes were evaluated, and the link rotation angles were reported to exceed the limits specified by ASCE. All archetypes were redesigned using modified parameters and reevaluated using the FEMA P695 guidelines. The results from the redesigned models showed that the modifications were adequate and sufficient. The seismic performance of six and twelve stories staggered walls has also been evaluated based on FEMA P695 guidelines. The adjusted collapse margin ratios (ACMR) of the studied structures were obtained from incremental dynamic analyses. Lee and Kim [11] reported these values to be larger than the limit state values specified by FEMA P695. Khojastehfar et al. [12] studied the seismic risk analysis of concrete moment-resisting frames in a near-fault earthquake scenario. The engineering demand parameter (EDP) in this study was taken as the maximum inter-storey drift ratio (MIDR) as specified in FEMA P695. The sampled frames in this study were, however, subjected to incremental dynamic analysis to get a probabilistic demand model.

The FEMA P695 guidelines have also been used to evaluate structures designed with four different editions of the Iranian Code of Practice for Seismic Resistant Design of Buildings. In a study by Sadeghpour and Ozay [13], the design reliability and seismic performance factors provided in the second and third edition of the aforementioned code were evaluated. The performance and seismic characteristics of the seismic design codes were evaluated by using the FEMA P695 methodology. As mentioned in FEMA P695 methodology, incremental dynamic analyses were conducted on a set of 48 reinforced concrete structural systems. The collapse margin ratios (CMR) and acceptable collapse margin ratios (ACMR) of FEMA P695 used in this study have been widely used in previous investigations of similar nature. For instance, Siddiquee et al. [14] evaluated the seismic response modification factor for concrete frame structures with shape memory alloy, based on an acceptable collapse margin ratio as given in FEMA P695. Similarly, Gallo et al. [15] used the CMR criterion to evaluate the seismic collapse evaluation of pre-retrofit and post-retrofit reinforced concrete structures.

The studies mentioned above, and others have focused on employing the FEMA P695 procedure for evaluating seismic performance of structures with different configurations and built according to local building codes. Until recently, there was no special guidelines for the design of high-rise structures in Turkey. With the latest Turkish Building Earthquake Code of 2018 dedicating a full section to design of such buildings, it is imperative to investigate the performance of buildings designed with the latest building code according to well-developed assessment techniques e.g., FEMA P695 guidelines. The novelty of the latest TBDY-2018 also calls for study of its failure criteria with respect to design optimization. Most of the tall buildings built in Turkey, and especially in Istanbul, have a rigid shear core at its center with columns situated at the periphery. In relation with this, the latest Turkish earthquake code has design criteria and limits put on the axial forces in core shear wall. As per the authors' knowledge, no study has been carried out to investigate the seismic evaluation of tall structures built according to TBDY-18 within the framework of FEMA P695. Thus, it will be beneficial to apply the FEMA P695 methodology for collapse assessment of tall structures with shear walls at its core, designed per the guidelines of TBDY-18. This study addresses this research gap by quantifying the criticality of the shear wall axial force limitations of TBDY-18, particularly in terms of collapse prevention of such buildings. Additionally, a comparison of two design options (increased shear wall thickness and increased characteristic concrete compressive strength) is carried out to arrive at the most optimal design solution for tall buildings built according to TBDY-2018. Worldwide, building codes for structural design are mainly prepared by targeting various damage possibilities for different building types. The effect of the limitations on shear wall axial forces, as specified by TBDY-18, on the probability of collapse is a new issue that should be investigated. Detailed investigation of this issue will play an essential role in determining axial force ratios, wall thicknesses, and concrete class. It can be determined whether a high-rise building has reached the collapse level according to the criteria specified in the specification for a particular earthquake level by performing nonlinear analysis.

In this study, a high-rise building designed entirely per the code was chosen as the base model. This base model has been analyzed with many different scaled earthquakes. By repeating the analysis and changing the axial force ratio of the walls, it was observed how critical the axial force ratio of the walls is in terms of collapse probability. A flowchart of the research methodology has been presented in Figure 1. In this study, using the method proposed in FEMA P695, the collapse levels determined for the same high-rise structure with different axial force ratios were calculated. Two different methods were chosen while creating alternative models. One of the methods is creating alternative models through changing the concrete material class, and the other is through changing the thickness of the shear wall. The failure safety of different structural models created in this way was determined by performing nonlinear analysis with Perform 3D software. As described in FEMA P695, raw earthquake records were used for nonlinear analysis.

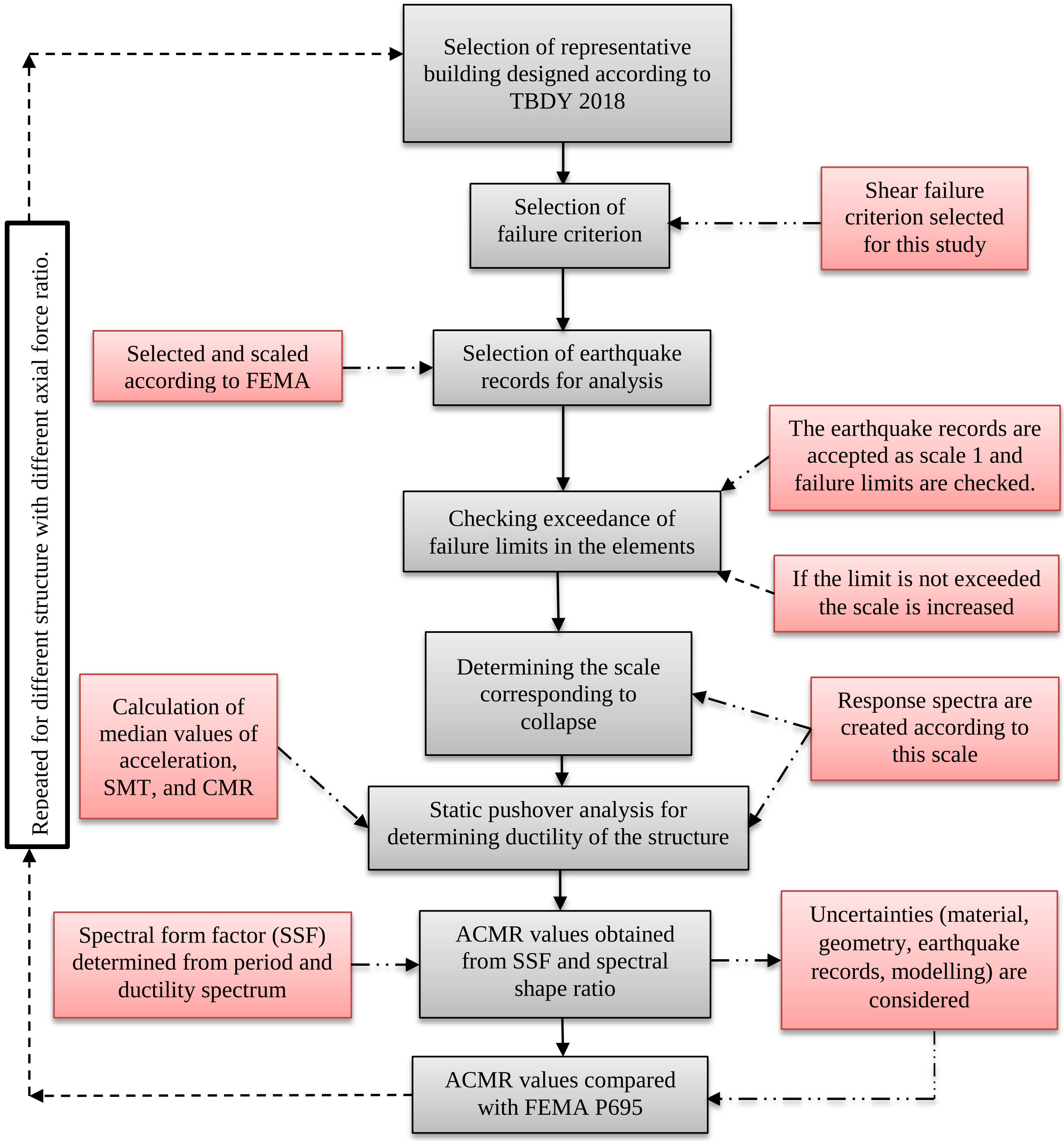


Figure 1. Flow chart of the methodology used in this study

2. Modelling Method

In this study, a tall building model that best represents the high-rise buildings inventory of Turkey was selected. The chosen representative building has a concrete core system, as is the case in most high-rise buildings in Turkey.

2.1. Creating Different Structural Models

The model structures were designed per the axial load limit specifications of TBDY-18 to obtain control analysis. In models, designs were made with axial load limit values different from those specified by the code with varying concrete strength and shear wall thickness. It is possible to change the axial force ratios of the shear walls in two ways without changing the number of floors and geometry. One of these ways is to change the section thickness, and the other is to use different concrete classes. The structure chosen as the base model in this study has a system consisting of tie-beam core walls and columns at the periphery, widely used in local design practices. Since the effect of varying axial forces in the shear walls on the structure's safety will be investigated, a nonlinear analysis model was created only for the core wall by considering the entire mass. This simplification of the model shortens the computational time for analysis of the model. Since the shear walls in the building's concrete core are connected with tie beams (as shown in Figure 2), analyses have been carried out in the x-x direction.

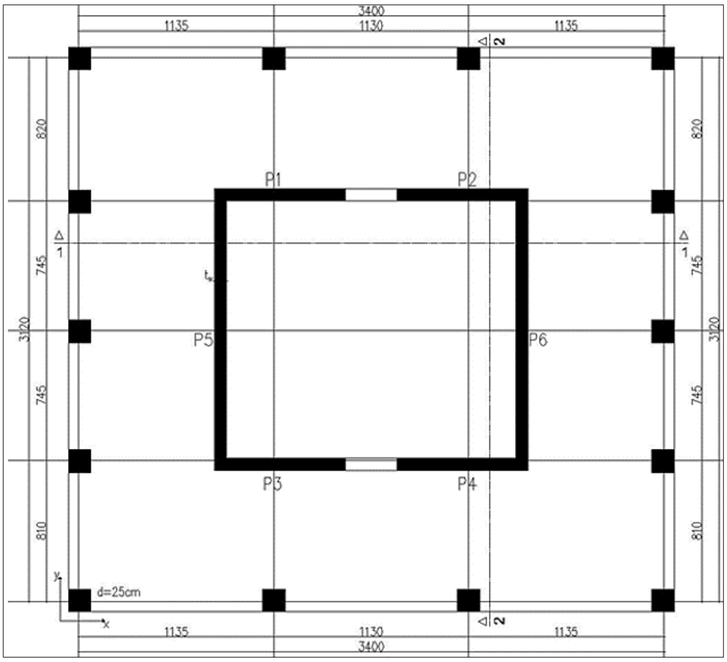


Figure 2. Plan of the building used for analyses (all dimensions are in centimeters)

In Table 1 below, the model named C45_70 represents the selected base model. The parameter t_w given in Table 1 represents the thickness of core walls for every 10 stories of the 40-story building (e.g., for the base model, the shear wall thickness for; the first 10 stories are 70cm, for the following 10 stories is 60cm, for the next 10 stories is 50cm, and the top 10 stories are 40cm). P/f_{ck} is the ratio of axial load (calculated under vertical and earthquake loads i.e., G+Q+E) to the total bearing capacity of the shear wall (calculated using the characteristic concrete strength). Figure 3 shows the 3D model of the structure.

Table 1. Details of different models considered for analyses

Model	Concrete Class	t_w (cm)	P/f_{ck}
C40_60	C40	60,60,50,40	0.45
C40_70	C40	70,60,50,40	0.40
C45_60	C45	60,60,50,40	0.40
C45_70(B)	C45	70,60,50,40	0.35
C50_60	C50	60,60,50,40	0.35
C50_70	C50	70,60,50,40	0.30
C60_60	C60	60,60,50,40	0.30
C60_70	C60	70,60,50,40	0.25

Figure 3. 3D model of the structure used for analyses

2.2. Modeling Nonlinear Element Behaviors

In the 3D model used in this study, fiber elements were used to model the walls and bar elements to model tie beams for nonlinear analysis. The fiber element model presented by Mazza and Vulcano [16] is the most common nonlinear model among distributed plasticity models. In this model concrete and steel sections are defined by dividing them into fibers. Since the behavior of reinforced concrete is defined by the uniaxial deformation of concrete and rebars, the fiber element model is defined using the stress-strain relationship of reinforced concrete. The idea of modeling with fiber elements, first put forward to reveal the cyclic behavior of beams, was transformed into an effective shear wall modeling tool proposed by Mander et al. [17].

Following the successful modeling of shifts in the neutral axes of the walls, this method was employed to create an axial load-bending-shear relationship in good agreement with experimental data. The method took its final form after a successful shear-bending relationship was presented by Kolozvari et al. [18] using this method. Although the fiber model is much more advanced than the lumped plasticity hinge model, its use in beams and columns is not very practical considering the time required for analysis. However, it is preferred because it effectively models regular and irregular (U or L) shear walls. As a result, the nonlinear behavior of the shear wall system to be examined in this study is defined using fiber elements. It should be noted that special attention has been paid to ensure that the inertial moment difference that will occur when dividing the walls into fiber elements is at the desired level (95% and above).

3. Results and Discussion

3.1. Evaluating Seismic Performance

The seismic performances of the building models, whose cross-section and material properties are given in Table 1, are evaluated according to FEMA P695 guidelines. Firstly, the earthquake records, taken from the database of PEER [19], to be used in the nonlinear analysis are scaled according to the velocity median as recommended by the FEMA P695 document. The median velocity of the earthquake records is calculated as per the given guidelines in FEMA P695. Then, the ratio of each earthquake record to the median velocity value and the normalization rates were determined. In Table 2, the velocity median and the normalization factors for the earthquake records are given.

Table 2. Considered earthquake records from PEER (2015) and their normalization factors

No.	Record S. No.	Min. Frequency (Hz)	File name (horizontal direction)		PGA_{max} (g)	PGV_{max} (cm/s)	PGV-PEER	NMI*
			1st component	2nd component				
1	953	0.25	Northr/mul009	Northr/mul279	0.52	63	57.2	0.65
2	960	0.13	Northr/los000	Northr/los270	0.48	45	44.8	0.83
3	1602	0.06	Duzce/bol000	Duzce/bol090	0.82	62	59.2	0.63
4	1787	0.04	Hector/hec000	Hector/hec090	0.34	42	34.1	1.09
5	169	0.06	Impvall/h-dlt262	Impvall/h-dlt352	0.35	33	28.4	1.31
6	174	0.25	Impvall/h-e11140	Impvall/h-e11230	0.38	42	36.7	1.01
7	1111	0.13	Kobe/nis000	Kobe/nis090	0.51	37	36.0	1.03
8	1116	0.13	Kobe/shi000	Kobe/shi090	0.24	38	33.9	1.10
9	1178	0.24	Kocaeli/dzc180	Kocaeli/dzc270	0.36	59	54.1	0.69
10	1148	0.09	Kocaeli/arc000	Kocaeli/arc090	0.22	40	27.4	1.36
11	900	0.07	Landers/yer270	Landers/yer360	0.24	52	37.7	0.99
12	848	0.13	Landers/clw-ln	Landers/clw-tr	0.42	42	32.4	1.15
13	752	0.13	Lomap/cap000	Lomap/cap090	0.53	35	34.2	1.09
14	767	0.13	Lomap/g03000	Lomap/g03090	0.56	45	42.3	0.88
15	1633	0.13	Manjil/abbar-l	Manjil/abbar-t	0.51	54	47.3	0.79
16	721	0.13	Superst/b-icc000	Superst/b-icc090	0.36	46	42.8	0.87
17	725	0.25	Superst/b-poe270	Superst/b-poe360	0.45	36	31.7	1.17
18	1244	0.05	Chichi/chy101-e	Chichi/chy101-n	0.44	115	90.7	0.41
19	1485	0.05	Chichi/tcu045-e	Chichi/tcu045-n	0.51	39	38.8	0.96
20	68	0.25	Sfern/pel090	Sfern/pel180	0.21	19	17.8	2.09
					Median PGV-PEER		**37.2**	

Each of the 20 different earthquake records given in Table 2 consists of two components as in x-x and y-y directions. Since the analyses are made only in the x-x direction, the y-y components are considered separate earthquake records and used in nonlinear analyses. For this reason, analyses were made for a total of 40 earthquake records. The earthquake records were multiplied by the normalization factor for use in nonlinear analysis. The selection of earthquake records and their scaling has been done in line with the guidelines provided in FEMA P695 as has been done in many other previous studies. For instance, Wu et al. [20], in a very similar study on dynamic responses of adjacent high-rise buildings used a ground motion record set in accordance with FEMA P695 guidelines. Similarly, Rahgozar et al [21] in their study on linear model for fully self-centering earthquake-resisting systems used a far-field ground motion set as it proposed by the guidelines of FEMA P69535 guideline. This is a robust sample of ground motions for considering variability on record-to-record basis. And, therefore, this study also uses the same ground motion selection criteria. The seismic hazard map of Turkey shown the location of the inspected building in Figure 4.

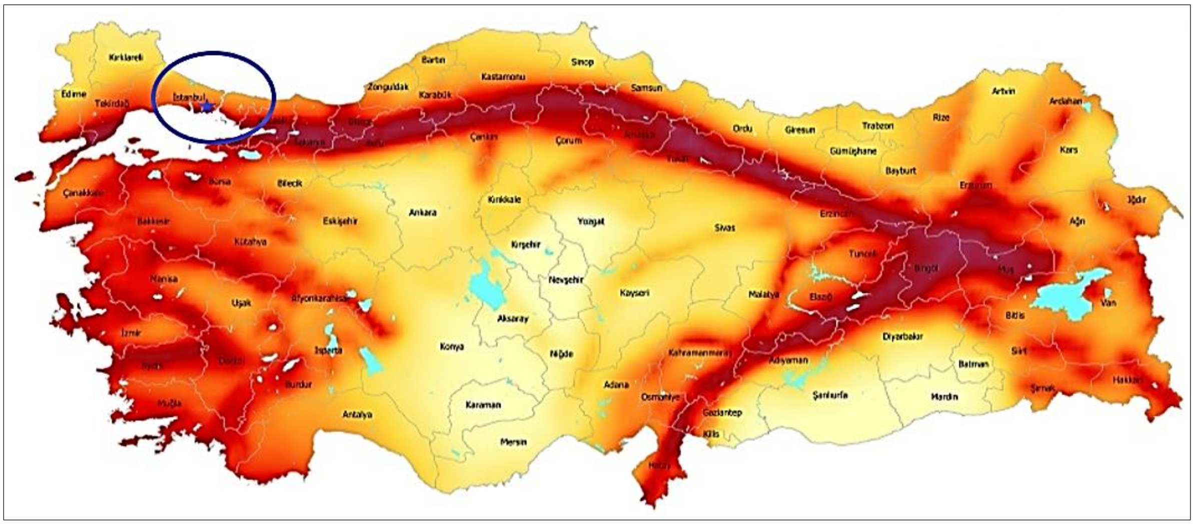


Figure 4. Seismic hazard map of Turkey [22] (The blue star shows the location of the inspected building)

3.2. Flat Slab Systems

During analysis of flat slab systems, the entire mass of the building is supposed to be carried by the shear wall elements. Therefore, a system consisting of only the shear walls and tie beams of the model is solved to analyze the structure. Since the columns are not connected to each other through beams, it is assumed that the seismic forces are taken entirely by the system of shear walls connected through tie beams. Thus, only the interior core comprising of shear walls is considered for analysis in line with the assumption that the seismic forces are taken entirely by the system of shear walls. A uniform slab thickness of 25 cm is chosen for all floors. All the columns at a floor are of same dimensions. The dimensions of columns and shear walls for different floors are shown in Tables 3 and 4 respectively.

Table 3. Dimensions of columns at different floor levels

Floor No.	Column Length	Column Width
01 to 10	130 cm	130 cm
11 to 20	110 cm	110 cm
21 to 30	90 cm	90 cm
31 to 40	70 cm	70 cm

Table 4. Dimensions of shear walls and tie-beams at different floor levels

Floor No.	Dimensions of tie beams in x-x direction (cm)	Dimensions of shear walls (cm)		
		P1-P3	P2-P4	P5-P6
01-10	70×100	760×70	1620×70	760×70
11-20	60×100	760×60	1620×60	760×60
21-30	50×100	760×50	1620×50	760×50
31-40	40×100	760×40	1620×40	760×40

3.3. Defining Collapse Limit

The analyses to be made aim to reach the collapse point for half of the earthquake records used (i.e., 20 earthquake records). This is achieved by increasing the scale of the first earthquake records set. For half of the records, the scale factor associated with the collapse limit is accepted as the scale factor corresponding to the building's collapse. One of the critical issues here is the definition of the collapse limit state. For this reason, three different collapse criteria were selected, and preliminary analyses were performed. These criteria are story drift, shear wall flexural deformation, and shear wall shear deformation.

The TBDY-18 requires shear walls to remain within the linear limits in terms of shear force carrying capacity. In the modeling of such shear elements, sliding behavior is also included in addition to bending to obtain closer performance to the actual one. The limit values given in Table 5 are taken from TBDY-18 in line with the purpose of this study. However, the limit value of shear strain for shear wall elements is the value taken as per FEMA P356, corresponding to the performance criterion for preventing collapse.

Table 5. Limit values for the collapse criteria

Collapse Criteria	Limit Value
Storey drift	0.0300
Bending deformation in shear wall (rebar)	0.0032
Bending deformation in shear wall (concrete)	0.0018
Shear deformation in shear wall	0.7500

As it is known, the limit values for story drifts are mostly given for the non-bearing secondary elements in structures with a rigid core. Therefore, it can be seen that these values for shear walls obtained from analysis stay well below the axial deformation and shear deformation criteria. Story drift may also become the primary failure criterion in systems with a combination of shear walls and frames or in systems consisting of frames only. For the first set of earthquake records, solutions were obtained by repeating the analysis with an increment of 0.5 in the scale factor until 6 was achieved. Analyses were made by monitoring both shear and axial strain limits. According to TBDY-2018, for nonlinear analysis of shear elements, shear deformation must be within linear limits. In this study, however, the shear strain is allowed to pass the nonlinear boundary, and it was, thus, determined that the model reached collapse due to shear strain first.

Moreover, it is evident from Tables 6 and 7 that the number of models failing due to exceedance of shear strain limit is higher than that failing due to axial strain. Therefore, as shear strain failure is more critical than axial strain it was decided to use shear strain as the governing failure criterion in the analysis.

Table 6. Number of failures due to exceeding shear deformation limits for the given scales

Scale	Shear Wall				Average
	P1	P2	P3	P4	
3.0	7	5	5	5	5.75
3.5	13	13	13	12	12.75
4.0	23	20	21	19	20.75
4.5	29	27	29	26	27.75
6.0	33	32	32	32	32.25

Table 7. Number of failures due to exceeding axial deformation limits for the given scales

Scale	Shear Wall								Average
	P1		P2		P3		P4		
	Flexure	Comp.*	Flexure	Comp.	Flexure	Comp.	Flexure	Comp.	
3.0	4	2	0	6	3	2	6	0	2.87
3.5	10	1	0	11	10	4	0	10	5.75
4.0	20	3	2	14	19	4	2	14	9.75
4.5	27	3	1	22	26	4	2	21	13.25
6.0	28	7	4	25	28	7	8	25	16.50

* Compression

As mentioned earlier, shear strain limit is taken as the governing failure criterion and henceforth, and Table 8 contains a summary of the results of the nonlinear analysis for all models failing in shear. The different models were created by changing the concrete strength and core wall thicknesses.

Table 8. Number of failures due to shear for each model

Model	Scale	Shear Wall				Average
		P1	P2	P3	P4	
C40_60	3.15	22	21	21	19	20.75
C40_70	3.60	22	18	22	18	20.00
C45_60	3.45	20	18	19	19	19.00
C45_70(B)	3.80	23	18	22	18	20.25
C50_60	3.65	22	18	22	17	19.75
C50_70	4.00	23	20	21	19	20.75
C60_60	3.70	20	19	19	19	19.25
C60_70	4.30	26	17	23	15	20.25

In the analyses made in x-x direction for shear walls P1, P2, and P3, P4 connected with tie beams, the number of earthquake records was determined concerning exceedance of shear deformation limits. As mentioned earlier, it is aimed that the average number of earthquake records for each shear wall exceeding the limits is half of the total number of earthquake records (i.e., 20 earthquake records). Scale values corresponding to these records are taken as earthquake scale factors, corresponding to the building's collapse.

3.4. Creating Response Spectra

In previous investigation by Gerami et al. [23], it was reported that for high-rise structures, under pushover analysis, selection of the response spectrum has significant influence on the resulting R factor. Therefore, the response spectra corresponding to the earthquake records were prepared by multiplying with the scale factors given in Table 8. The median value for spectral acceleration ($S_a(g)$) values, corresponding to the structure's period, is calculated. This median value is called the failure capacity intensity (S_{ct}). This process was repeated for all models, and the response spectrum prepared according to the base model is given in Figure 5.

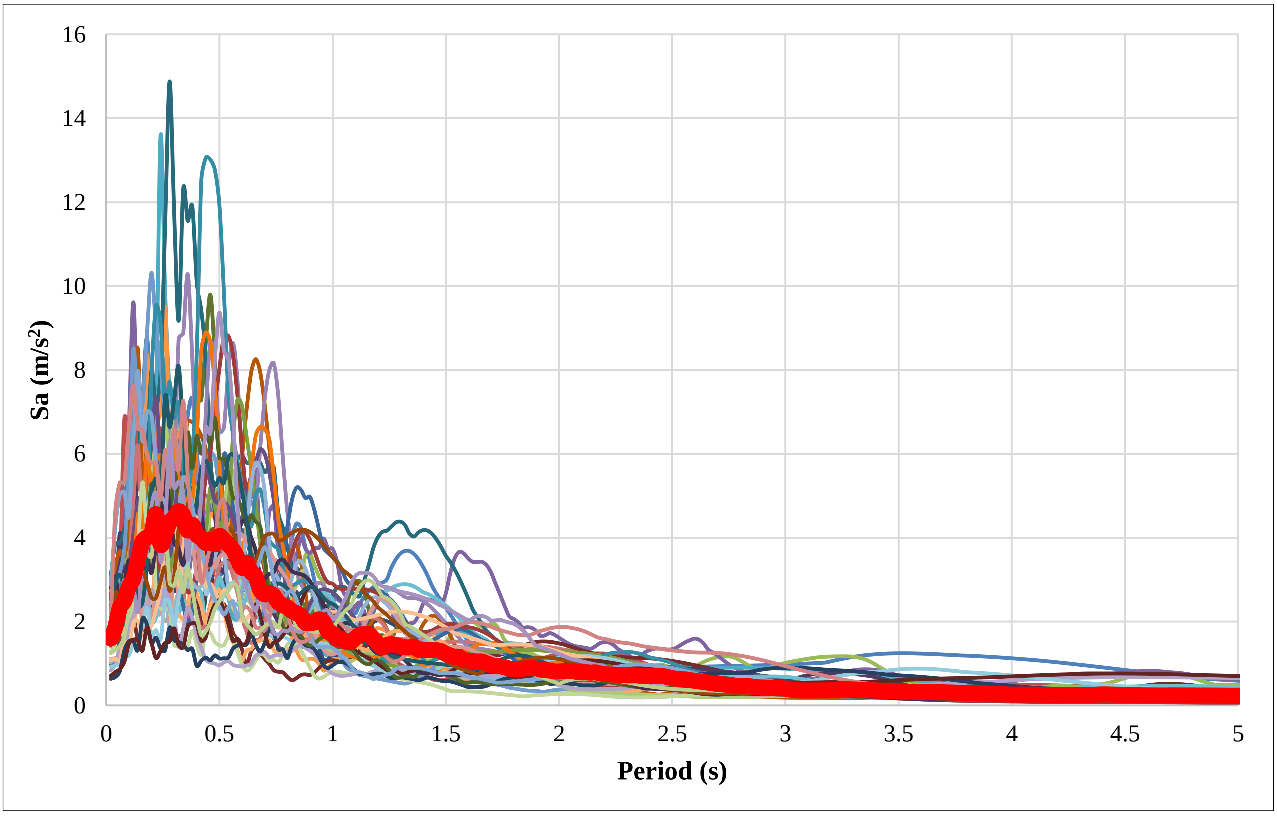


Figure 5. Response spectrum for the base model

3.5. Determination of Collapse Margin Ratio (CMR)

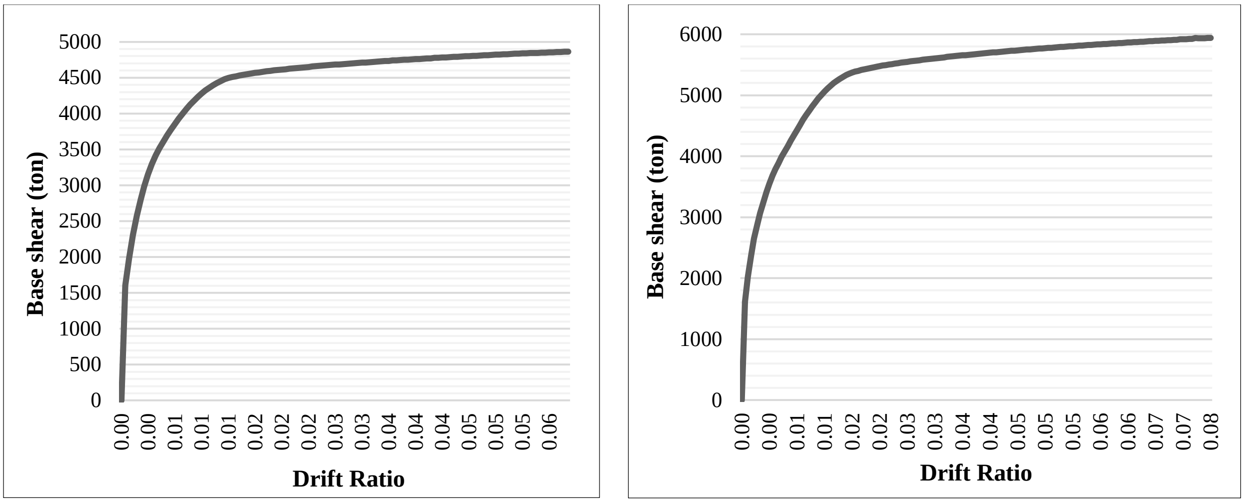
The spectrum used in the linear design phase is defined as the acceleration value (Smt) corresponding to the structure's dominant period in that direction. By calculating the ratio of Sct to Smt, the CMR is obtained. The damping ratio of 2.5% is used in the calculation of both values. Smt, Sct, and CMR (CMR = Sct /Smt) values for each model are given in Table 9.

Table 9. Calculated CMR values for all the models

Model	S_{mt}	S_{ct}	CMR
C40_60	0.137	0.178	1.302
C40_70	0.141	0.217	1.538
C45_60	0.141	0.209	1.479
C45_70(B)	0.144	0.220	1.527
C50_60	0.142	0.212	1.495
C50_70	0.146	0.235	1.605
C60_60	0.147	0.219	1.495
C60_70	0.146	0.261	1.783

3.6. Determining the Acceptable Collapse Limit Ratio

The acceptable collapse margin ratio (ACMR) is obtained by multiplying the CMR value by the spectral shape factor (S_{sf}) value. The S_{sf} value is selected from the Table 10 given in FEMA P695, depending upon the structure's period (T) and ductility (μ_T). Since the building period has to be defined for each model, the ductility must be calculated too. The nonlinear displacement (δ_u) and linear displacement ($\delta_{y,eff}$) values are determined by performing static pushover analysis. Their ratio gives the ductility value for any model considered. The pushover curves for two of the models are given in Figure 6. In Table 10, the values of δ_u, $\delta_{y,eff}$, μT, T, S_{sf}, CMR, and ACMR calculated for each model are given.



(a) (b)

Figure 6. Pushover curves for model C45_60605040 (a) and model C45_70605040 (b)

Table 10. S_{sf}, CMR and ACMR values for all 8 models

Model	δ_u	$\delta_{y,eff}$	μ_T	T	S_{sf}	CMR	ACMR
C40_60	0.0080	0.5220	2.45	4.571	1.2800	1.302	1.67
C40_70	0.0113	0.5785	3.13	4.376	1.3304	1.538	2.05
C45_60	0.0076	0.4927	2.47	4.446	1.2723	1.479	1.88
C45_70(B)	0.0109	0.5476	3.18	4.258	1.3344	1.527	2.04
C50_60	0.0074	0.4705	2.52	4.330	1.2768	1.495	1.91
C50_70	0.0106	0.5203	3.25	4.150	1.3368	1.605	2.15
C60_60	0.0068	0.4304	2.54	4.140	1.2786	1.495	1.91
C60_70	0.0081	0.4491	2.87	4.018	1.3083	1.783	2.33

The total system failure uncertainty given in FEMA P695 is denoted by β_{TOT}. The value of β_{TOT} consists of the resultant of four different uncertainties i.e., β_{RTR}, β_{DR}, β_{TD}, and β_{MDL}. Each of the mentioned uncertainties is discussed in detail in FEMA P695. In many previous studies on seismic assessment of reinforced concrete structures, these uncertainty factors of FEMA P695 have been shown to adequately predict the dispersion related to such uncertainties [24]. For this study, the β_{TOT} value is calculated by combining the 4 different uncertainty values for 8 different models. Table 11 includes all uncertainty values and total uncertainty values for each model.

Table 11. Value of total system failure uncertainty as per FEMA P695

Model	β_{RTR}	β_{DR}	β_{TD}	β_{MDL}	β_{TOT}
C40_60	0.35	0.20	0.20	0.20	0.49
C40_70	0.40	0.20	0.20	0.20	0.53
C45_60	0.35	0.20	0.20	0.20	0.49
C45_70(B)	0.40	0.20	0.20	0.20	0.53
C50_60	0.35	0.20	0.20	0.20	0.49
C50_70	0.40	0.20	0.20	0.20	0.53
C60_60	0.35	0.20	0.20	0.20	0.49
C60_70	0.39	0.20	0.20	0.20	0.52

3.7. Comparison of Calculated ACMR Values and Limit ACMR Values of FEMA P695

The total system failure uncertainty (β_{TOT}) is calculated using β_{TOT} values given in the FEMA P695 document. For this purpose, $ACMR_{10\%}$ and $ACMR_{20\%}$ values that should not be exceeded, depending on the β_{TOT} value, are selected from the said Table. Finally, a comparison of the calculated ACMR values of the models with the $ACMR_{10\%}$ values from FEMA P695 is presented in Table 12.

Table 12. Comparison of calculated and FEMA P695 specified values for ACMR

Model	$a_c.f_{ck}$	Period (sec)	β_{TOT}	μ_T	CMR	$ACMR_{10\%}$	$ACMR_{20\%}$	ACMR	$ACMR/ACMR_{10\%}$
C40_60	0.45	4.571	0.49	2.45	1.30	1.89	1.51	1.67	0.88
C40_70	0.40	4.376	0.53	3.13	1.54	1.97	1.56	2.05	1.04
C45_60	0.40	4.445	0.49	2.47	1.48	1.89	1.51	1.88	0.99
C45_70(B)	0.35	4.258	0.53	3.18	1.53	1.97	1.56	2.04	1.04
C50_60	0.35	4.330	0.49	2.52	1.49	1.85	1.51	1.91	1.03
C50_70	0.30	4.150	0.53	3.25	1.60	1.97	1.56	2.15	1.09
C60_60	0.30	4.140	0.49	2.54	1.50	1.88	1.50	1.91	1.02
C60_70	0.25	4.018	0.52	3.36	1.78	1.95	1.55	2.40	1.23

In this study, nonlinear analyses were carried out to investigate the effect of variations in axial forces on collapse safety. In these analyzes, the axial force value of $0.35f_{ck}$, as specified for the design of shear wall elements in TBDY-18, was taken as the basis. Subsequently, the effect of an increase or decrease on collapse limit was determined using the method suggested in the FEMA P695 document. The purpose of the FEMA P695 document is to determine the building behavior coefficient of any building group. For this reason, the ACMR value calculated for each structure is expected to exceed $ACMR_{20\%}$, and the average is expected to exceed $ACMR_{10\%}$ as specified by FEMA P695. In this study for high-rise buildings, a comparison is drawn between the performances of shear walls with different axial force ratios. It is aimed that the ACMR value of each building exceeds the $ACMR_{10\%}$ value as specified by FEMA P695. Since the ratio of $ACMR/ACMR_{10\%}$ for the C45_70 model, which meets all design criteria according to TBDY-18 and selected as the base model, is 1.04, the result is sufficiently close limit value as expected.

In this study, nonlinear analyses were carried out to investigate the effect of variations in axial forces on collapse safety. In these analyzes, the axial force value of 0.35fck, as specified for the design of shear wall elements in TBDY-18, was taken as the basis. Subsequently, the effect of an increase or decrease on collapse limit was determined using the method suggested in the FEMA P695 document. The purpose of the FEMA P695 document is to determine the building behavior coefficient of any building group. For this reason, the ACMR value calculated for each structure is expected to exceed $ACMR_{20\%}$, and the average is expected to exceed $ACMR_{10\%}$ as specified by FEMA P695. In this study for high-rise buildings, a comparison is drawn between the performances of shear walls with different axial force ratios. It is aimed that the ACMR value of each building exceeds the $ACMR_{10\%}$ value as specified by FEMA P695.

The main findings of the present study are as follows.

- The results indicate that instead of increasing the concrete strength, increasing the thickness of shear wall is more effective in keeping the axial stresses in the high-rise reinforced concrete core walls within the concrete characteristic strength limits. This is significant with respect to design optimization during the design of high-rise buildings in seismically active regions such as Istanbul.
- Another significant observation of this study is that according to the TDBY earthquake design, the nonlinear behavior is not taken into consideration in shear deformations in high-rise buildings, and thus the main design criterion is shear rather than axial deformation. In other words, the determining factor in shear wall design is primarily the shear force. This result of the study can also be interpreted in terms of the type of failure i.e., in core shear walls of a high-rise building, the failure of longitudinal reinforcement or the crushing of concrete due to excess axial forces do not occur unless the shear limit value is exceeded in a building designed according to TBDY 2018.
- A comparison with design and failure limits given in FEMA P695 indicate that the shear force limits used in this study and as stipulated by TBDY 2018, are in good agreement with those specified in FEMA P695. Additionally, the limit values remain within the optimal range i.e., the structure is not over-designed and shear force limit values are slightly above the recommended limit values as they should be.
- The findings are also significant in terms of their implications in the design and implementation phases of a project. The effect of any reduction in total area of shear-walls (e.g., reduction in their cross-sectional area due to provision for utilities etc.) on the failure safety has been determined more clearly. As long as the capacity doesn't decrease by more than 15%, as shown by the results of this study, the effect of gaps in the cross-sectional area that are imposed later in the implementation phase, has no effect on the failure safety of the building. Such gaps are usually requested by the architects or project owners during the implementation phase. This demand from architects tends to reduce the cross-sectional area of the core walls, after the structural design phase has been already completed. Therefore, the quantification of the effect of such gaps has been addressed in the findings of this study.

4. Conclusions

The following is a summary of the main conclusions of the present study:

- If the concrete class of the high-rise core wall, designed according to TBDY-18, is reduced by one level (equal to a reduction of 5 MPa in concrete strength) or the wall thickness is reduced by 10 cm, the structure remains within the recommended safety limits as a result of the change in axial stress.
- As compared to the 10 cm reduction in wall thickness of the core walls, the down gradation of the concrete class by 5 MPa had a lesser effect on the ductility ratio of the building. Therefore, the effect of concrete class down gradation on building safety was more negligible.
- Moreover, a 15% increase or decrease in the axial force limits, as defined for shear wall design in TBDY-18, does not significantly affect the collapse safety of the structure, i.e., the structure remains within acceptable safety limits. However, when the axial force ratio is increased or decreased by more than 15%, the structure is either overdesigned or deficient in providing the intended safety level.

This has significant implications in real-life design practices where the total cross-sectional area of core shear walls can be affected by any later modifications. Such practices are highly discouraged and should be avoided. However, to the authors' knowledge, many cases arise during the implementation phase, which makes the changes unavoidable. It is, therefore, recommended that should such a modification be unavoidable, the effect of the total reduction in cross-sectional area of shear walls during the implementation phase be checked to ensure that the reduction does not exceed 15% of the total shear-wall cross-sectional area in the originally designed plans.

The present study has the limitation of being carried out with the assumption that the entire seismic forces are resisted by only the central shear wall system. To better understand the efficacy of the new guidelines provided in TBDY-2018, it is recommended to analyze similar models but where the columns are connected through beams. In such a model, the columns and the shear walls will be modelled to resist the seismic loads instead of the idealization applied in this study. The authors confirm they will investigate this aspect as an extension of this work in the future.

5. Declarations

5.1. Author Contributions

Conceptualization, O.I.E. and T.A.O; methodology, T.A.O.; validation, T.A.O.: analysis, O.I.E.; investigation, O.I.E. and T.A.O.; resources, O.I.E.; writing, O.I.E.; writing-review, O.I.E. and T.A.O. All authors have read and agreed to the revised version of the manuscript.

5.2. Data Availability Statement

The data presented in this study is available on request to the corresponding author.

5.3. Funding

The authors received no financial support for the research, authorship, and/or publication of this article.

5.4. Acknowledgements

The authors would like to thank Fatih Yeşilselvi for his valuable input and review during the preparation of this study.

5.5. Conflicts of Interest

The authors declare no conflict of interest.

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