EFFECT OF STAIRCASES ON THE SEISMIC PERFORMANCE OF REINFORCED CONCRETE BUILDINGS

Master of Science Thesis

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Master of Science Thesis

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FINAL APPROVAL FOR THESIS

This thesis titled "EFFECT OF STAIRCASES ON THE SEISMIC PERFORMANCE OF REINFORCED CONCRETE BUILDINGS" has been prepared and submitted by Ayberk KARAASLAN in partial fulfillment of the requirements in "Eskişehir Technical University Directive on Graduate Education and Examination" for the Degree of Master of Science in Structural Engineering Department has been examined and approved on 19/11/2018.

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ABSTRACT

EFFECT OF STAIRCASES ON THE SEISMIC PERFORMANCE OF REINFORCED CONCRETE BUILDINGS

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Department of Civil Engineering

Programme in Structural Engineering

Eskişehir Technical University, Graduate School of Sciences, November,2018

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It is known that, during the structural analysis phase, staircases are not taken into account as structural members but as dead load. Staircases, as secondary structural members, not only serve for connecting separate layers but also provide considerable amount of strength and stiffness to the building which can cause the structure to behave differently rather than it is designed.

In this study, in order to examine the effect of staircases on the seismic response of reinforced concrete (RC) buildings, the analytical models were developed for different material properties, number of spans, the existence and location of staircases. Non-Linear Static Pushover Analysis (NSPA) and Non-Linear Time History Analyses (NLTHA) were conducted to compare several engineering demand parameters (EDPs) such as inter-story drift ratio (ISDR), floor accelerations, modal properties, member shear forces and plastic hinge distribution. In addition to these, short column effect, variation in shear forces of columns that are attached to the staircase slab, failure mechanism and nonlinear deformation in staircase models have also been studied. As the result of the performed study, it has been numerically proved that staircases affect the seismic behavior of RC buildings.

Keywords: Reinforced Concrete Buildings, Staircases, Seismic Behavior, Short Column, Plastic Hinge

ÖZET

MERDİVENLERİN BETONARME BİNALARIN SİSMİK PERFORMANSINA ETKİSİ

Ayberk KARAASLAN

İnşaat Mühendisliği Anabilim Dalı

Yapı Bilim Dalı

Eskişehir Teknik Üniversitesi, Fen Bilimleri Enstitüsü, Kasım 2018

Danışman: Doç. Dr. Özgür AVŞAR

Yapısal analiz aşamasında, merdivenlerin çoğunlukla taşıyıcı bir sisteme yapısal eleman olarak değil de, sadece zati yük olarak dikkate alındıkları bilinmektedir. Merdivenler, ikincil yapısal elemanlar olarak, iki ayrı kat seviyesini birbirlerine bağlamakla kalmayıp, yapıya dikkate alınmaya değer ölçüde rijitlik ve dayanım da sağlamaktadır ve bu durum yapının öngörülenden farklı bir şekilde davranmasına yol açabilmektedir.

Bu çalışmada, merdivenlerin betonarme binaların sismik davranışına etkisini incelemek için farklı malzeme sınıfı, kat adedi, açıklık sayısı ve merdiven konumuna sahip sayısal modeller Doğrusal Olmayan Statik İtme Analizi ve Doğrusal Olmayan Zaman Tanım Alanında Analizlere tabi tutulmuştur. Elde edilen sonuçlar göreli kat ötelenmeleri, kat ivmeleri, modal kütle katılım oranları, mod şekilleri, eleman kesme kuvvetleri ve plastik mafsal dağılımı açısından kıyaslanmıştır. Bunlara ek olarak, kısa kolon etkisi, merdiven sahanlık döşemesini taşıyan kolonlardaki kesme kuvveti değişimi ve merdivenli modellerdeki göçme mekanizması ve doğrusal olmayan deformasyon ayrıca incelenmiştir. Yapılan çalışmanın sonucunda, merdivenlerin binaların deprem davranışlarını etkilediği sayısal olarak ortaya konulmuştur.

Anahtar Kelimeler: Betonarme Binalar, Merdivenler, Sismik Davranış, Kısa Kolon, Plastik Mafsal

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I would like to thank my supervisor, Assoc. Prof. Dr. Özgür Avşar, for the patient guidance, encouragement and advice he has provided throughout my time as his student. I have been extremely lucky to have a supervisor who cared so much about my work, and who responded to my questions and queries so promptly.

I must express my very profound gratitude to my family for providing me with unfailing support and continuous encouragement throughout my years of study and through the process of researching and writing this thesis. This accomplishment would not have been possible without them.

Ayberk KARAASLAN

18/11/2018

STATEMENT OF COMPLIANCE WITH ETHICAL PRINCIPLES AND RULES

I hereby truthfully declare that this thesis is an original work prepared by me; that I have behaved in accordance with the scientific ethical principles and rules throughout the stages of preparation, data collection, analysis and presentation of my work; that I have cited the sources of all the data and information that could be obtained within the scope of this study, and included these sources in the references section; and that this study has been scanned for plagiarism with "scientific plagiarism detection program" used by Eskişehir Technical University, and that "it does not have any plagiarism" whatsoever. I also declare that, if a case contrary to my declaration is detected in my work at any time, I hereby express my consent to all the ethical and legal consequences that are involved.

Ayberk KARAASLAN

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A_c : Cross-sectional area

A_{sw} : Total cross-sectional area of the shear reinforcement

b_w : Width of the section

c : Damping coefficient

CP : Collapse prevention

d : Depth of the section

D : Displacement of the top floor

E_c : Modulus of elasticity of concrete

DOF : Degrees of freedom

F : Base shear force

f_{co} : Unconfined concrete strength under compression

f_{ct} : Tensile strength of concrete

f_{yw} : Yield strength of shear reinforcement

G : "Dead Load" load case

g_i : Dead load of the i^{th} floor

H : Height of the building

IO : Immediate occupancy

k : Stiffness

LS : Life safety

m : Mass

n : Live load participation factor

Nd : Design axial load

PGA : Peak ground acceleration

PGV : Peak ground velocity

Q : "Live Load" load case

q_i : Live load of the i^{th} floor

R : Structural behavior factor

s : Spacing between confinement bars

S(T) : Spectrum coefficient depending on the local site conditions and the building natural period

T : Natural period of the building

T_A, T_B : Spectrum characteristic periods

TEC : Turkish Earthquake Code

V_c : Contribution of concrete to shear strength of the section

V_{cr} : Diagonal cracking strength

V_r : Shear strength of the section

V_w : Contribution of shear reinforcement to shear strength of the section

W : Weight of the building

w_i : Storey weight of the i^{th} floor

$\ddot{u}(t)$: Acceleration – time series

$\dot{u}(t)$: Velocity – time series

u(t) : Displacement – time series

$F(t)$: Force – time series

ε_{cg} : Confined concrete strain

ε_{cu} : Unconfined concrete strain

ε_s : Deformation of reinforcement steel unit

ρ_s : Volumetric ratio of the lateral reinforcement

ρ_{sm} : Volumetric ratio of the transverse reinforcement that should be provided according to TEC 2007

1. INTRODUCTION

1.1. Overview and Motivation

Earthquakes are strong ground motions that generate a dynamic impact on the supports of buildings and this may lead buildings to be exposed to the forces much larger than they experience during their static state. The force triggered by the seismic activity of an earthquake strikes the weakest spot in the whole structure. Poor design and construction process may end up with many weak spots in the structure which cause a serious threat to life and property. One of the examples, which shook Gölcük Province on 17th of August 1999, is the Marmara Earthquake that caused 17.480 casualties with over 300.000 buildings either damaged or collapsed.

Staircases are one of the most crucial parts of a building due to its functional importance since they are used for connecting separate floors of a building (Figure 1.1). In fact, the presence of staircases provides a considerable amount of stiffness to the building. The effect of presence of staircases in the RC framed buildings which were mentioned in the literature can be summarized as imparting discontinuity in the structure, leading to failure of the adjacent structural members, causing torsional irregularity, altering the non-linear behaviour of the buildings, and impacting in several seismic parameters such as reducing the modal vibration periods, inter-storey drift ratio of the building. Thus, it can be underlined that the presence of the staircases during analysis phase should not be ignored.

Figure 1.1 *A typical staircase used in a RC building*

Turkey as a country that suffered from severe earthquakes in the past, is also home to considerable amount of poorly designed and constructed substandard reinforced concrete (RC) structures which cause serious risks to its residents and the ones around them. In order to avoid these risks, substandard structures should be subjected to a seismic performance assessment analysis which determines whether a structure is safe or not.

The key point for preparing an accurate numerical model is considering the staircase members as structural elements since staircases can alter the whole seismic behavior of structures due to their local stiffening effect. The inclusion of staircase elements to a numerical model is not only important for assessing the performance of an existing building accurately, but also important for designing a new building. So that member force demands could be more accurately calculated and design results could satisfy the actual demands. Unfortunately, the contribution of staircases to stiffness and strength of buildings is mostly neglected during the past and current engineering applications and may lead a local failure or the collapse of a building (Figure 1.2 and Figure 1.3)

Figure 1.2 *Damage on the column due to interaction with staircase (Li and Mosalam, 2013)*

1.2. Literature Review

Previous studies show that staircases take part in imparting additional stiffness and strength to the structure under seismic excitation, hence, the structural members which are adjacent to staircases are often exposed to high seismic demands. This increases the amount of shear force at short columns and may cause a premature brittle failure.

Li and Mosalam (2013) carried out a post-earthquake field reconnaissance to examine minor to extreme damage that large number of stairways experienced during Wenchuan Earthquake on 12th May 2008. The failure mechanisms of both primary structures and staircases were investigated. The authors emphasized that, unexpected seismic behaviour occurs when the interaction between staircase and primary structure was not taken into account during the design stage. Damages observed in main structures as formation of short columns and short beams which were caused by neglected interaction between primary load carrying members and staircases as shown in Figure 1.3.

(a) Damage to “short columns” under platform

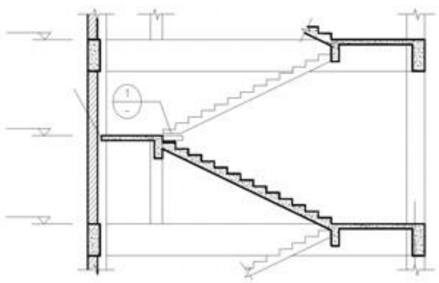
(b) Damage to horizontal slab and landing beam

(c) Shear damage to short column and short beam

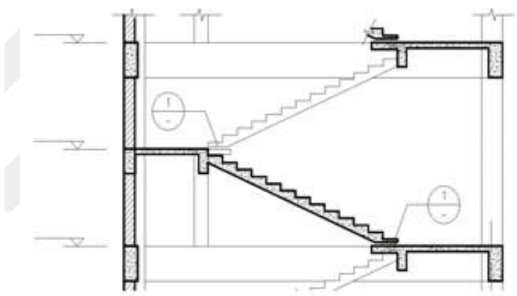
Figure 1.3 *Formation of short column: (a) damage to short columns under platform; (b) damage to horizontal slab and landing beam; (c) shear damage to short column and short beam (Li and Mosalam, 2013)*

Hongling *et al.* (2013) studied the stair influence on frame structures for their seismic performances, the structural dynamic property about the stair location, the whole structural response and the bearing capacity of the stair components. Their analyses showed that the supporting effect of staircase decreases natural period, increases base shear and alters angular displacement between the floors and internal forces in frame columns of the whole structure and makes great influence on the whole structural strength.

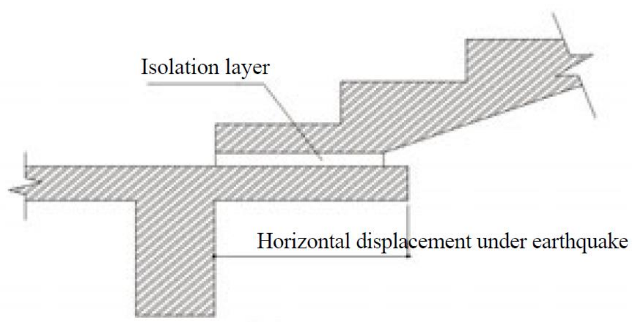
Feng *et al.* (2013) performed elasto-plastic time history analyses for 18 RC structure models with and without staircases to investigate the influence of the staircase on the stiffness, displacements and internal forces of the structures. Based on the features observed in the analyses, a new type of staircase design i.e., isolating them from the primary structure to eliminate the effect of K-type struts, is proposed in Figure 1.4 and discussed. It is concluded that the proposed method of staircase isolation is effective and feasible for engineering design, and does not significantly increase the construction cost. The authors also proposed that the effect of staircases can be neglected in the direction perpendicular to the ladder running since the increase of the lateral stiffness in this direction is not apparent. The most important recommendation of this study is; for frames away from the staircases, the internal forces of the frame members in the models with staircases are smaller than for the models without staircases. It is inaccurate to design these frames merely based on the internal forces resulting from the models with staircases, because a re-distribution of the internal forces will occur when the staircases are damaged.



(a) Section of full-isolated staircases



(b) Section of semi-isolated staircases



(c) Detail of isolation layer

Figure 1.4 *Proposed staircase isolation by Yuan et al. (2013)*

Singh and Choudhury (2012) studied the effects of staircase on the seismic performance of the RC frame buildings of different heights and different plans. They observed that, the presence of staircase tremendously influence the peak value of response quantities of beams and columns around staircase. The landing beams and columns adjacent to staircase have been found to fail due to excessive demand imposed owing to the presence of staircase. As they incorporated the stair model, it has been observed that, columns connected to the landing beams subjected to an increase in axial force by an average of 19%. The lateral moment in such columns increased on average by 32%. Shear force in landing beam increased by 36% on average. The torsional moment in landing beam increased enormously. The inter-storey drift ratio has been found to reduce by 33% in short direction and 23% in long direction on average. They mentioned that, non-incorporation of stair element in computer model may lead to failure of staircase under major earthquakes.

Onkar *et al.* (2015) modelled six storey RC buildings which differ in grades of concrete and designed it without considering staircase and they are subjected to non-linear pushover analysis and a superior ductile performance was achieved. After inclusion of staircases, it has been observed that the superior performance of the building models has been drastically reduced. Some building models with higher grade of concrete exhibited brittle failure due to collapse of columns which support staircases. Even for the building model with low strength concrete (20 MPa) up to 70% reduction in ductility capacity due to inclusion of staircase has been observed. From their study, they concluded that for the considered building, ignoring the contribution of staircase in structural modelling and design can lead to excessive damage and even collapse under a seismic event.

Cosenza *et al.* (2008) investigated the effect of staircases on gravity load designed buildings, which were subjected to non-linear pushover analysis and modelled with and without staircases. They observed that, inclusion of the staircases causes an increase of strength and a reduction in deformation capacity with respect to the building without staircase. On the contrary, the results have confirmed the need to utilize biaxial bending modelling and to account for the interaction of the different internal forces, such as bending moment-axial force interaction that characterizes the inclined elements, and the bending moment-shear interaction that governs the behaviour of short columns. In the studied case, as soon as more refined modelling is used, shear failure becomes predominant in the short columns and in the reinforced concrete slabs and precedes the conventional ductile failure due to pure flexure.

Xu and Li (2012), calculated elastic seismic response of the models by response spectrum method and bottom shear method in order to study the mechanical performance and the overall performance of the reinforced concrete frame structure with and without staircase under the earthquake action. The results they obtained showed that, the staircase increases the seismic lateral stiffness and internal force of frame beam column of the reinforced concrete frame structure, and may alter translational vibration mode into torsional vibration mode.

Jiang *et al.* (2012) studied the influence of staircases on the structural behavior of a typical RC frame structure by the comparison of internal force in the structural members considering and neglecting the effect of staircases under frequent earthquakes. Besides, the effect of staircases on the yielding and failure mechanism of the frame structure is investigated through static elasto-plastic analyses. They stated that, staircases act as the first line of seismic defense and are the first yielding structural members (for consuming energy). The severe damages of staircases lead to loss of its functionality as a safe evacuating passage during the emergency and the dissatisfaction of the anticipated seismic requirements. The seismic design of staircase should be improved to protect it from earthquake, or careful details should be provided to prevent the earthquake forces transmitted from the primary structure by isolating it.

Tegos *et al.* (2013) studied different types of staircases with respect to the earthquake design requirements and their complex interactions with the multistory space frame in which they are supported. In the first case the essential influence of the vertical component of the earthquake’s acceleration on the structure’s performance is examined. In the second case the staircases’ behavior is studied, as well as the important role that they play as structural seismic connections, in the response of space structures. In particular, external staircases, which connect structurally independent multistory systems are studied. In addition, in this work, a multistory external staircase that connects buildings in an area of high seismicity is analyzed. Staircases that present special design challenges due to gravity and earthquake loadings are also investigated. These structures include staircases with a free landing as well as helical cases. They concluded that, the inclusion of staircases in the analysis of the three-dimensional structure increases the stiffness of the structure in the transversal direction and reduces the relative displacement of the floors. Also, the presence of staircases influences the dynamic response of the structure. It has been stated that, the earthquake combination of loads is more crucial than the vertical load combinations in most types of staircases. Besides, the influence of the vertical component of the earthquake is crucial in staircases with a free landing as well as to the helical ones.

Zaid *et al.* (2013) investigated the effect of staircase on RC frame structures. Various building models (a bare frame, a frame having infill panels and a frame having infill except first storey) with and without staircase and number of storeys of the building have been considered with varying number of storeys, from 4 stories to 10 storeys. The Linear Response Spectrum analysis of the models has been carried out with the help of a FEM based software. Seismic characteristics in terms of time period, mass participation factor and storey drift have been compared with the seismic characteristics of models without staircase. Further, the effect of the staircase position in the building has also been investigated. In addition to these, short column effect, variation in moments of beams and columns that support the staircase slab, failure and deformation in staircase models and comparison of effects of infill panels have also been studied. Their study revealed that, presence of staircase and infill panels increased both the mass and stiffness of the building, storey drifts are considerably reduced when the effect of staircase is considered, staircase in building considerably affect the seismic behaviour of building by changing mode shape configuration. Additionally, dog-legged staircase on mid landing imparted additional shear demand in the supporting column leads to short column effect, resulting in a brittle type of failure.

Cao *et al.* (2014) conducted a study that includes two computer models of concrete frame with and without staircase. The seismic-performance of the models in elastic-phase was calculated by adopting base shear method, spectrum analysis by ETABS. The results show that including staircase into models will change the seismic performance of frame structure significantly. As a result the authors found that, stairs have a significant contribution to the structural lateral stiffness. The integral lateral stiffness increases more in the direction which is parallel to the flight running direction. Moreover, when staircase is considered in the overall structural calculation, the analysis result may be greatly influenced by the staircase model and calculation method. In the current case that there is no specific provision on how staircase participates in the overall structural analysis and calculation. It is suggested that a model with staircase should be adopted in the seismic design and spectrum analysis should be used in the earthquake action calculations. The paper proposes that the computer model with staircase and the response spectrum analysis should be used firstly in the seismic design of RC frames with staircase.

The buildings that are not able to satisfy the provisions of the recent design and earthquake codes are called “substandard buildings”. Some of the biggest handicaps of substandard buildings are having poor material quality, non-ductile reinforcement detailing and strong beam weak column phenomenon so that they do not perform with a desired behavior during earthquakes and cause a potential threat for its residents and the ones around them. Due to lack of audit and qualified workmanship, the mechanical properties of materials are usually lower than they are planned in structural analysis and design phase.

As can be seen from the literature review, there are not any present study based on the effect of staircases on substandard buildings. Owing to that fact that, carrying out a study on the effect of staircases on the seismic performance has been found beneficial for both enriching the literature and raising awareness among the citizens of countries which suffer from the existence of substandard buildings.

1.3. Objectives of the Present Study

It is known that, in most the current engineering applications, the effect of staircases on the seismic performance of RC buildings is usually neglected. Yet, it has been seen from the literature research that, staircases make considerable amount of impact to the seismic performance of buildings. Thus, a parametric study to investigate the seismic performance of both substandard and standard buildings were compared by various engineering demand parameters such as; inter-storey drift ratio (ISDR), base shear, shear force distribution of the columns supporting the staircase, modal properties (mode participation factor, mode shapes and natural vibration periods) and plastic hinge distribution.

The effect of staircases was examined only in the Y direction, which is parallel to inclined flight running direction in order to examine the truss action effect created by RC staircase due to its geometric shape. On the other hand, the effect of staircases have not been examined in the other direction since as Feng *et al.* (2013) proposed in their study that, the effect of staircases can be neglected in the direction perpendicular to the flight running since the increase of the lateral stiffness in this direction due to staircase is not apparent.

2. DESCRIPTION OF THE BUILDING MODELS

A parametric study is presented herein. The parameters used in this study are; conditions of the models such as standard and substandard which are functions of material type and cross-sectional properties, numbers of stories, number of spans and position of staircase.

2.1. Materials

As standard and substandard RC structures, there are two types of RC materials used for the building models.

2.1.1. Materials used in substandard models

In the substandard models, C10 was used for concrete and S220a was used for reinforcing steel. Mechanical properties of the given material are presented in Figure 2.1 and Figure 2.2. Modulus of elasticity of the concrete E_c is calculated according to Equation (2.1) which is proposed in Turkish Earthquake Code (TEC 2007) for calculating the approximate modulus of elasticity of concrete for performance asssesement analysis:

$$E_c \cong 5000\sqrt{f_{co}} \qquad [\text{MPA}] \qquad \textbf{(2.1)}$$

where, f_{co} denotes unconfined concrete strength under compression.

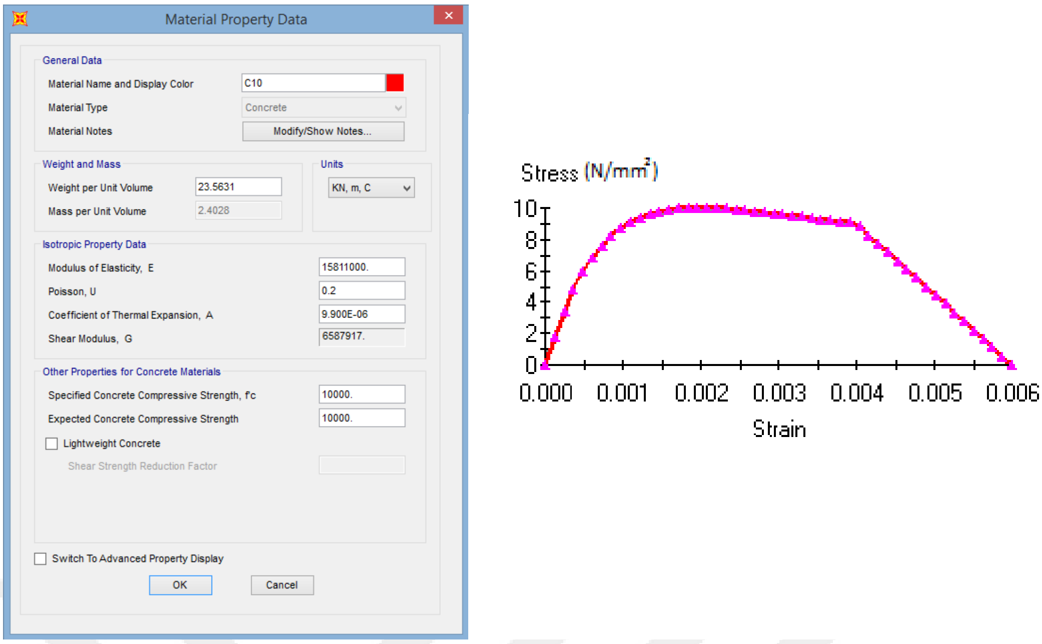


Figure 2.1 *Mechanical Properties of C10 concrete*

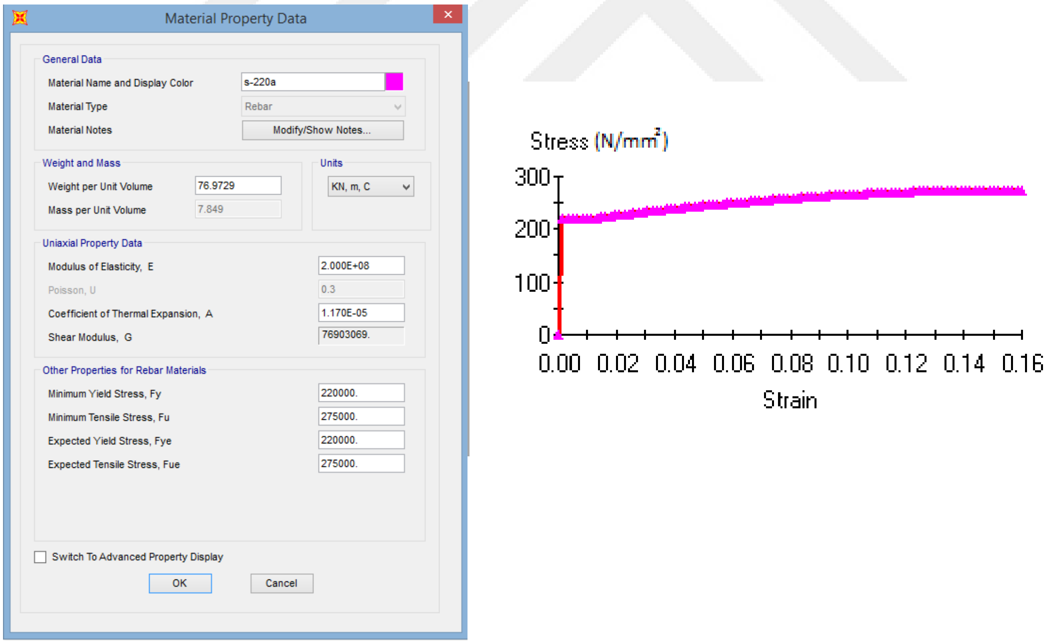


Figure 2.2 *Mechanical properties of S220a steel*

2.1.2. Materials used in standard models

In the standard models, C25 was used for concrete and S420a was used for reinforcing steel. Mechanical properties of the given material are presented in Figure 2.3 and Figure 2.4.

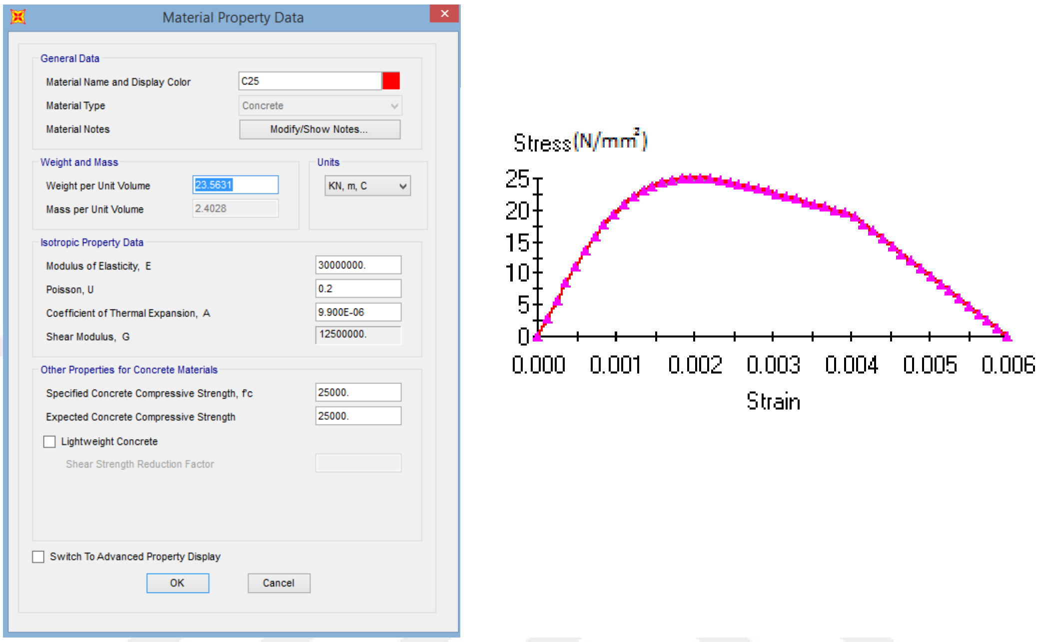


Figure 2.3 *Mechanical properties of C25 concrete*

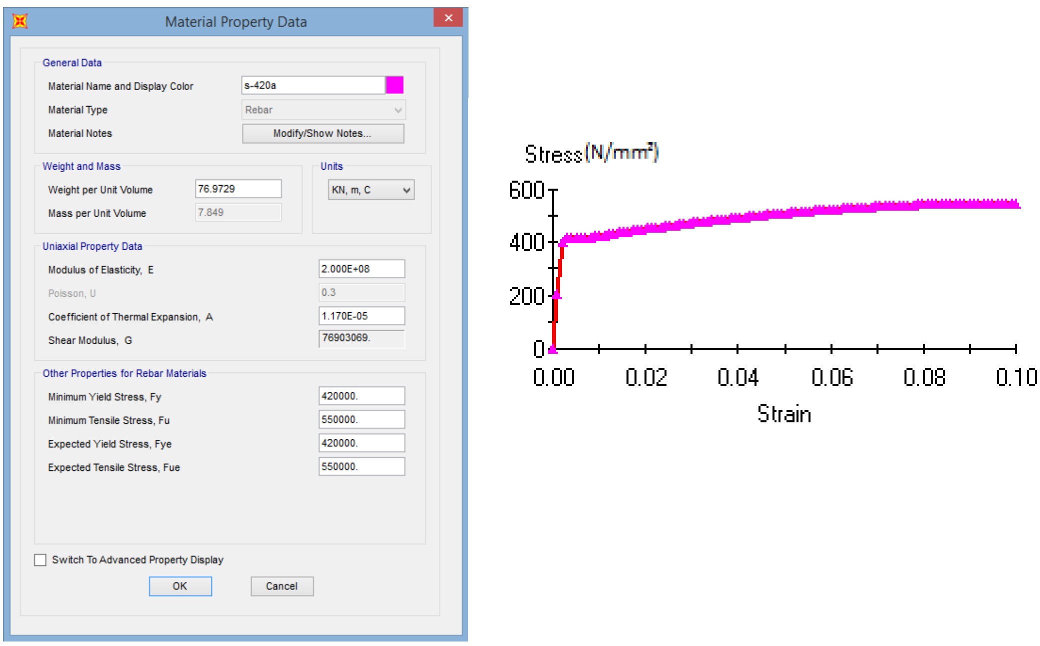


Figure 2.4 *Mechanical properties of S420a steel*

2.2. Geometrical Parameters

For a deeper insight into the effect of staircases on structural behavior, models with different geometrical parameters were used such as different number of spans and floor numbers. In this study, numbers of spans are 3 and 5 while the numbers of floors are 3, 5 and 8. Each span length is 5 meters and each storey height is 3 meters in all models. In addition, staircase existence and position were used as well as a geometrical parameter. The width of the inclined flights and landings are 1.2 meters long. The staircase details and storey layout plans of the models are given in Figures 2.5 – 2.11.

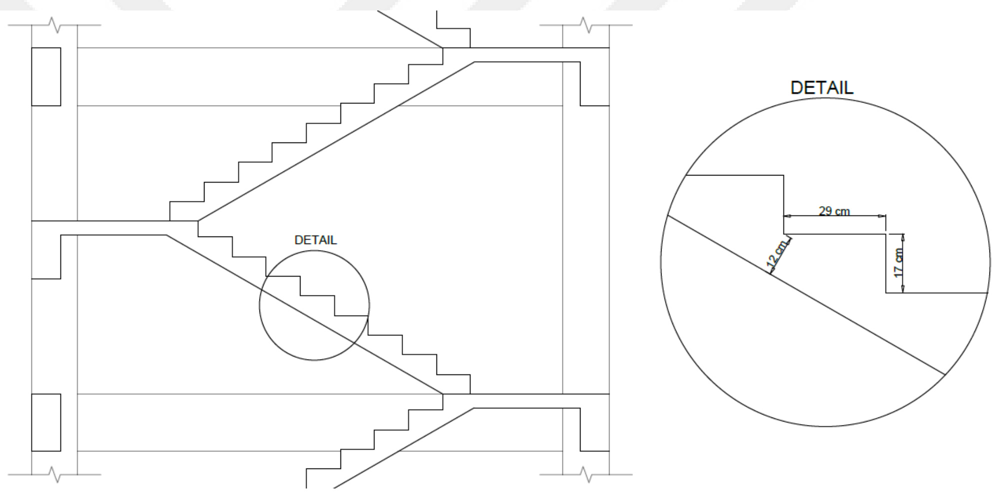


Figure 2.5 *Staircase details*

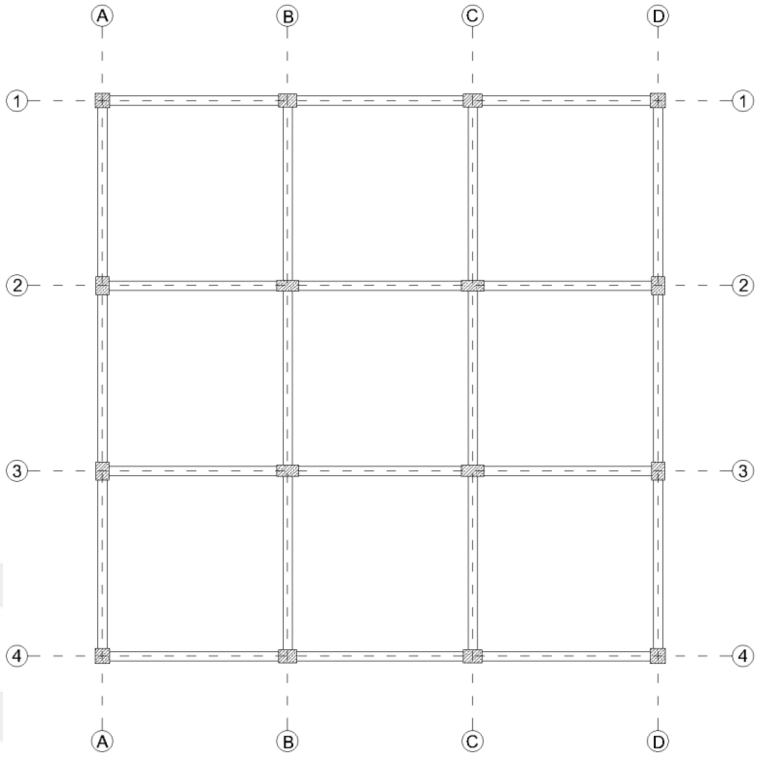


Figure 2.6 *Storey layout plan of three spanned non-staircased models*

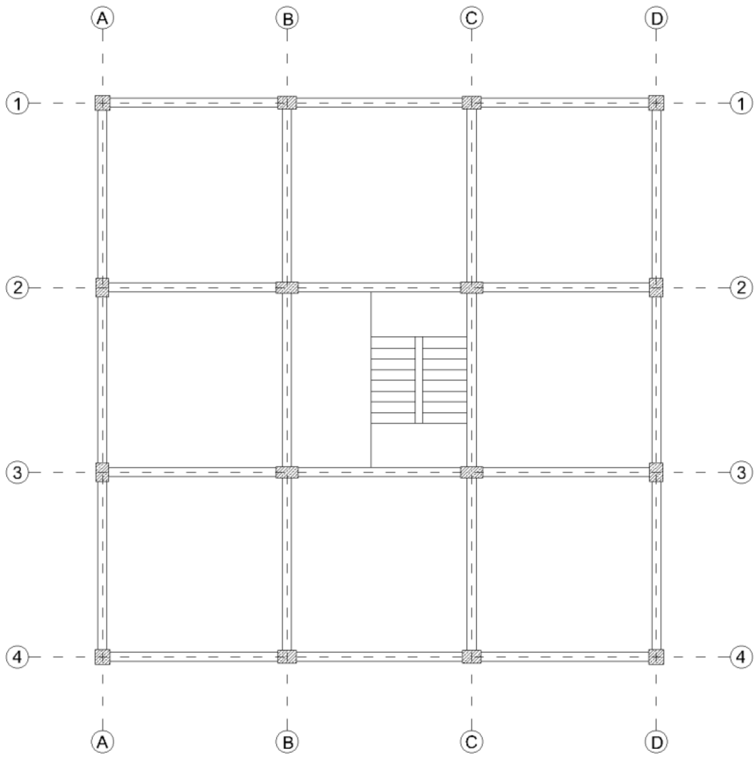


Figure 2.7 *Storey layout plan of three spanned centric staircased models*

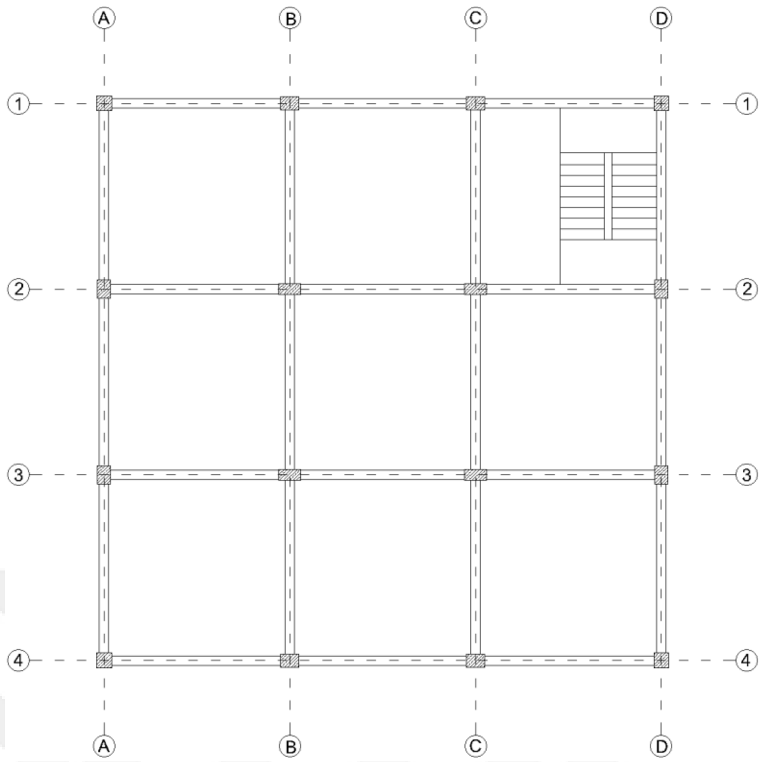


Figure 2.8 *Storey layout plan of three spanned corner staircased models*

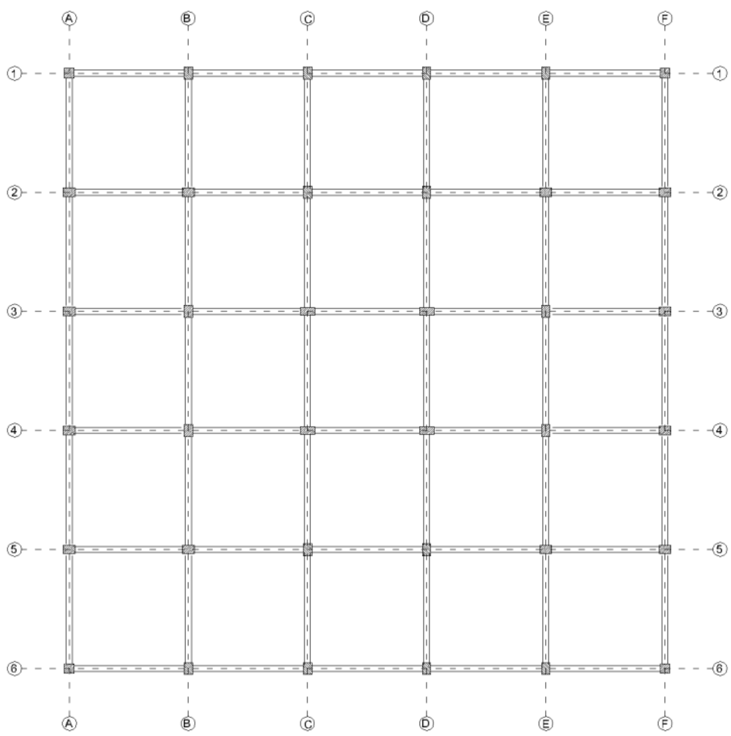


Figure 2.9 *Storey layout plan of five spanned non-staircased models*

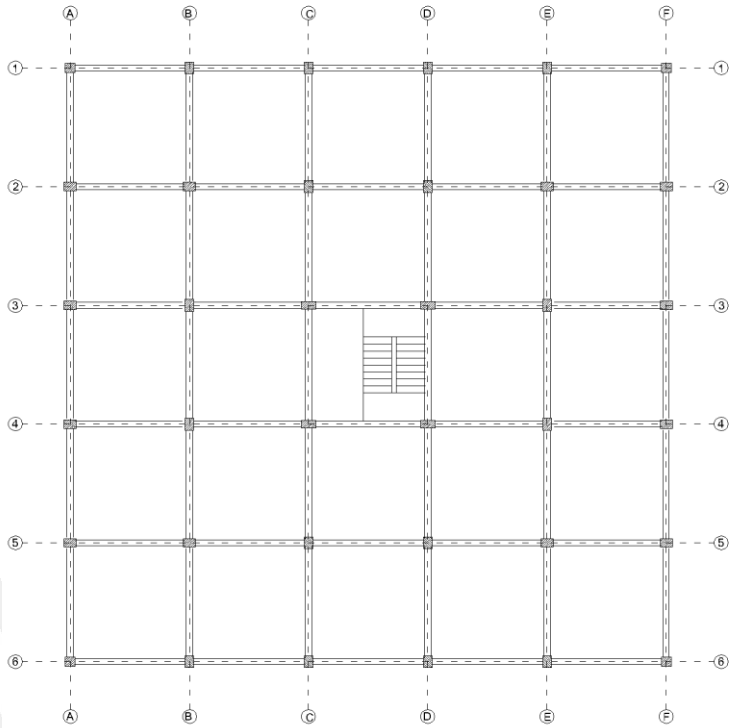


Figure 2.10 *Storey layout plan of five spanned centric staircased models*

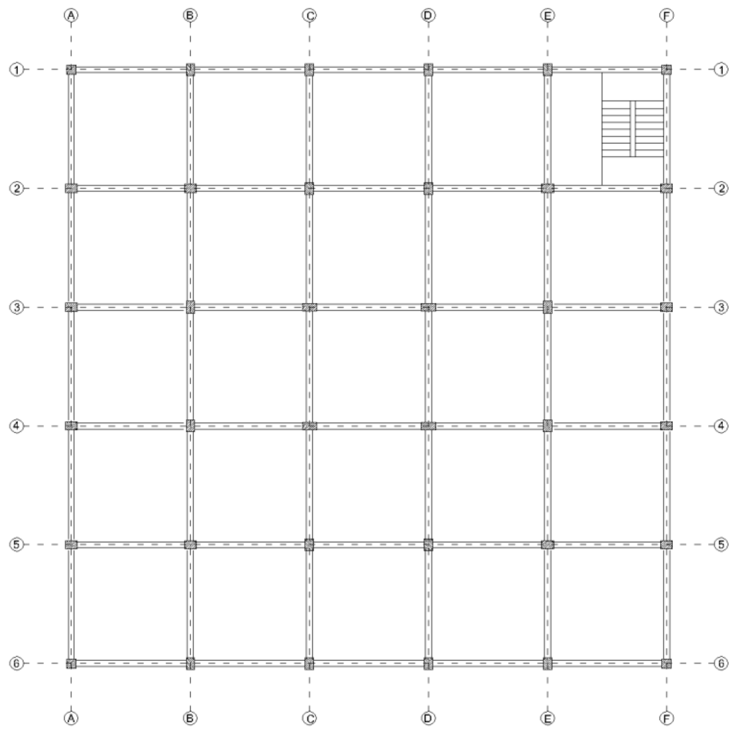


Figure 2.11 *Storey layout plan of five spanned corner staircased models*

Note that the size of the columns in the given figures are not scaled and do not represent the actual size. Column sizes depend on the numbers of floors, yet the directions of them remain the same.

2.3. Load Assignments

Self-weight of the frame members were auto-calculated by the structural analysis software. Besides, the additional loads are given below;

Wall Load : 7.5 kN/m

Slab Self Load : 5 kN/m^2

Live Load : 2kN/m^2

2.4. Cross-Sectional Properties of Frame Members

2.4.1. Substandard models

Compared to standard ones, strong beams and relatively small-sized columns were used on substandard structures. This situation causes "*strong beam weak column phenomenon*" and may end up with the formation of the plastic hinges at the ends of the columns which is an undesirable case since columns are the main load carrying structural members, their failure can lead to failure of the whole building.

In this study, it was meant to stimulate strong beam weak column phenomenon so that more realistic substandard building action could be obtained. To do so, relatively larger and more reinforced beams with respect to standard beams, were used.

During the design process of the substandard models, a software for structural analysis and design called "STA4CAD" was used. STA4CAD is able to design buildings according to "1975 Turkish Seismic Code", which is a code that most of the existing old buildings were designed accordingly. Even though some of these old buildings received engineering service, they are classified as "substandard buildings" due to poor material quality and poor worksmanship used during their construction phase.

Before modeling, the cross-sectional areas of the columns and beams were pre-determined. Until having all the sections sufficient, bigger dimension values were given and re-analysed. The analysis parameters employed in STA4CAD are shown in Figure

2.12 and 2.13 for an example analysis model. 3D view of a sample building analytical model is shown in Figure 2.14.

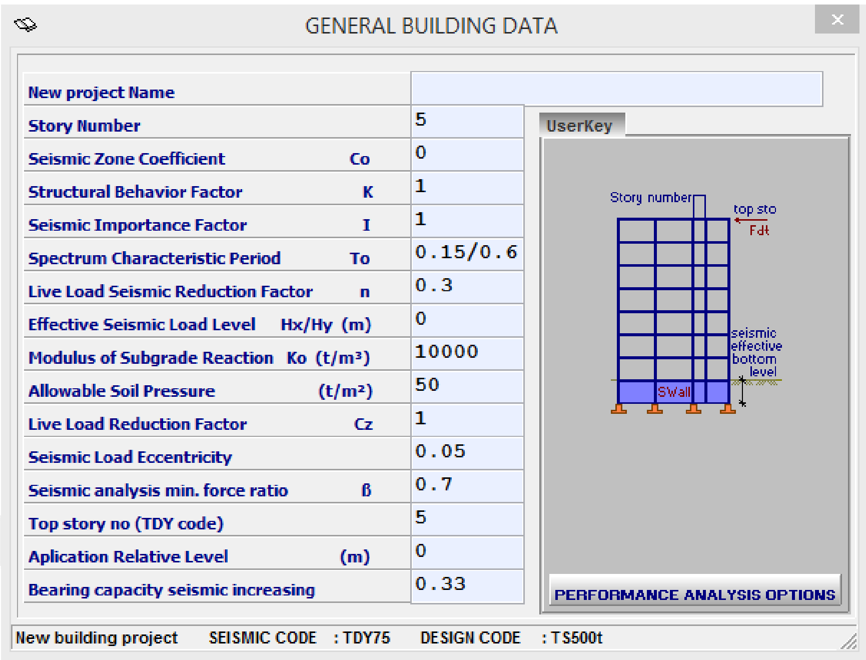


Figure 2.12 *General building data used in STA4CAD analysis*

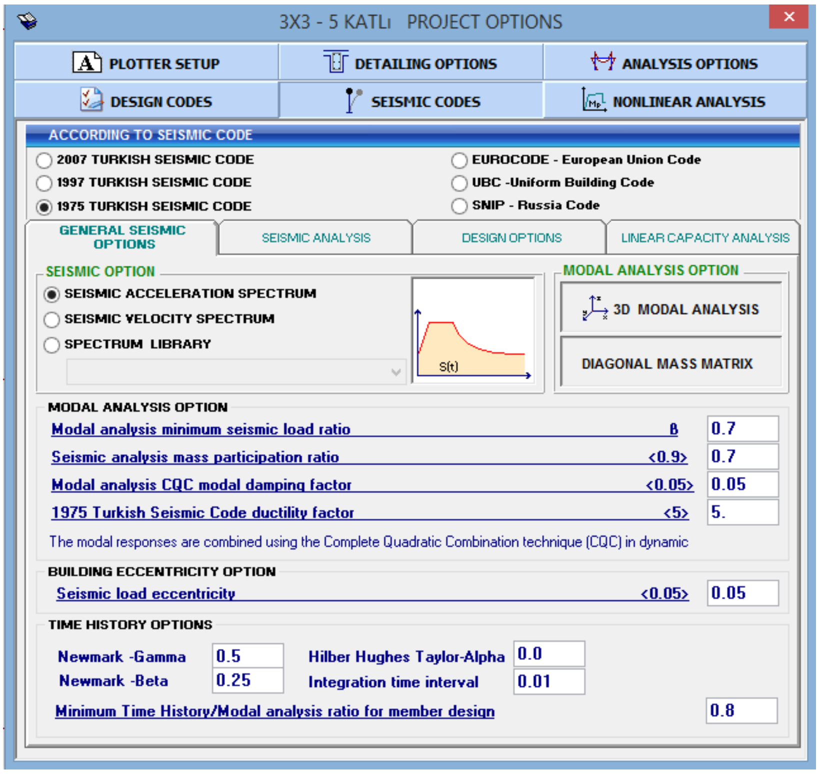


Figure 2.13 *Project options menu used in STA4CAD analysis*

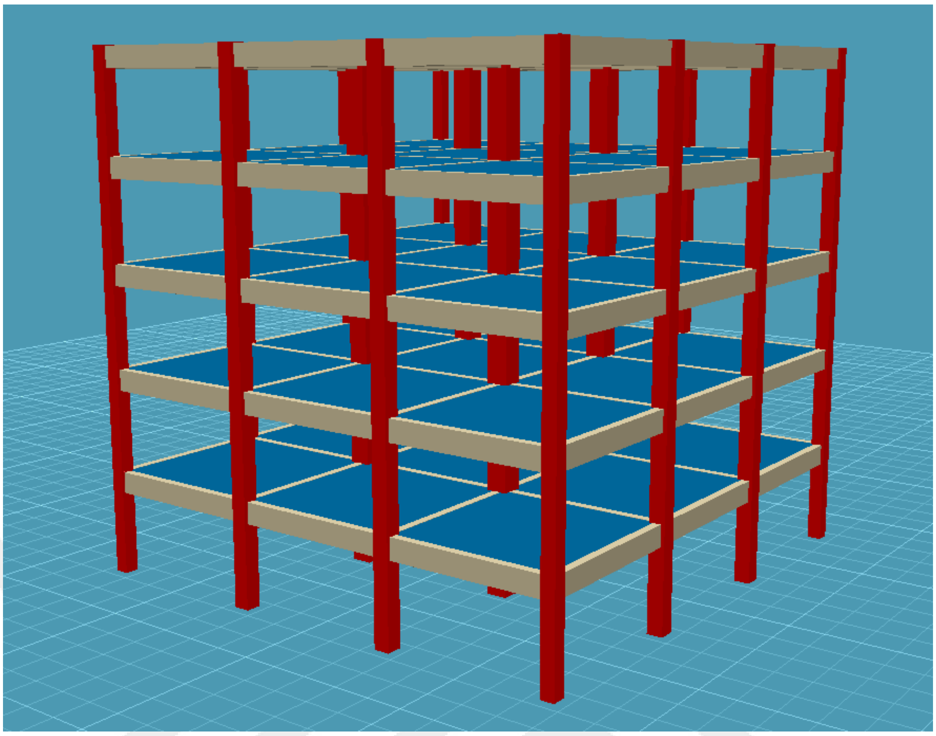


Figure 2.14 *3D model of 5 storey 3 spanned substandard building*

2.4.2. Standard models

Since the standard models were designed according to TEC 2007, unlike substandard structures, it is not expected to observe strong beam weak column effect so that the plastic hinges would occur at the ends of the beams firstly, which is a desired behavior for earthquake resistant buildings.

In order to design standard models, they were modelled with IDECAD, which is a sophisticated structural analysis and design software.

In Table 2.1, “Structural Behavior Factors (R)” presented in TEC 2007 are listed. For modeling of the standard structures, the R factor is selected as 8, since seismic loads are fully resisted by frames, and systems have high ductility level. First seismic zone is assumed to be the earthquake zone used in this study. The analysis parameters are shown in Figure 2.15.

Table 2.1 *Structural Behavior Factors (R)(TEC 2007)*

BUILDING STRUCTURAL SYSTEM	*Systems of Nominal Ductility Level*	*Systems of High Ductility Level*
(1) CAST-IN-SITU REINFORCED CONCRETE BUILDINGS		
(1.1) Buildings in which seismic loads are fully resisted by frames..	4	8
(1.2) Buildings in which seismic loads are fully resisted by coupled structural walls...	4	7
(1.3) Buildings in which seismic loads are fully resisted by solid structural walls..	4	6
(1.4) Buildings in which seismic loads are jointly resisted by frames and solid and/or coupled structural walls...........	4	7
(2) PREFABRICATED REINFORCED CONCRETE BUILDINGS		
(2.1) Buildings in which seismic loads are fully resisted by frames with connections capable of cyclic moment transfer	3	7
(2.2) Buildings in which seismic loads are fully resisted by single-storey frames with columns hinged at top...............	—	3
(2.3) Prefabricated buildings in which seismic loads are fully resisted by prefabricated or cast-in-situ solid and/or coupled structural walls with hinged frame connections	—	5
(2.4) Buildings in which seismic loads are jointly resisted by frames with connections capable of cyclic moment transfer and cast-in-situ solid and/or coupled structural walls	3	6
(3) STRUCTURAL STEEL BUILDINGS		
(3.1) Buildings in which seismic loads are fully resisted by frames..	5	8
(3.2) Buildings in which seismic loads are fully resisted by single-storey frames with columns hinged at top..............	—	4
(3.3) Buildings in which seismic loads are fully resisted by braced frames or cast-in-situ reinforced concrete structural Walls		
(a) *Concentrically braced frames*......................................	4	5
(b) *Eccentrically braced frames*..	—	7
(c) *Reinforced concrete structural walls*............................	4	6
(3.4) Buildings in which seismic loads are jointly resisted by frames and braced frames or cast-in-situ reinforced concrete structural walls		
(a) *Concentrically braced frames*......................................	5	6
(b) *Eccentrically braced frames*..	—	8
(c) *Reinforced concrete structural walls*............................	4	7

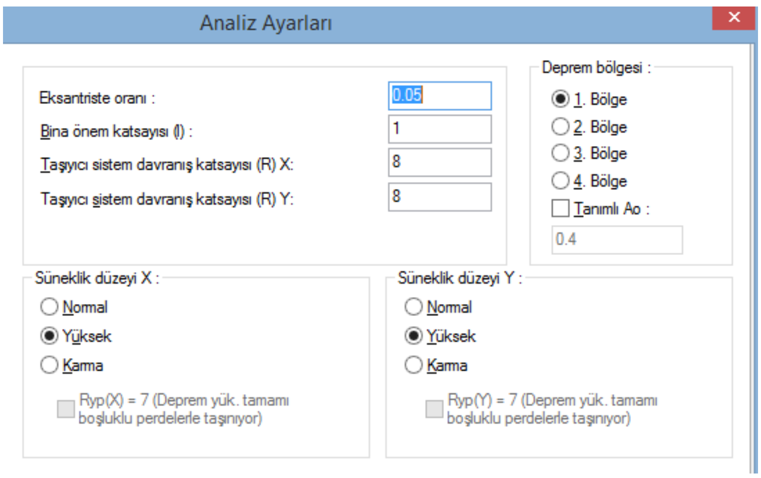


Figure 2.15 *Analysis Options*

Z3 is assumed to be the site class for the analysis of the models and was chosen in the design spectrum function menu of IDECAD as shown in Figure 2.16.

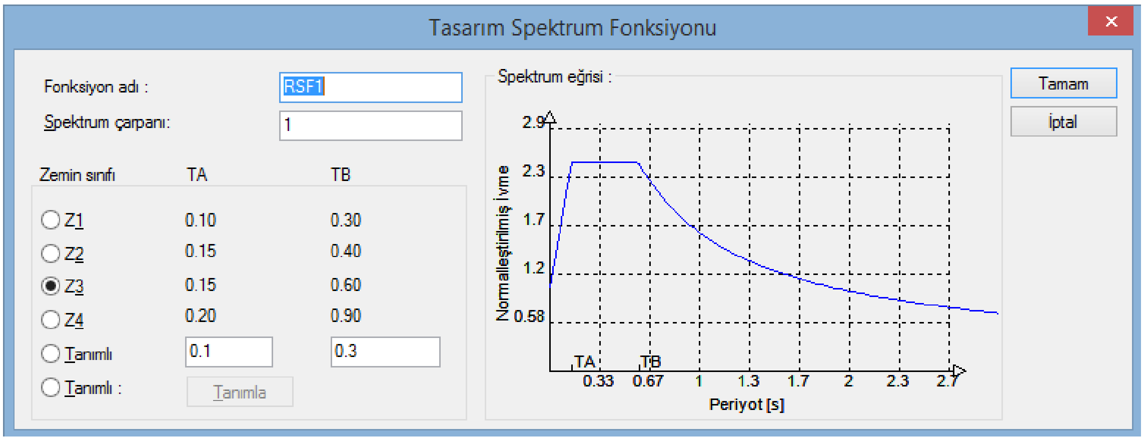


Figure 2.16 *Selected response spectrum for analysis*

One of the example 3D models prepared in IDECAD is shown in Figure 2.17.

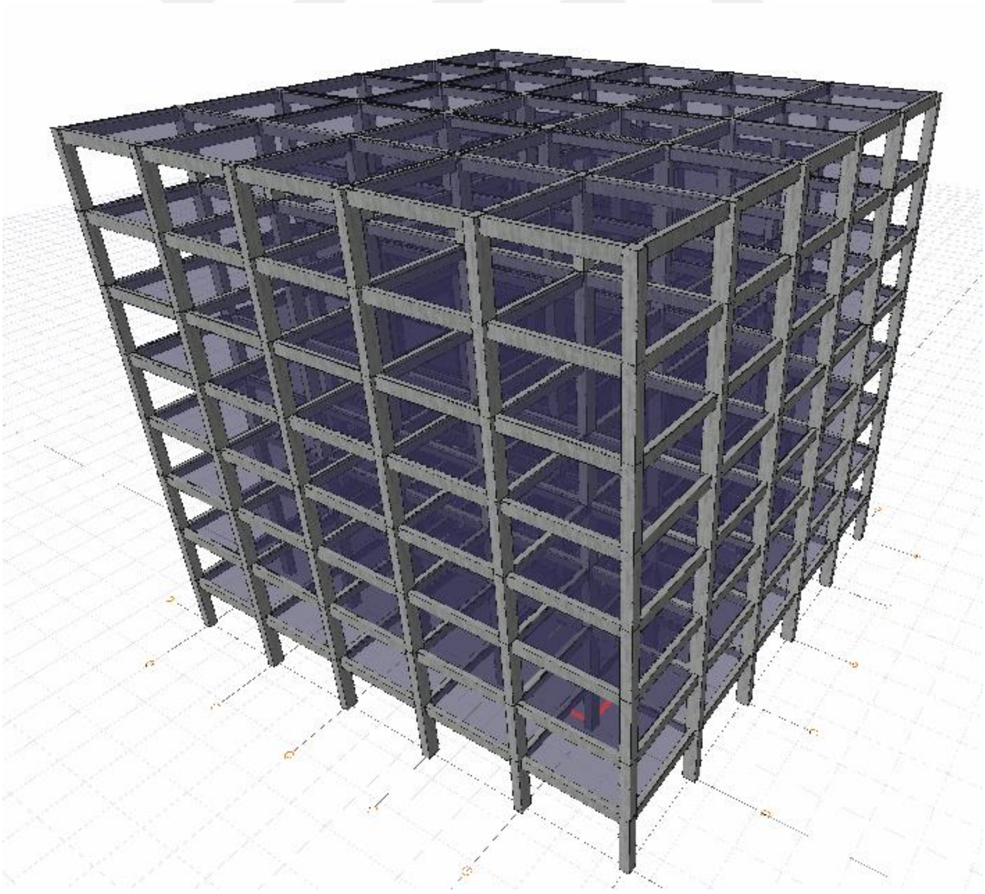


Figure 2.17 *3D model of 8 storey 5 spanned standard building*

2.5. Cross-Sectional Dimensions and Reinforcements Determined By STA4CAD and IDECAD

Note that all analyses were conducted without taking staircases into account. The results obtained from the analyses are presented in Tables 2.2, 2.3 and 2.4 in terms of cross-sectional dimensions and numbers of reinforcement bars. In these tables, all reinforcing bar sizes are equal and are Φ14. Additionally, 3 direction represents vertical axis, 2 direction represents horizontal axis (t3 and t2 are the cross-sectional dimensions corresponding in 3 and 2 axes.) In Table 2.2, building nomenclature is formatted in the shape of; Storey Number x Span Number x Span Number_st (where “st” stands for “standard”) or Storey Number x Span Number x Span Number_sub (where “sub” stands for “substandard”) (e.g. 8x5x5_sub stands for 8 storey 5 spanned substandard buildings while 5x3x3_st stands for 5 storey 3 spanned standard buildings). The reinforcements of the staircases shown in Table 2.4 were found from the non-integrated staircase analyser of IDECAD.

Cross-sectional dimensions determined to be kept constant in the standard and substandard models with the same number of stories and same number of spans in order not to consider the cross-sectional dimensions as a parameter.

Table 2.2 *Cross-sectional dimensions and longitudinal reinforcements of the columns according to the results obtained from STA4CAD and IDECAD*

Building Nomenclature	Cross-Sectional Dimensions (cm) (t3/t2)	Number of Longit. Bars Along 3-Dir Face	Number of Longit. Bars Along 2-Dir Face	Total Number of Longit. Bars
3x3x3_sub	35/40	3	4	10
	45/35	4	3	10
	50/35	4	4	12
3x3x3_st	35/40	3	4	10
	45/35	4	4	12
	50/35	4	4	12
3x5x5_sub	35/40	3	4	10
	45/35	4	3	10
	50/35	4	4	12
3x5x5_st	35/40	3	4	10
	45/35	4	4	12
	50/35	4	4	12
5x3x3_sub	40/40	4	4	12
	35/50	4	4	12
	60/35	4	4	12
5x3x3_st	40/40	8	8	28
	35/50	7	5	20
	60/35	4	5	14
5x5x5_sub	40/40	4	4	12
	35/50	4	4	12
	60/35	4	5	14
5x5x5_st	40/40	6	6	20
	35/50	6	5	18
	60/35	5	7	20
8x3x3_sub	45/45	4	4	12
	40/60	6	4	16
	75/40	6	6	20
8x3x3_st	45/45	5	5	16
	40/60	6	4	16
	75/40	6	9	26
8x5x5_sub	45/45	4	4	12
	40/60	6	4	16
	75/40	6	6	20
8x5x5_st	45/45	5	5	16
	40/60	6	4	16
	75/40	6	9	26

Table 2.3 *Cross-sectional dimensions and longitudinal reinforcement areas at the ends of the beams*

Frame Type	Cross-Sectional Dimensions (cm)(t3/t2)	Top Reinforcing Bar Area (cm2)	Bottom Reinforcing Bar Area (cm2)
Substandard Beam	60/25	7.70	3.36
Standard Beam	50/25	3.40	2.26

Table 2.4 *Cross-sectional dimensions and longitudinal reinforcements of the staircase slab*

Frame Type	Cross-Sectional Dimensions (cm) (t3/t2)	Number of Longit. Bars Along 3-Dir Face	Number of Longit. Bars Along 2-Dir Face	Total Number of Longit. Bars
Substandard Staircase Frame	12/120	7	2	14
Standard Staircase Frame	12/120	7	2	14

3. ANALYTICAL MODELING OF THE BENCHMARK BUILDINGS

After the geometry, cross-sections and steel reinforcements were determined in the previous section, the analytical models were generated by using SAP2000 in this section. Besides, in order to define the non-linear behavior of RC cross-sections, XTRACT was used for cross-sectional analysis.

3.1. Defining Material Properties

Properties of the materials were given in Figure 2.1, 2.2, 2.3 and 2.4. As mentioned in Section-2, for substandard models C10/S220a is used while for standard models C25/S420a is used.

3.2. Defining Cross-Sections

According to the data obtained from Table 2.2, 2.3 and 2.4, cross-sections of the frame members are defined in SAP2000 and XTRACT. All longitudinal bars are Φ14 in all models. Note that longitudinal spacing between confinement bars is 10 centimeters in standard sections while it is 20 centimeters in substandard sections. Number of confinement bars in both directions is 2 in substandard sections while it varies depending on the analysis results obtained from IDECAD. Size of confinement bars is Φ8 in all models. Cross-sections of the beams supporting the landings of the staircase were modelled as column with P-M2-M3 plastic hinges at both ends since they are not part of the rigid diaphragm and will be affected from axial force and bi-axial moments. The cross-sectional properties were defined from the menu shown in Figure 3.1 and the reinforcement data of the section were defined from the menu shown in Figure 3.2 for columns. For beams the properties were shown in Figures 3.3 and 3.4, for staircase sections were shown in Figures 3.5 and 3.6. In Figure 3.7, the reinforcement data of a substandard column section is shown.

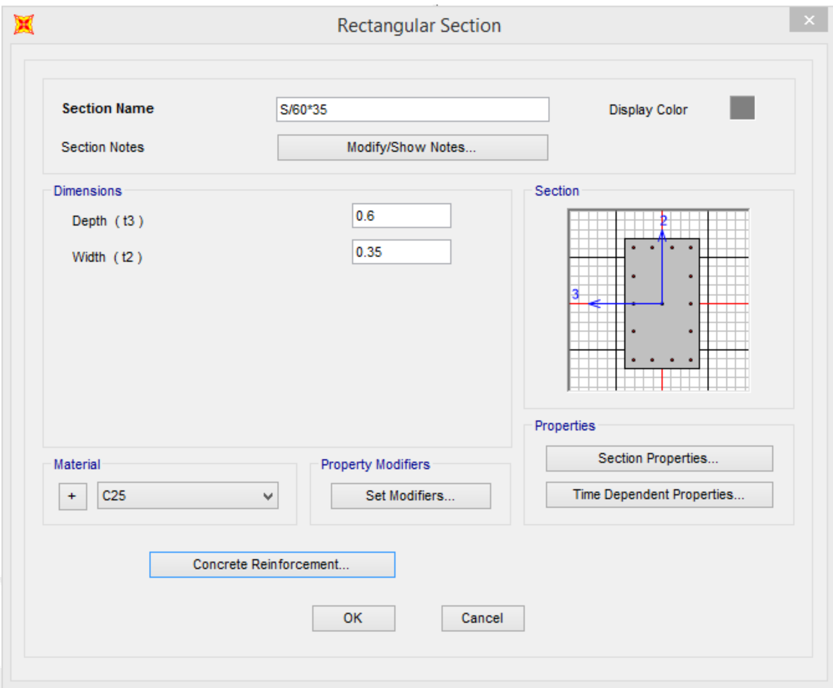


Figure 3.1 *Definition of dimensional parameters of 60/35 standard column*

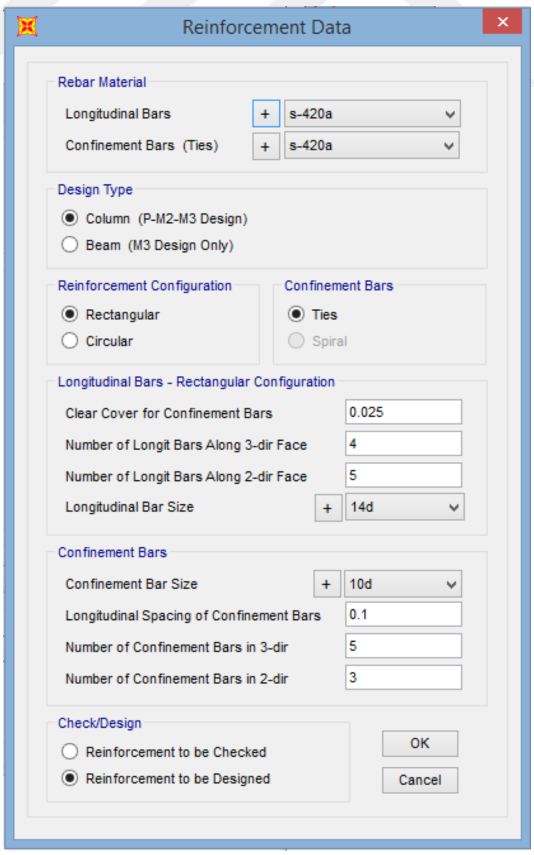


Figure 3.2 *Definition of reinforcement parameters of 60/35 standard column*

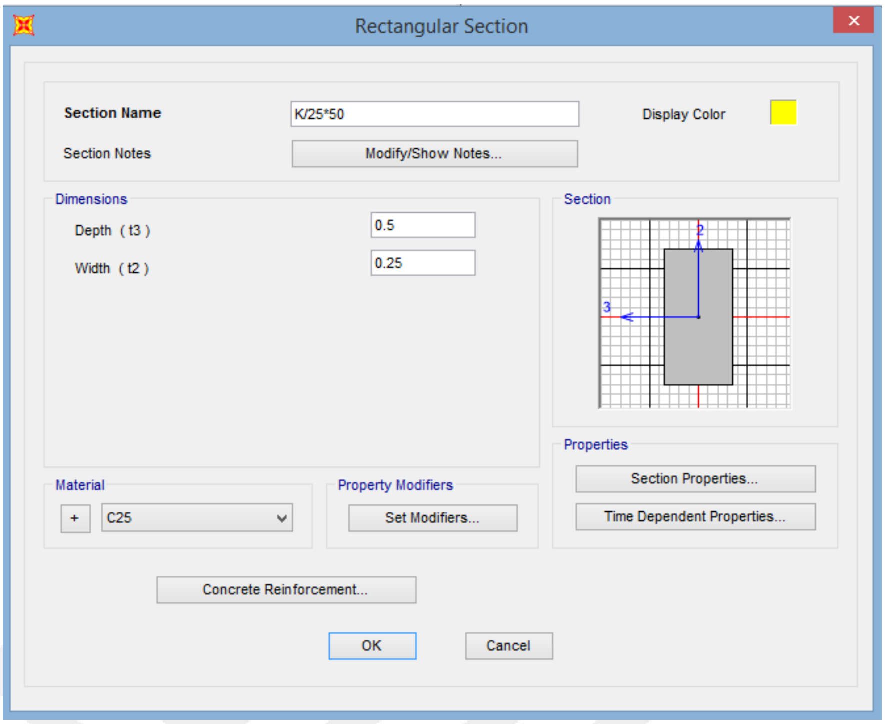


Figure 3.3 *Definition of dimensional parameters of 50/25 standard beam*

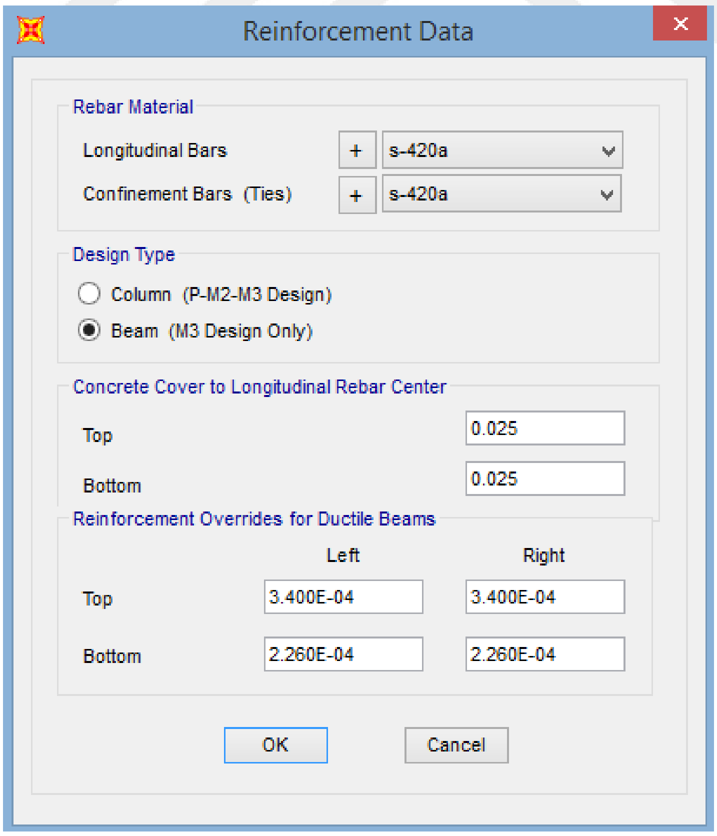


Figure 3.4 *Definition of reinforcement parameters of 50/25 standard beam*

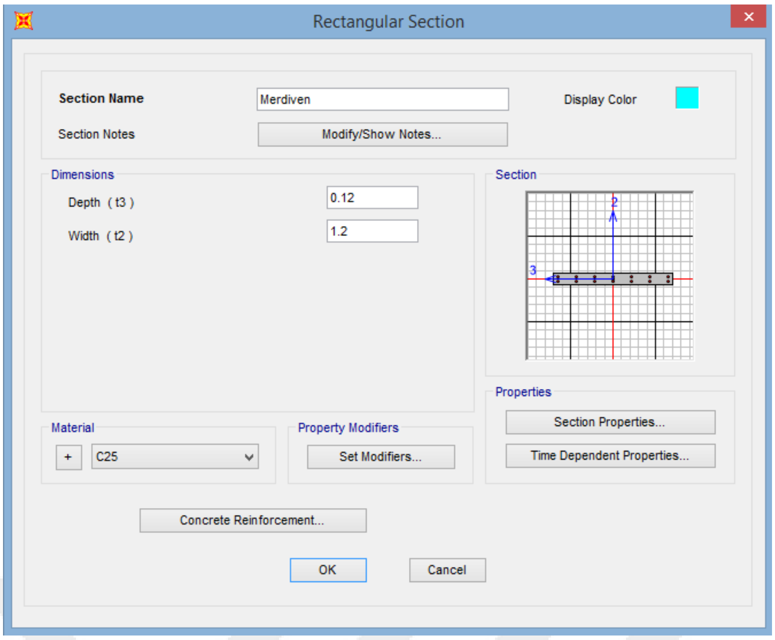


Figure 3.5 *Definition of dimensional parameters of 12/120 standard staircase frame*

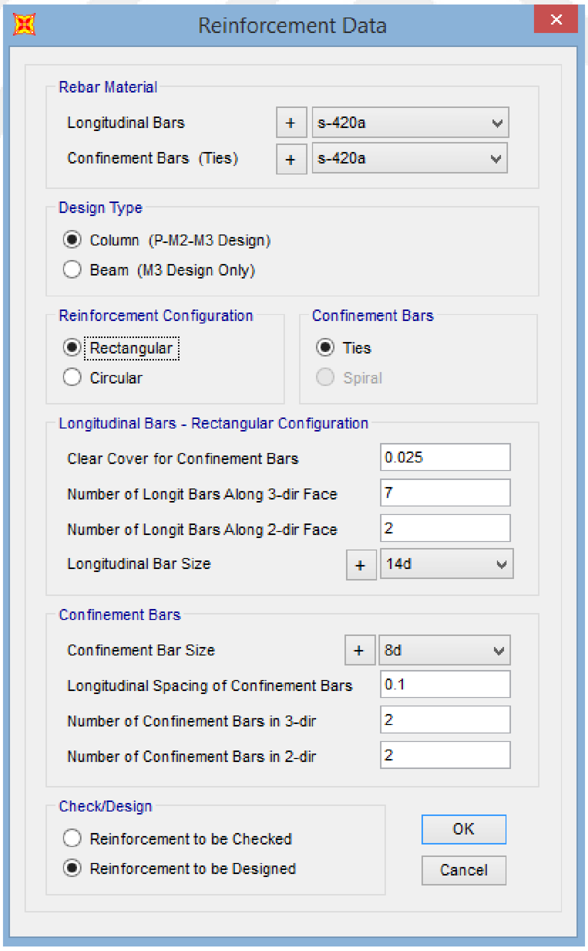


Figure 3.6 *Definition of reinforcement parameters of 12/120 standard staircase frame*

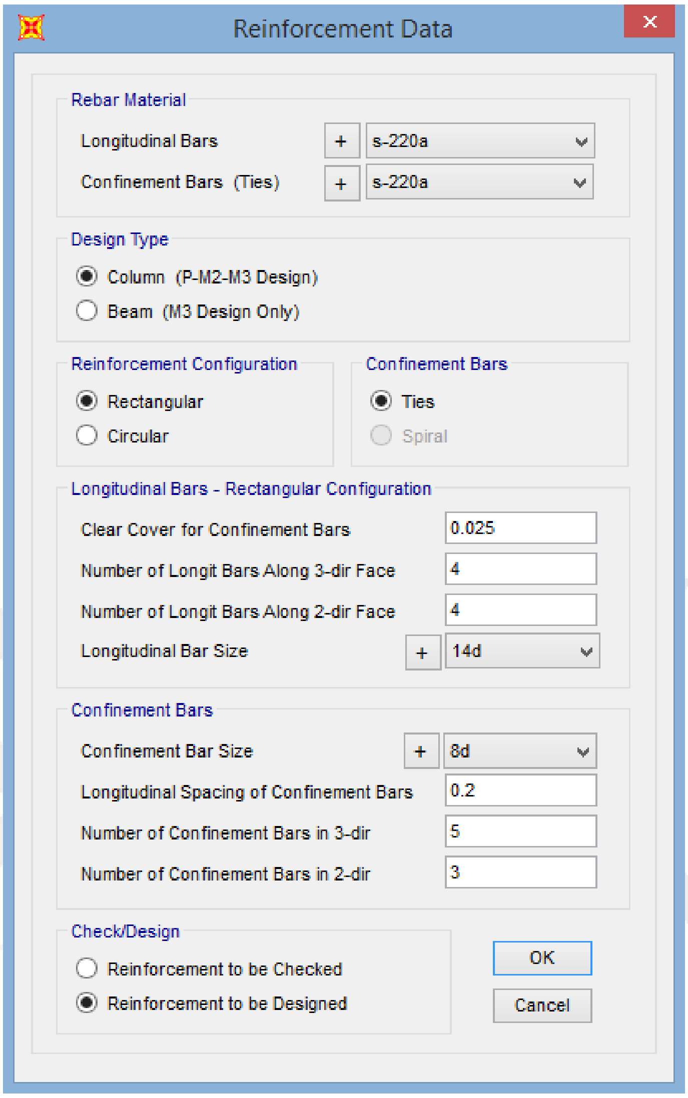


Figure 3.7 *Definition of reinforcement parameters of 60/35 substandard column*

3.3. Defining 3d Analysis Models

Firstly, grid lines were created to provide reference points for placing frame members. Grids may vary depending on the number of spans and number of floors of the related model.

Secondly, frame members were drawn. Note that all ends of the frame members are continuous and bottom ends of the ground floor columns are assumed to be fully restraint.

Thirdly, the contribution of stiffness by infill walls to the overall stiffness of the models and member rigid-end-zones are neglected.

It is important to underline that, staircase elements were modelled as frame members. Location or the existence of the staircase may vary. Standard models labelled as "_st", substandard models labelled as "_sub", non-staircased models are labelled as "_non" (e.g. 5x3x3_sub_non), centric staircased models are labelled as "_cent" (e.g. 8x5x5_st_cent) and corner staircased models are labelled as "cor" (e.g. 3x3x3_sub_cor). In Figure 3.8, 3.9 and 3.10, 3D analysis models of 3x3x3_st_cent, 5x5x5_sub_cor and 8x3x3_sub_non are shown as examples, respectively.

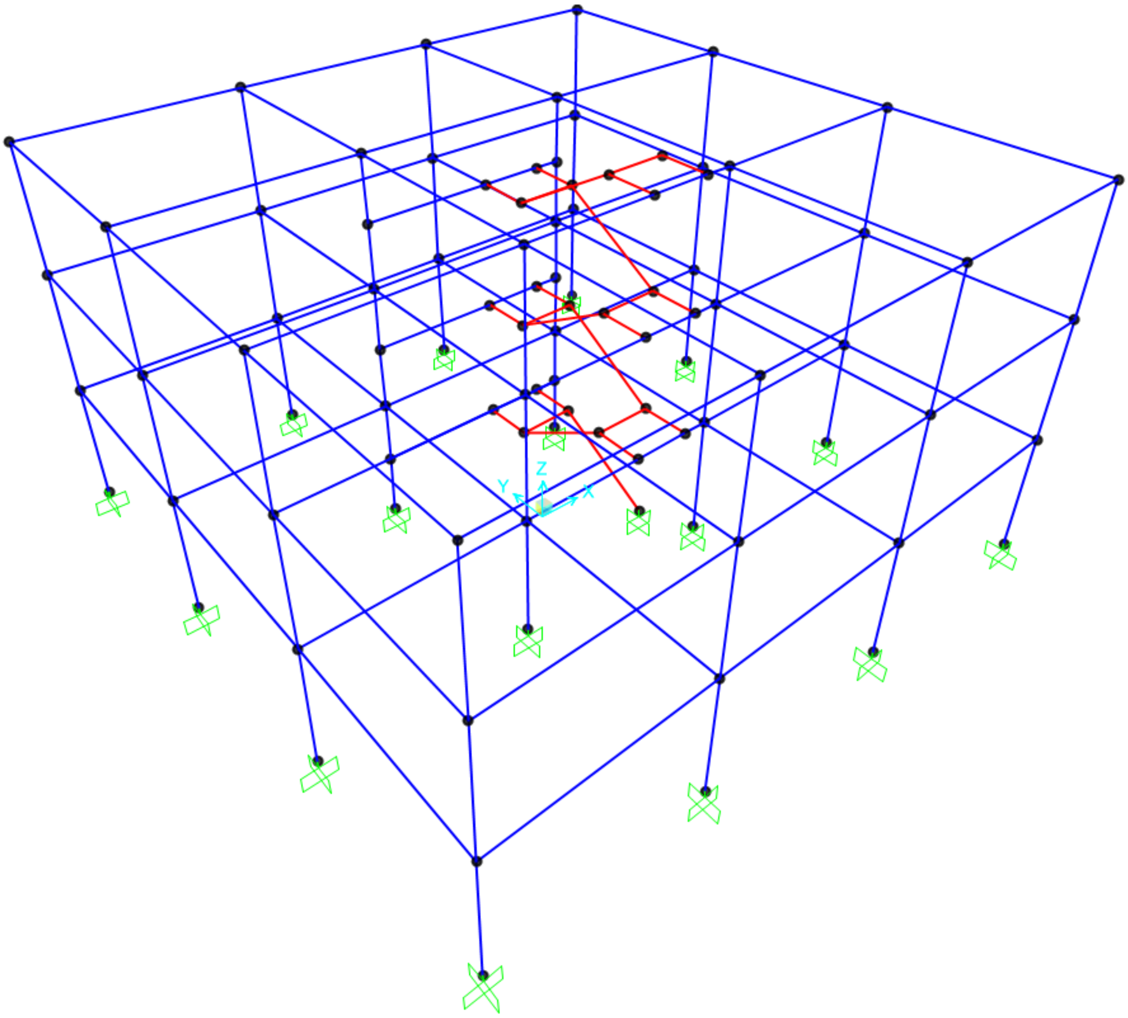


Figure 3.8 *3D analysis model of 3x3x3_st_cent*

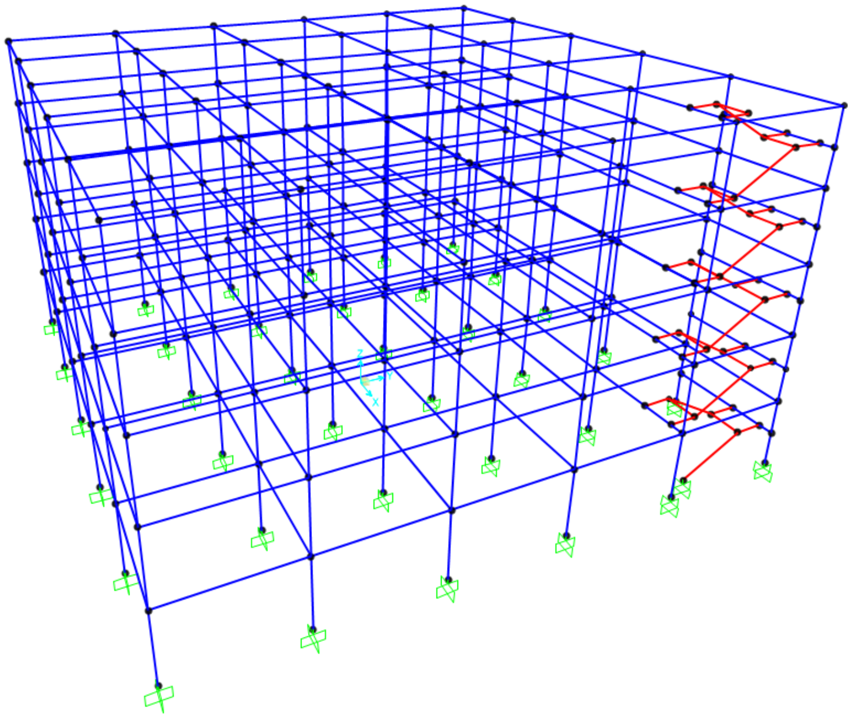


Figure 3.9 *3D analysis model of 5x5x5_sub_cor*

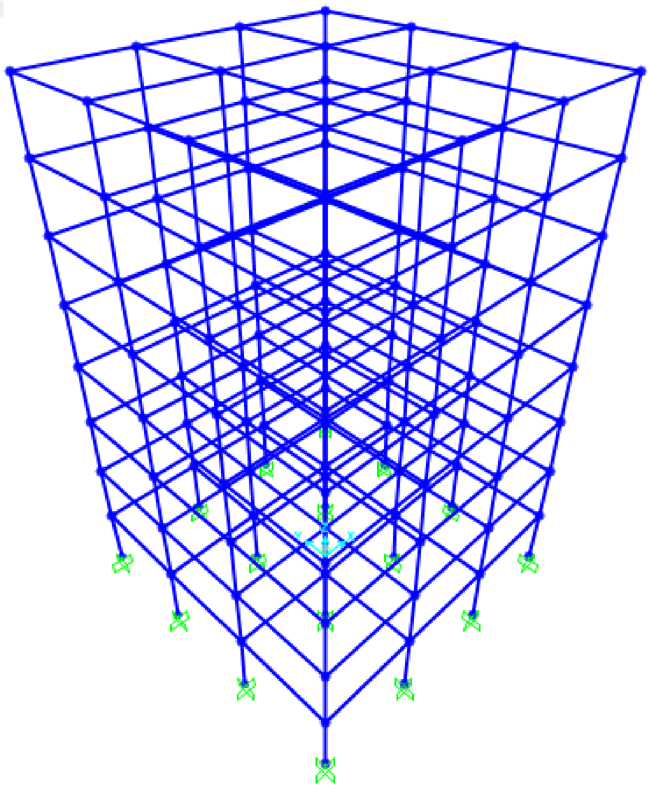


Figure 3.10 *3D analysis model of 8x3x3_sub_non*

3.4. Assigning Rigid Diaphragm

Slabs are relatively rigid members along their inplane direction compared to beams due to their larger cross-sectional area and stability. Therefore, slabs attract the vast majority of the axial force that the adjacent beams supposed to carry and prevent them to bend out of plane so that the adjacent beams would only be affected by M3 moments. One of the accurate ways of simulating this behavior in our analysis model is to assign rigid diaphragm as a constraint to the points that are adjacent to slabs.

Note that the landing slabs of the staircases at the mid-floors have their own rigid diaphragms.

In Figure 3.11, an assigned rigid diaphragm to the top floor can be seen, and in Figure 3.12 an assigned rigid diaphragm to the mid-floor slab can be seen with the green dots.

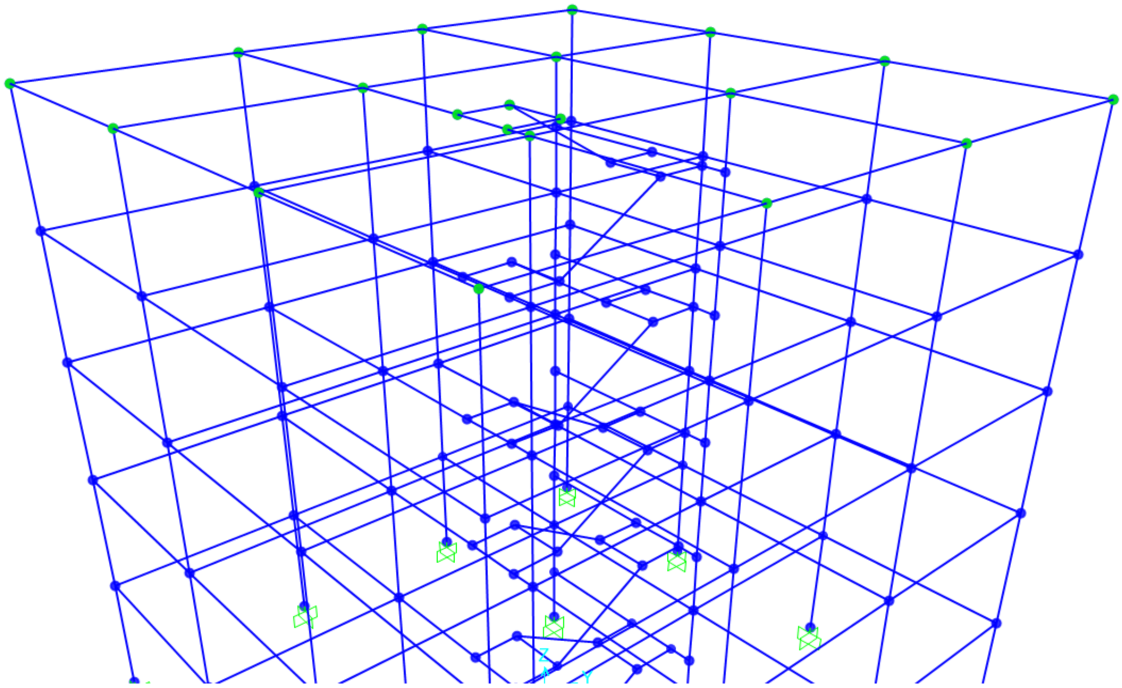


Figure 3.11 *Rigid diaphragm of the 5th floor of 5x3x3_sub_cent*

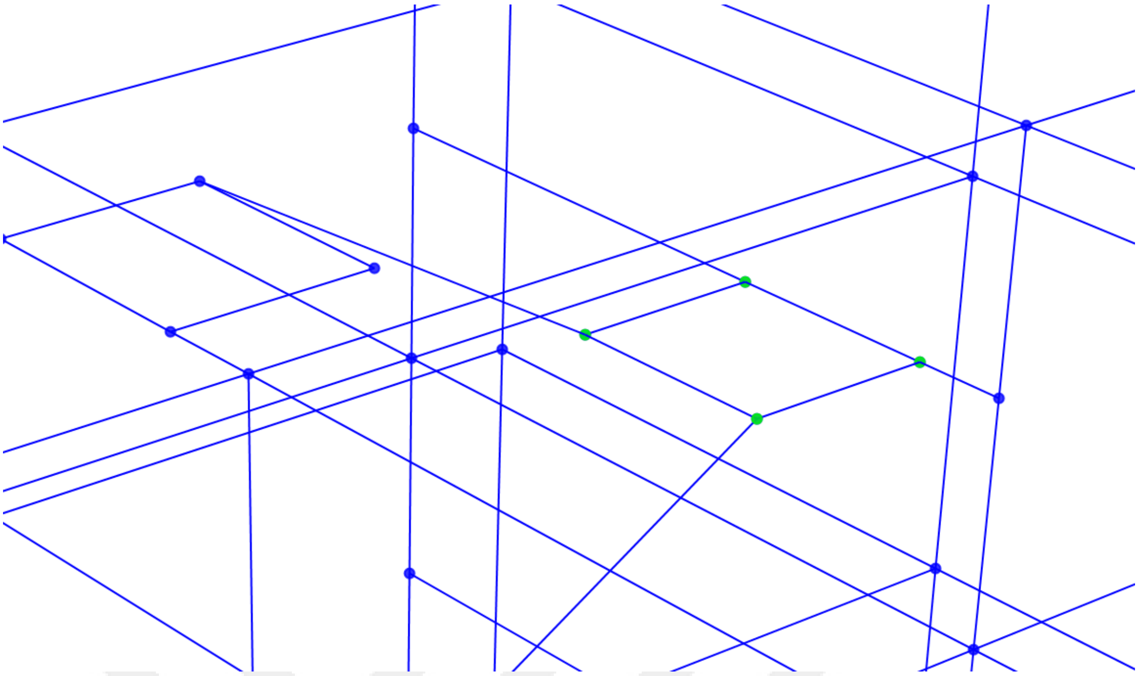


Figure 3.12 *Rigid Diaphragm of the landing between 4th and 5th floor*

3.5. Assigning Dead (G) and Live (Q) Loads

Dead load is a permanent load in a structure due to the weight of its own members. Since slabs and walls are sources of dead load, their weights are included to “dead” load case. Sap2000 automatically calculates and adds the weights of the frame members to the dead load case.

Live load is a temporary load in a structure due to the weight of people, machinery, furniture, appliances, etc. In order to choose appropriate live load in terms of safety and economic concerns, the purpose of building usage needs to be known.

The dead and live loads imposed to slabs are area loads. These loads should be transmitted to the beams without modeling slabs in order to simplify the modelling and were determined based on the TS 498 specifications. Since the given slabs are square shaped, the beams adjacent to the slab would equally share the load which can be represented by triangular tributary area per each beam. In Figure 3.13, dead loads are pentagonal-shaped due to the superposition of triangular line load due to the weight of slabs and rectangular line load due to the weight of infill walls.

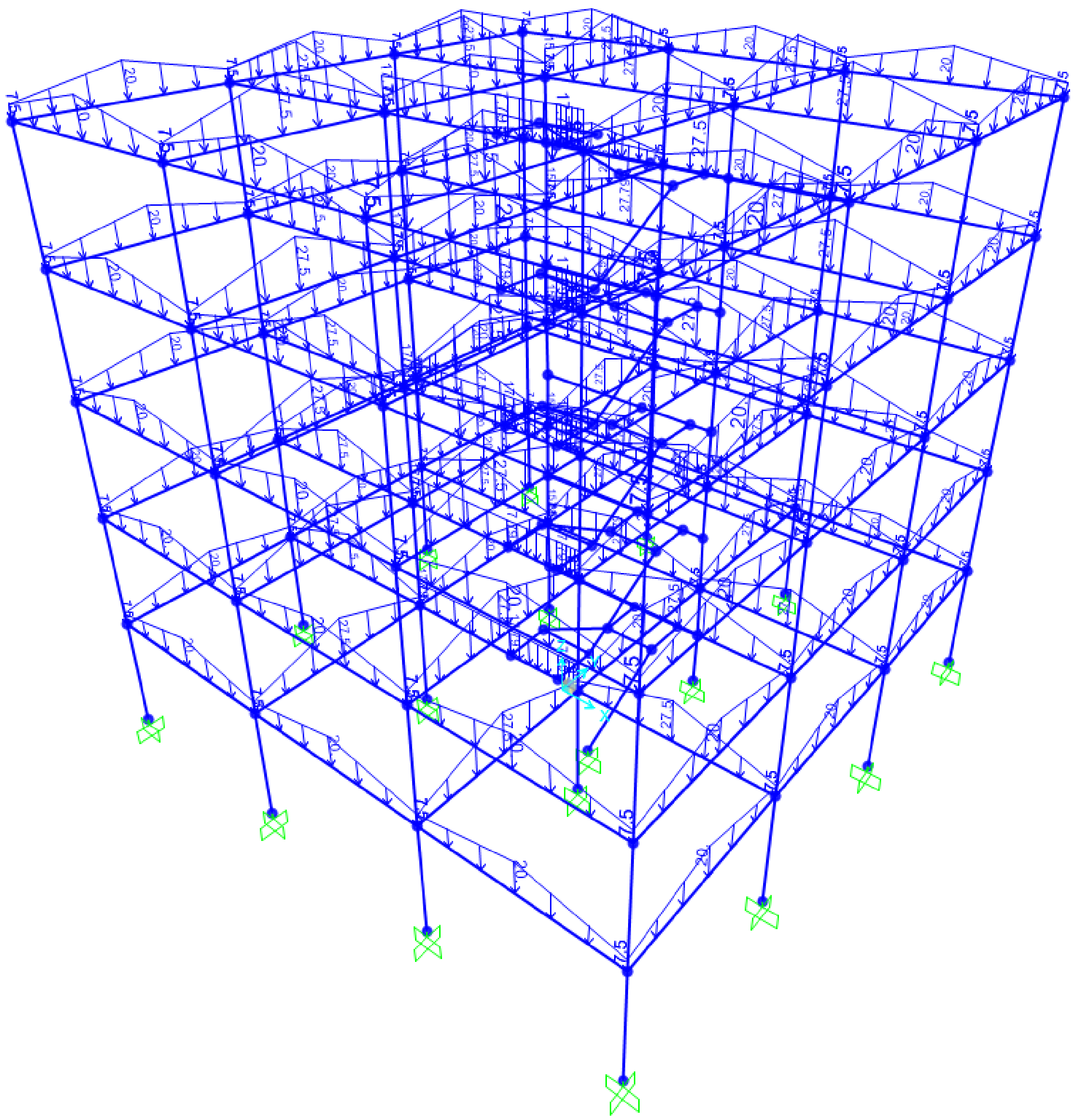


Figure 3.13 *Dead Load Case of 5x3x3_st_cent*

In Figure 3.14, the live load imposed on the beams from the slabs can be seen. Inner beams receive twice as much load as outer beams do, since the inner beams are adjacent to two slabs while the outer beams are adjacent to only one slab.

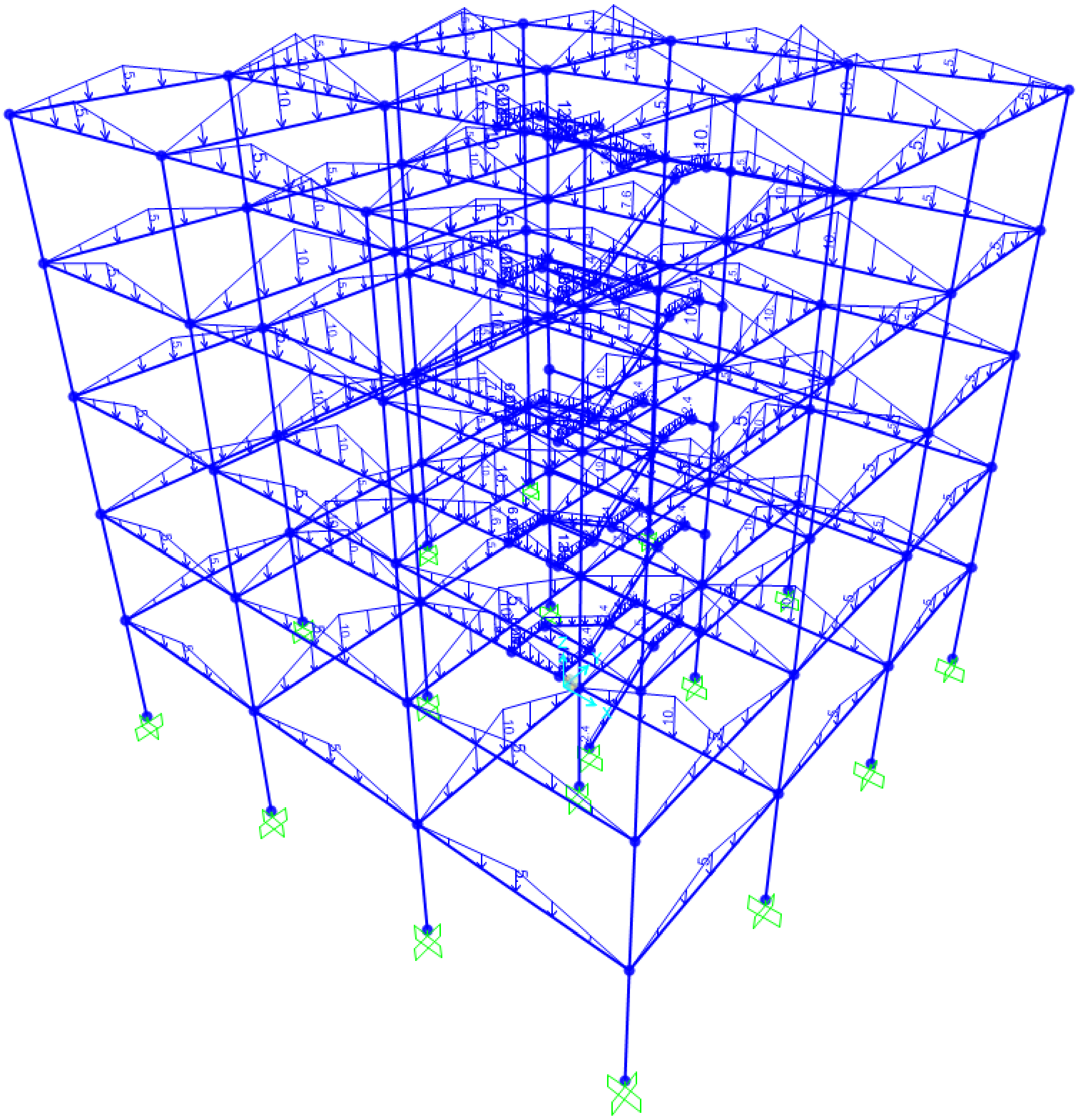


Figure 3.14 *Live Load Case of 5x3x3_st_cent*

3.6. Definition of Mass

Mass is an important parameter in dynamic analyses and used for calculating inertial effects due to self-weight of the building and temporary loads due to the weight of people and objects inside the building. G and Q load cases are used as mass source. According to Turkish Earthquake Code 2007, storey weights to be used as mass sources are calculated as follows;

$$w_i = g_i + \mathrm{n}\, q_i \quad \textbf{(3.1)}$$

Where w_i denotes storey weight of the related floor, g_i denotes dead load of the related floor, n denotes live load participation factor, q_i denotes live load of the related floor.

Table 3.1 presents the live load participation factors (n) defined as per Turkish Earthquake Code 2007. Since the models in this study are residential buildings, n factor can be taken as 0.3. The related data is defined in SAP2000 as shown in Figure 3.15.

Table 3.1 *Live load participation factors according to Turkish Earthquake Code 2007*

Purpose of Occupancy of Building	*n*
Depot, warehouse, etc.	0.80
School, dormitory, sport facility, cinema, theatre, concert hall, car park, restaurant, shop, etc.	0.60
Residence, office, hotel, hospital, etc.	0.30

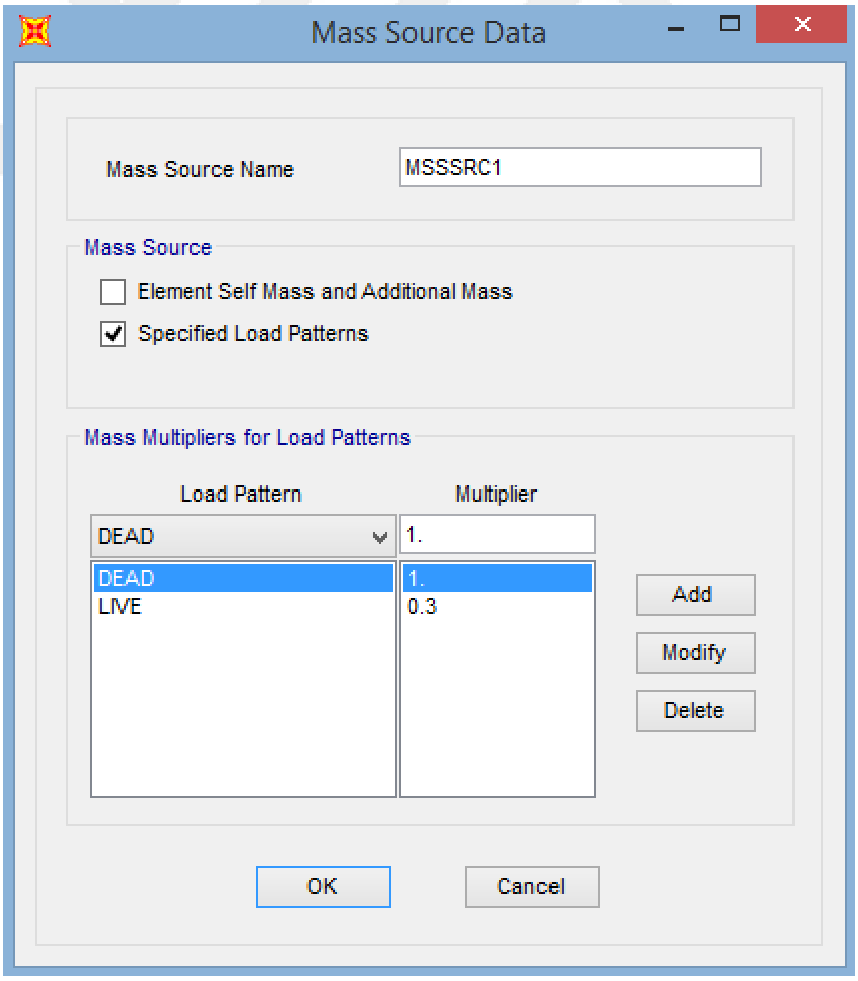


Figure 3.15 *Mass source data of the present study*

3.7. Defining Plastic Hinges

3.7.1. Concept of plastic hinge

A plastic hinge refers to the deformation of a part of a frame where non-linear deformations take place. Plastic deformation starts when the entire cross-section of a frame reaches its yield stress. When the critical force yields the cross-section “plastic hinge” occurs.

To be used for nonlinear static, and nonlinear direct-integration time-history analyses, post-yield behavior can be simulated by assigning concentrated plastic hinges to frame objects. The deformation beyond the elastic limit occurs completely within hinges, which are modeled in spesific locations while elastic behavior occurs over member length. Nonlinear behavior is achieved by the integration of the plastic strain and plastic curvature which occurs within a pre-determined hinge length, which is half of the cross-sectional dimension in the related direction as mentioned in TEC 2007. To simulate inelastic behavior, a series of hinges can be modelled and be assigned to the related locations along the member legth. Nonlinear behavior and damage thresholds are shown in Figure 3.16, where IO refers to “immediate occupancy”, LS refers to “life safety” and CP refers to “collapse prevention”.

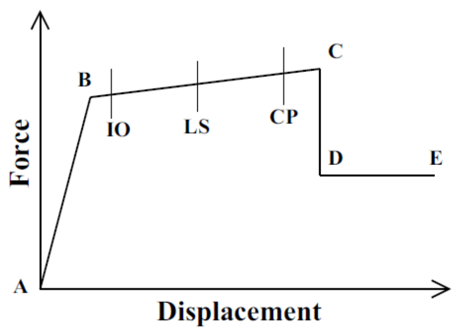


Figure 3.16 *Different Steps of Hinge Formations (Http-1)*

3.7.2. Cross-sectional analysis by using XTRACT

In order to model the non-linear behavior of the sections used, XTRACT was employed. XTRACT is a cross-sectional analysis software and is used for obtaining moment-curvature data of the defined sections. As demonstrated, cross-sectional analysis steps of 75/40 column which is at the first storey of the 8x5x5_sub_non model were performed.

3.7.2.1. Material modeling

3.7.2.1.1. Reinforcement steel modeling

In substandard models, as reinforcement steel, S220 is used. According to TEC 2007, S220 was modeled based on the material parameters as shown in Table 3.2;

Table 3.2 *Reinforcement steel properties according to TEC 2007*

Quality	f_{sy} (Mpa)	ε_{sy}	ε_{sh}	ε_{su}	f_{su} (Mpa)
S220	220	0.0011	0.011	0.16	275
S420	420	0.0021	0.008	0.10	550

Above mentioned S220 steel properties are defined to the "parabolic strain hardening model" menu of XTRACT as shown in Figure 3.17.

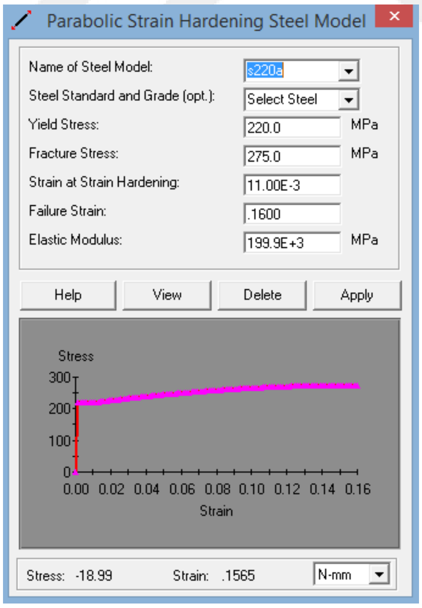


Figure 3.17 *Steel model was created according to proposed reinforcement steel properties in TEC 2007*

3.7.2.1.2. Unconfined concrete modeling

In substandard models, as concrete, C10 is used. Unconfined concrete model is created as shown in Figure 3.18;

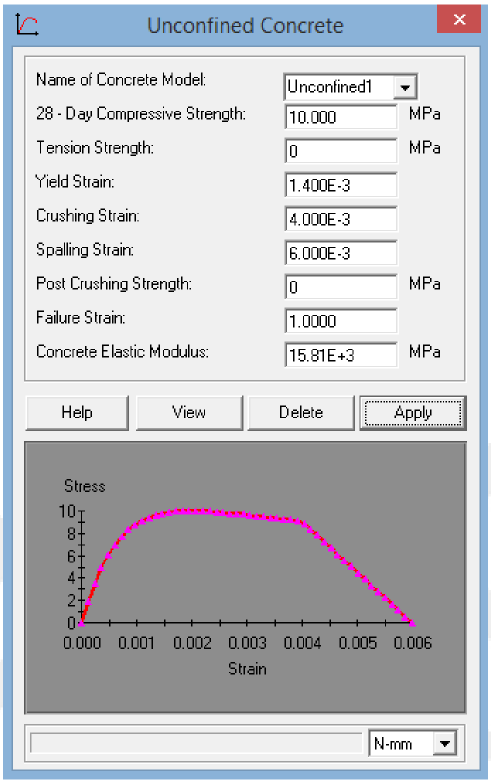


Figure 3.18 *Unconfined concrete model 75/40 substandard column*

3.7.2.1.3. Confined concrete modeling

Note that, in substandard models, the longitudinal spacing between confinement bars is 20 centimeters and is constant. Φ8 rebar is used as confinement bar. Confinement type is “single hoop” in substandard sections.

In Figure 3.19, confined concrete model properties of a 75/40 substandard column are shown.

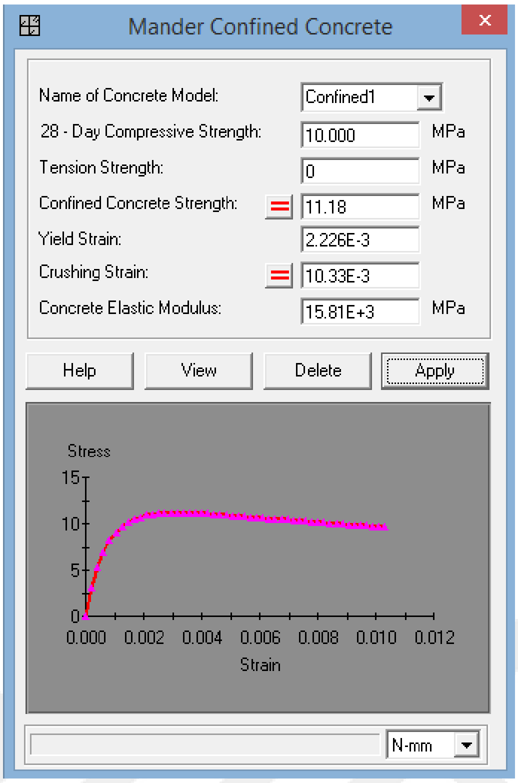


Figure 3.19 *Confined concrete model of 75/40 substandard column*

3.7.2.2. Moment-curvature analysis

After defining material models, the section in Figure 3.20 is meshed and prepared for moment-curvature analysis.

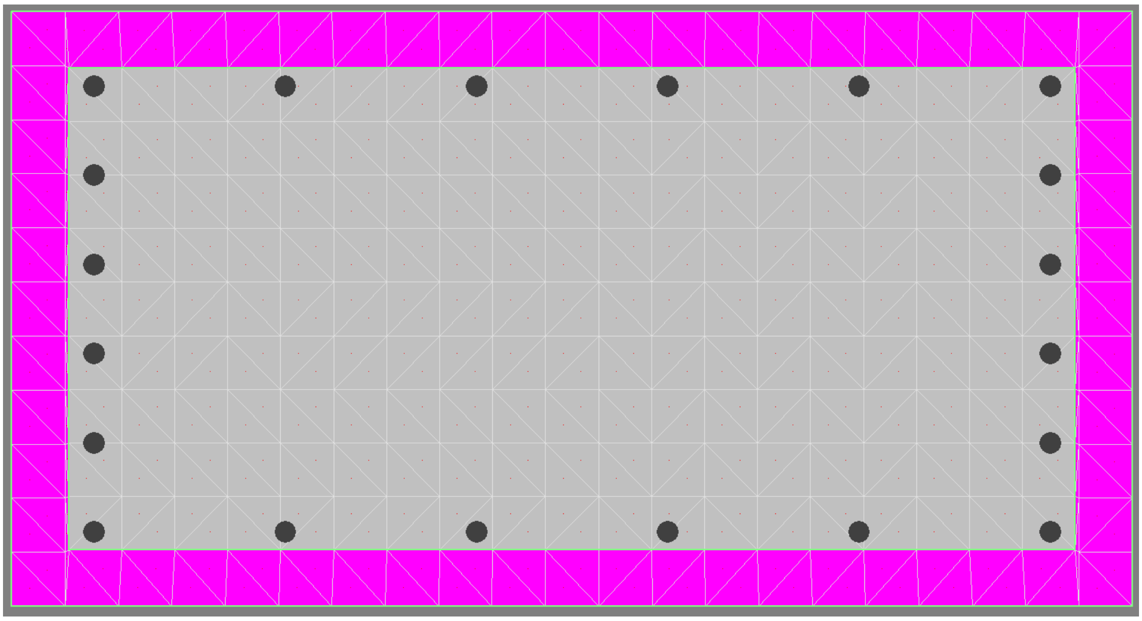


Figure 3.20 *Meshed cross-section of 75/40 substandard column*

In moment-curvature analysis, the section will be subjected to incremental moment about the selected axis. During the analysis, the loading would push the section to its limits and moment-curvature data of the related section is plotted. Since 75/40 column is not square with uniform reinforcement arrangement, the moment-curvature loading would be different about X and Y axes. For demonstration, only the input parameters for moment-curvature analysis about Y axis are presented herein which can be seen in Figure 3.21.

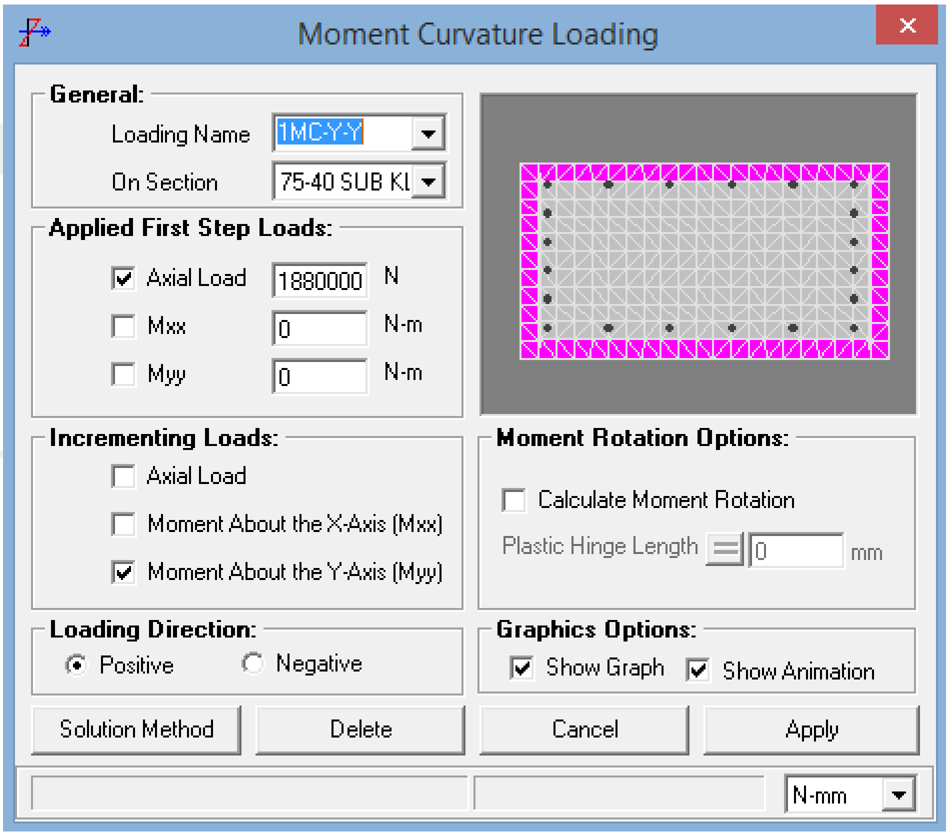


Figure 3.21 *Moment-curvature loading data of 75/40 substandard first storey column*

Axial load data must be known in advance to be defined before performing moment-curvature analysis. In order to obtain the required data, the related building was analysed in SAP2000 under G+0.3Q loading and as a result, 1880 kN of axial force was found in the related column.

3.7.2.3. *Obtaining results of the cross-sectional analysis*

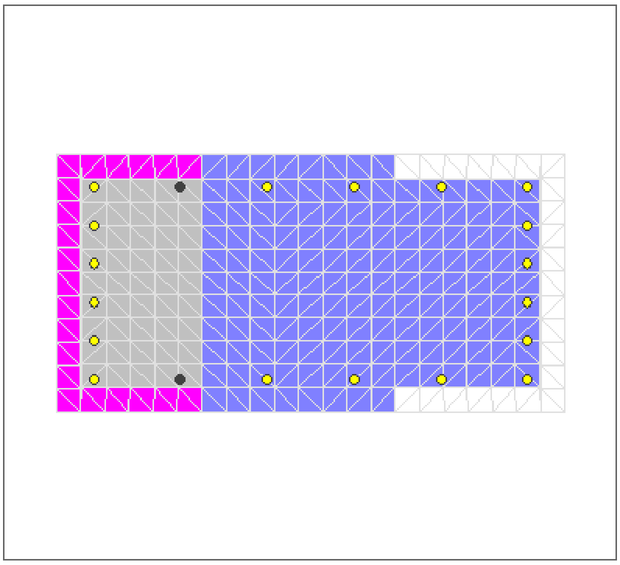
In the analysis report menu of XTRACT, ultimate curvature data and effective yield curvature data is presented and in Figure 3.22 the analysis report of a 75cm/40cm substandard column is shown as an example.

XTRACT Analysis Report - Demonstration

Section Name:	75*40sub
Loading Name:	MC-1-YY
Analysis Type:	Moment Curvature

Section Details:

X Centroid:	-.1168E-3 mm
Y Centroid:	2.640E-3 mm
Section Area:	300.0E+3 mm^2

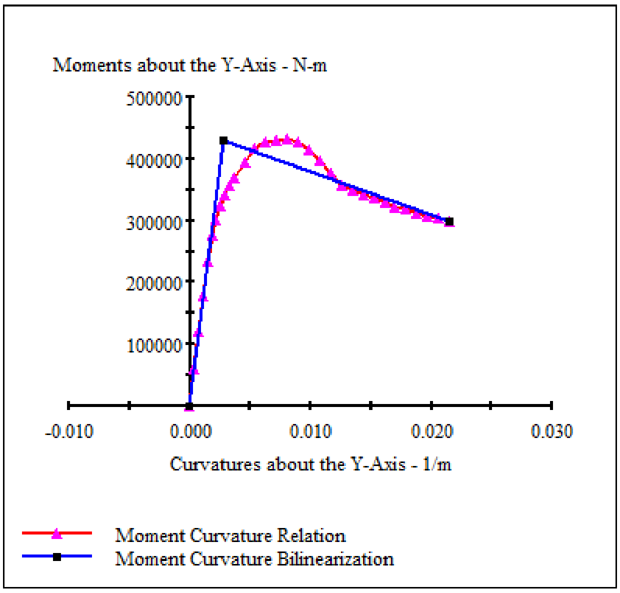


Loading Details:

Constant Load - P:	1.880E+6 N
Incrementing Loads:	Myy Only
Number of Points:	30
Analysis Strategy:	Displacement Control

Analysis Results:

Failing Material:	Confined1
Failure Strain:	10.33E-3 Compression
Curvature at Initial Load:	-.6512E-11 1/m
Curvature at First Yield:	1.815E-3 1/m
Ultimate Curvature:	21.48E-3 1/m
Moment at First Yield:	276.5E+3 N-m
Ultimate Moment:	298.4E+3 N-m
Centroid Strain at Yield:	.5196E-3 Comp
Centroid Strain at Ultimate:	3.347E-3 Comp
N.A. at First Yield:	-286.2 mm
N.A. at Ultimate:	-155.8 mm
Energy per Length:	7406 N
Effective Yield Curvature:	2.825E-3 1/m
Effective Yield Moment:	430.4E+3 N-m
Over Strength Factor:	.6934
EI Effective:	1.52E+8 N-m^2
Yield EI Effective:	-7.073E+6 N-m^2
Bilinear Harding Slope:	-4.644 %
Curvature Ductility:	7.603



Comments:

User Comments

Figure 3.22 *Moment-curvature analysis report of 75/40 substandard column*

In the moment-curvature input menu for the plastic hinge definition of Sap2000, it is required to define plastic moment-curvature data. In order to obtain plastic moment-curvature, effective yield curvature must be subtracted from ultimate curvature. The obtained result represents point "C" in Figure 3.23, where vertical axis stands for "force" and horizontal axis stands for "deformation";

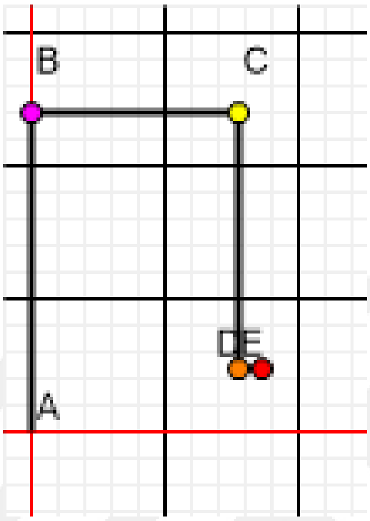


Figure 3.23 *Moment-curvature curve of 75/40 substandard column*

According to TEC 2007, damage thresholds with the moment-curvature value between point "B" and "C" should be defined as follows;

(a) For Immediate Occupancy (IO), upper bounds of the unconfined concrete compressive strain in the outermost fiber of the section and the reinforcement steel strain:

$$(\varepsilon_{cu})_{IO} = 0.0035 \quad ; \quad (\varepsilon_s)_{IO} = 0.010 \tag{3.2}$$

(b) For Life Safety (LS), upper bounds of the concrete unit pressure deformation in the outermost fiber of hoop and the reinforcement steel unit deformation volitions:

$$(\varepsilon_{cg})_{LS} = 0.0035 + 0.01(\rho_s/\rho_{sm}) \leq 0.0135 \quad ; \quad (\varepsilon_s)_{LS} = 0.040 \tag{3.3}$$

(c) For Collapse Prevention (CP), upper bounds of the concrete unit pressure deformation in the outmost fiber of hoop and the reinforcement steel unit deformation volitions:

$$(\varepsilon_{cg})_{CP} = 0.004 + 0.014(\rho_s/\rho_{sm}) \leq 0.018 \quad ; \quad (\varepsilon_s)_{CP} = 0.060 \tag{3.4}$$

Where ε_{cu} is unconfined concrete strain in the outermost fiber of the section of the cross section , ε_s is reinforcement steel strain, ε_{cg} is confined concrete strain in the outermost fiber of the section inside of the lateral reinforcement, ρ_s is volumetric ratio of lateral reinforcement which are available in the cross section and arranged as “special seismic hoops and crossties” according to 3.2.8 of TEC 2007, ρ_{sm} is volumetric ratio of the transverse reinforcement necessary to be required in the cross section according to 3.3.4, 3.4.4 of TEC 2007.

According to equations above, the data extracted from interactive output menu of XTRACT and processed in excel in order to obtain plastic moment-curvature value.

Point “D” in Figure 3.22 has the same curvature with point C yet it has %20 of the moment carrying capacity of the point C. Point “E” has the same moment carrying capacity of the point D, yet it has %10 more of the curvature of the point D.

According to these, the data is defined in Sap2000.

3.7.2.3. Plastic hinge definition in Sap2000

According to TEC 2007, plastic hinge length should be taken as half of the section length in the active direction. In a 75/40 column, plastic hinge length was taken as 0.375 meters as can be seen in Figure 3.24.

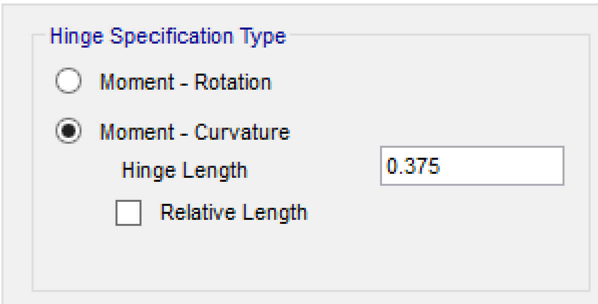


Figure 3.24 *Defining plastic hinge length*

Moment-curvature dependence of the section assumed to be doubly symmetric about M2 and M3 axis. As curve angles for moment-curvature curves, 0 and 90 degrees are defined. Since the moment-curvature dependence is symmetric, 180 and 270 degrees would be automatically added to curve angles for moment-curvature curves. The symmetry condition and the angle data is shown in Figure 3.25.

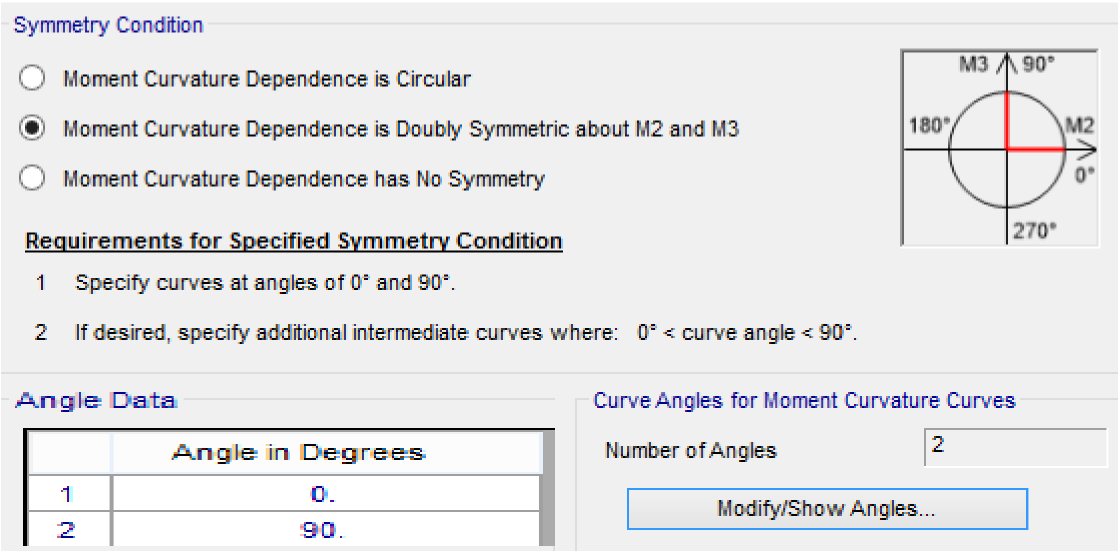


Figure 3.25 *Symmetry and angle data of the hinge*

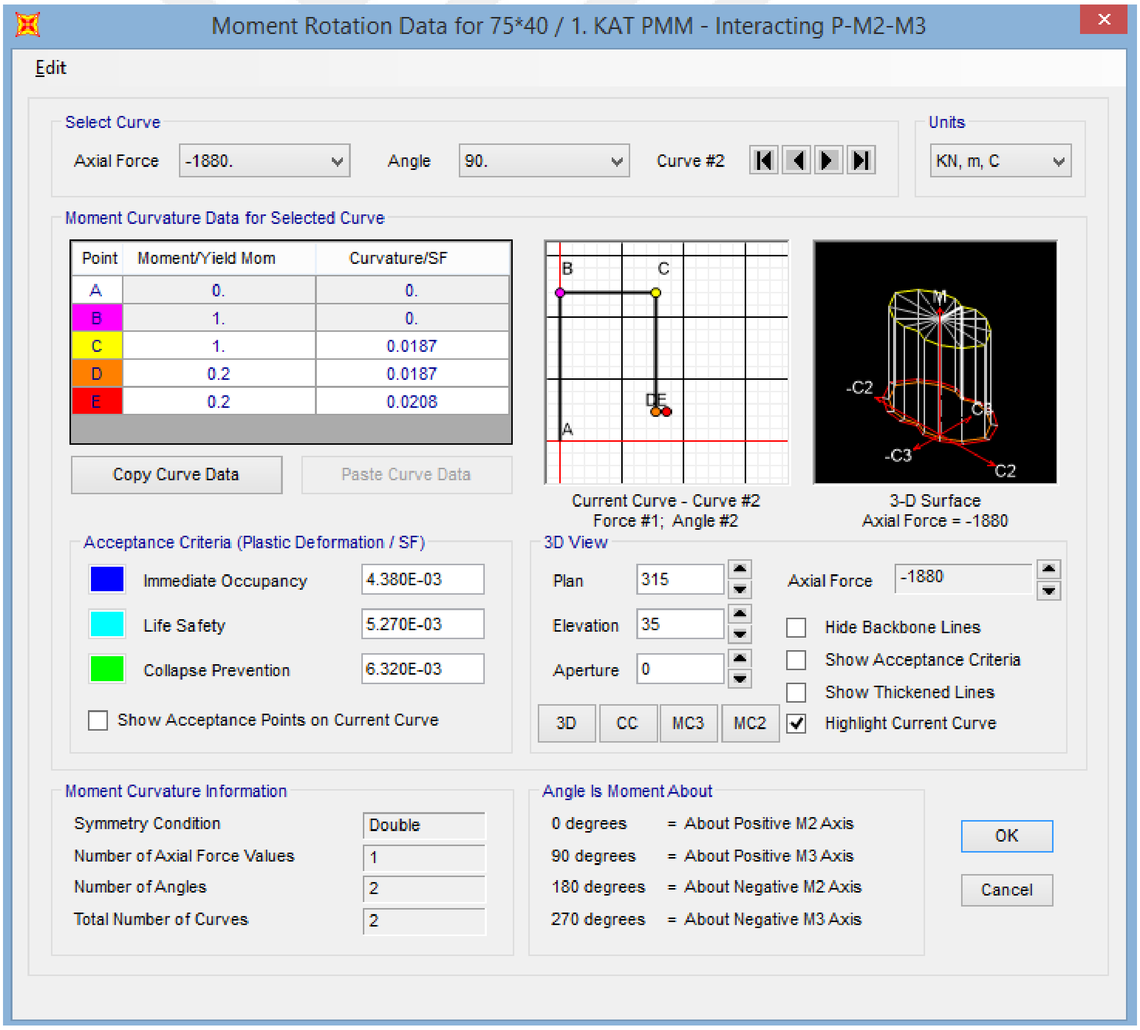


Figure 3.26 *Moment-curvature data for 75/40 column*

After defining axial force data, moment-curvature data is defined as shown in Figure 3.26. P-M2-M3 interaction surface data obtained from auto-hinge feature of Sap2000 for each section as shown in Figure 3.27.

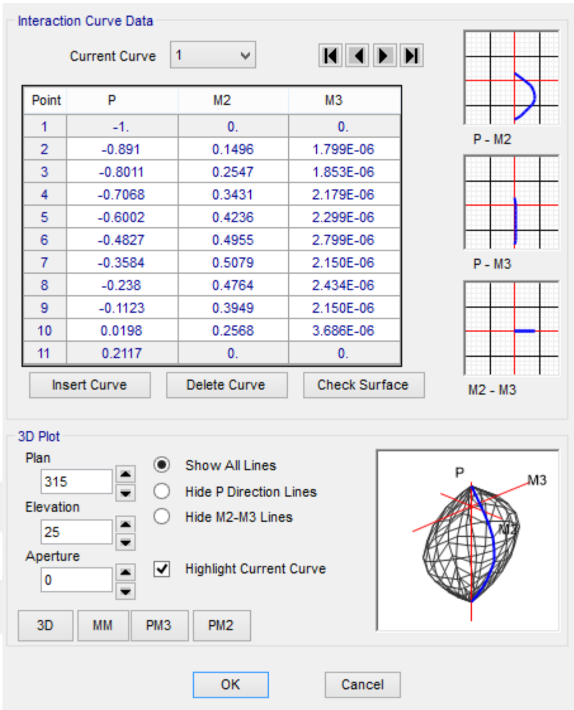


Figure 3.27 *Interaction curve data*

3.8. Defining Shear Hinges

Since the previously defined flexural plastic hinges are all about moment carrying capacity -under certain amount of axial force- of the section, they are uncoupled with the shear forces. In order to make sure that the section has not reached to its shear force carrying capacity, shear hinges need to be defined.

In order to calculate shear force carrying capacity first, diagonal cracking strength V_{cr}, needs to be calculated according to TS500 as given in Equation (3.4);

$$V_{cr} = 0.65\, f_{ct}\, b_w\, d\, (1 + \gamma\, N_d / A_c) \qquad \textbf{(3.4)}$$

Where, f_{ct} is tensile strength of concrete, b_w is the width of the frame, d is the depth of the beam, N_d is design axial load, γ is a factor, A_c is the cross-sectional area of the frame. N_d is positive in this equation whether it is a compressive or tensile force. In the case of axial compression, γ should be taken as $\gamma = 0.07$. In the case of axial tension, γ should be taken as $\gamma = -0.3$. When the axial tensile stress calculated is lower than 0.5 Mpa, γ can be taken as zero.

Secondly, contribution of concrete to shear strength V_c, needs to be calculated as shown in Equation (3.5);

$$V_c = 0.8\, V_{cr} \tag{3.5}$$

Thirdly, contribution of shear reinforcement V_w needs to be calculated as shown in Equation (3.6);

$$V_w = \frac{A_{sw}}{s} f_{yw}\, d \tag{3.6}$$

Where A_{sw} is total cross-sectional area of the shear reinforcement, s is the spacing between confinement bars, f_{ywd} is the yield strength of shear reinforcement.

Now, shear strength V_r can be calculated as shown in Equation (3.7);

$$V_r = V_c + V_w \tag{3.7}$$

This process has been applied to determine the shear strength of 75/40 substandard first floor column in 2-2 direction (Figure 3.28) as an example;

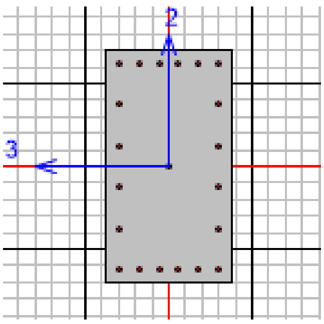


Figure 3.28 *Local axes of 75/40 column*

Characteristic compression strength of the concrete = 10 MPa

f_{ctd} = 1.1 Mpa

b_w = 400 mm

d = 750 mm

N_d = 1880000 N

$\gamma = 0.07$

$A_c = 300000 \text{ mm}^2$

$A_{sw} = 100.5 \text{ mm}^2$

$S = 200 \text{ mm}$

$f_{ywd} = 220 \text{ MPa}$

$$V_{cr} = 0.65 \ x \ 1.1 \ x \ 400 \ x \ 750 \ x \ \left(1 + 0.07 \frac{1880000}{300000}\right) = 310.5 \ kN$$

$$V_c = 0.8 \ x \ 310.5 = 248.4 \text{ kN}$$

$$V_w = \frac{100.5}{200} \ x \ 220 \ x \ 750 = 82.5 \ kN$$

$$V_r = 248.4 + 82.5 = 330.9 \ kN$$

Shear capacities of all columns in both directions have been calculated for each storey in order to consider the changes in axial force since increment in axial compression is advantageous in terms of shear capacity.

In Figures 3.29 and 3.30, the results have been defined in Sap2000 as force controlled (brittle) hinges.

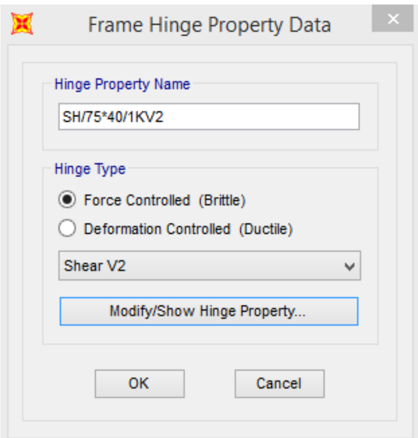


Figure 3.29 *Frame hinge property data - 1*

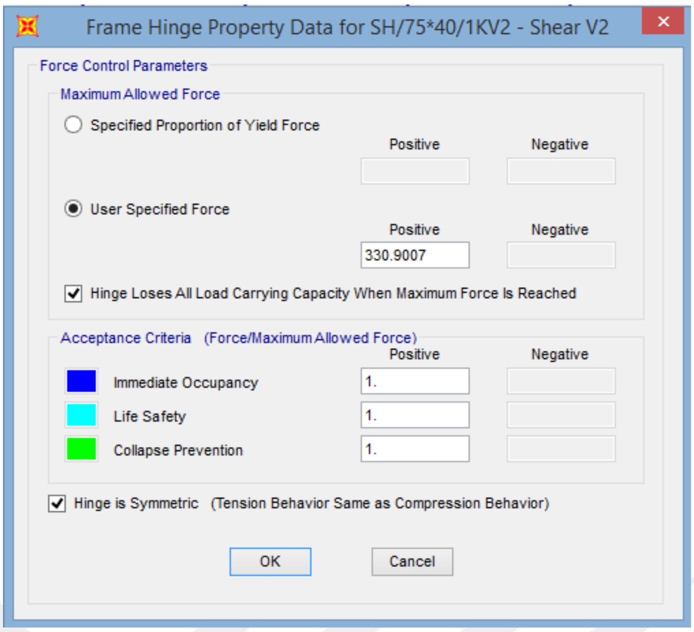


Figure 3.30 *Frame hinge property data – 2*

3.9. Assigning Hinges to Frames

In the previous sections, it has been mentioned that there are deformation controlled ductile plastic hinges for flexural behavior and force controlled brittle shear hinges.

The plastic hinges such as P-M2-M3 and M3 hinges shall be assigned to the ends of the frames where the maximum moments occur. P-M2-M3 hinges were assigned to ends of columns, staircase frames and landing beams which are not part of a rigid floor. M3 hinges were assigned to the ends of the beams that are part of a rigid floor.

The shear force is constant in a section which means that shear hinges can be assigned to anywhere in a section. In this study, they have been assigned to mid-points of all frames.

4. MODAL ANALYSIS AND RESULTS

4.1. Definition of Modal Analysis

Modal analysis is a linear dynamic-response procedure which evaluates and superposes free-vibration mode shapes and is used for determining the dynamic characteristics of a system such as natural vibration period, mode shapes and modal participating mass ratio (MPMR), etc.

A system with a certain number of degrees of freedom will have same number of mode shapes. Each of these mode shapes is an independent and normalized displacement pattern which may be scaled and superposed to create a resultant displacement pattern, as shown in Figure 4.1;

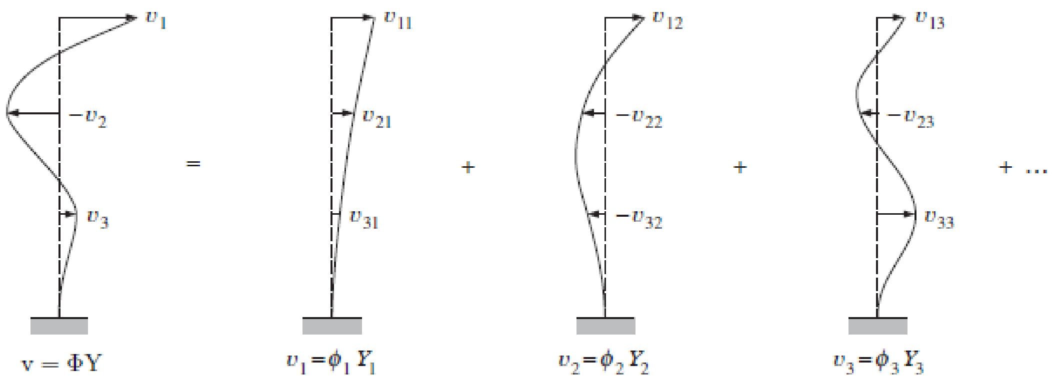


Figure 4.1 *Resultant displacement and modal components (Http-2)*

MPMR is calculated for each mode in order to investigate the contribution of that mode to the overall response. In other words, MPMR is a critical indicator in idetifying the importance of each mode. Therefore, the higher the MPMR for a mode, the more dominant of that mode on the overall structural response.

4.2. Parameters of Modal Analysis

In modal analysis, the analysis was run from zero initial conditions –unstressed state. As type of modes, “*Eigen Vectors”* was used.

Numbers of modes were chosen between 1 and 12 and convergence tolerance was chosen as 1.000E-9 which are the default values in SAP2000.

4.3. Results of Modal Analysis

For this study, only the translational mode in Y direction was considered since the main scope of this study is to evaluate the effects of staircases parallel to the flight running direction. The results were given in terms of modal participating mass ratios (in translational movement in Y direction (U_y) and torsion about Z axis (R_z)) of first modes in Table 4.1, 4.2 and 4.3, and natural vibration periods of first modes in Table 4.4, 4.5 and 4.6.

Table 4.1 *Modal participating mass ratios (MPMR) of 3 storey models*

	3 Storey Models									
	Standard					Substandard				
	Non	Corner		Centric		Non	Corner		Centric	
Span No.	**MPMR**	**MPMR**	**%Diff.**	**MPMR**	**%Diff.**	**MPMR**	**MPMR**	**%Diff.**	**MPMR**	**%Diff.**
3x3 in Y	0.865	0.771	10.867	0.855	1.156	0.879	0.820	6.712	0.871	0.910
3x3 in Tors.	0.000	0.067	N/A	0.009	N/A	0.000	0.044	N/A	0.005	N/A
5x5 in Y	0.865	0.629	27.259	0.852	1.470	0.878	0.695	20.794	0.867	1.247
5x5 in Tors.	0.000	0.103	N/A	0.010	N/A	0.000	0.090	N/A	0.009	N/A

Table 4.2 *Modal participating mass ratios (MPMR) of 5 storey models*

	5 Storey Models									
	Standard					Substandard				
	Non	Corner		Centric		Non	Corner		Centric	
Span No.	**MPMR**	**MPMR**	**%Diff.**	**MPMR**	**%Diff.**	**MPMR**	**MPMR**	**%Diff.**	**MPMR**	**%Diff.**
3x3 in Y	0.832	0.763	8.374	0.826	0.698	0.844	0.818	3.105	0.841	0.307
3x3 in Tors.	0.000	0.050	N/A	0.006	N/A	0.000	0.031	N/A	0.003	N/A
5x5 in Y	0.832	0.589	29.219	0.818	1.649	0.842	0.672	20.245	0.830	1.452
5x5 in Tors.	0.000	0.100	N/A	0.010	N/A	0.000	0.090	N/A	0.011	N/A

Table 4.3 *Modal participating mass ratios (MPMR) of 8 storey models*

	8 Storey Models									
	Standard					Substandard				
	Non	Corner		Centric		Non	Corner		Centric	
Span No.	**MPMR**	**MPMR**	**%Diff.**	**MPMR**	**%Diff.**	**MPMR**	**MPMR**	**%Diff.**	**MPMR**	**%Diff.**
3x3 in Y	0.807	0.759	6.016	0.805	0.344	0.816	0.796	2.392	0.815	0.175
3x3 in Tors.	0.000	0.036	N/A	0.004	N/A	0.000	0.015	N/A	0.001	N/A
5x5 in Y	0.807	0.644	20.266	0.805	0.301	0.817	0.742	9.155	0.815	0.207
5x5 in Tors.	0.000	0.060	N/A	0.002	N/A	0.000	0.038	N/A	0.002	N/A

Table 4.4 *First mode vibration periods (T_1) of 3 storey models*

	3 Storey Models									
	Standard					Substandard				
	Non	Corner		Centric		Non	Corner		Centric	
Span No.	**T_1 (s)**	**T_1 (s)**	**%Diff.**	**T_1 (s)**	**%Diff.**	**T_1 (s)**	**T_1 (s)**	**%Diff.**	**T_1 (s)**	**%Diff.**
3x3	0.511	0.490	4.110	0.482	5.734	0.642	0.612	4.673	0.610	4.984
5x5	0.530	0.522	1.509	0.516	2.642	0.667	0.657	1.499	0.652	2.249

Table 4.5 *First mode vibration periods (T1) of 5 storey models*

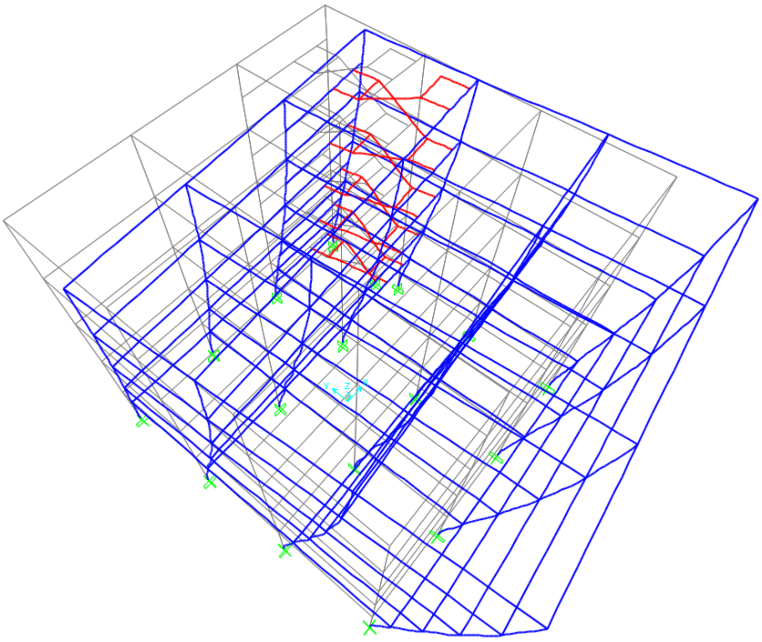
	5 Storey Models									
	Standard					Substandard				
	Non	Corner		Centric		Non	Corner		Centric	
Span No.	T_1 (s)	T_1 (s)	%Diff.	T_1 (s)	%Diff.	T_1 (s)	T_1 (s)	%Diff.	T_1 (s)	%Diff.
3x3	0.806	0.774	3.970	0.761	5.583	1.024	0.984	3.906	0.973	4.980
5x5	0.840	0.829	1.310	0.817	2.738	1.028	1.017	1.070	1.008	1.946

Table 4.6 *First mode vibration periods (T1) of 8 storey model*s

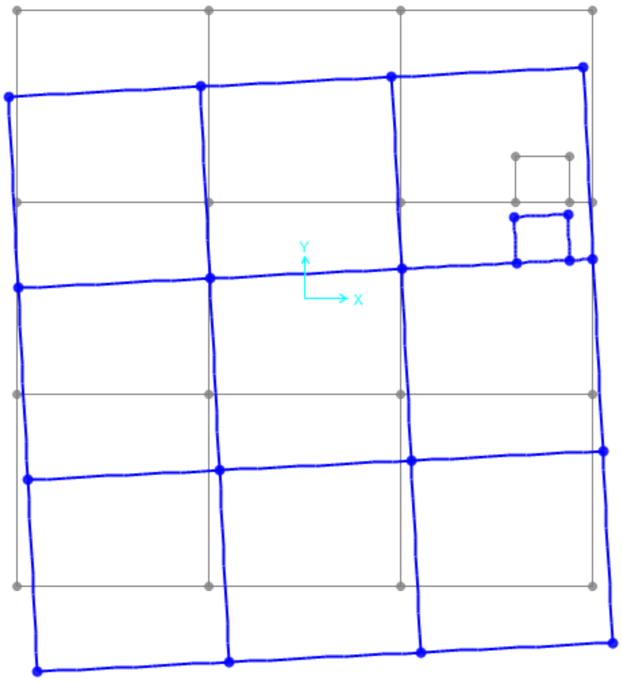
	8 Storey Models									
	Standard					Substandard				
	Non	Corner		Centric		Non	Corner		Centric	
Span No.	T_1 (s)	T_1 (s)	%Diff.	T_1 (s)	%Diff.	T_1 (s)	T_1 (s)	%Diff.	T_1 (s)	%Diff.
3x3	1.195	1.154	3.431	1.128	5.607	1.397	1.360	2.649	1.350	3.364
5x5	1.216	1.202	1.151	1.192	1.974	1.440	1.427	0.903	1.419	1.458

From the results of modal analysis in terms of modal participating mass ratios and natural vibration periods of the first mode (translational mode in Y direction);

- As the staircases are included to the mathematical model, MPMR in U_Y of mode-1 decreases which indicates that translational modes are influenced by the torsional behavior with the inclusion of staircases as shown in Figure 4.2. A similar result was found by Xu and Li (2012).
- Eccentrically placed staircases have a greater effect on decreasing the MPMR by causing the non-uniform distribution of stiffness in building layout.
- Staircases decreased the natural vibration period of the buildings owing to their local stiffening effect.
- As the number of spans and stories increase, natural vibration periods of the models increased.
- Eccentric staircases caused less decrement in natural vibration period with respect to centric staircases. Thus, it can be said that centric staircased buildings are stiffer.
- For buildings with the same number of floors and spans, substandard ones have longer natural vibration periods with respect to standard ones.



(a)



(b)

Figure 4.2 *Fundamental mode shape of 5x3x3_sub_cor (a) perspective view (b) top view*

5. NONLINEAR STATIC PUSHOVER ANALYSIS AND RESULTS

5.1. Definition of Nonlinear Static Pushover Analysis

Nonlinear static pushover analysis (NSPA) is a method where a structure is subjected to gravity loading and a monotonic displacement-controlled lateral load pattern which increases step by step through linear and non-linear behavior until an ultimate limit state is reached.

Lateral load represents the base shear induced by earthquake loading, and in this study, its configuration is determined to be proportional to the first mode shape, which is translational mode in Y direction.

By using the output obtained from NSPA, a static-pushover curve can be plotted.

5.2. Nonlinear Static Pushover Parameters of the Current Study

5.2.1. “NSPA Grav” load case

Before performing a nonlinear static pushover analysis (NSPA), the analysis model needs to be loaded in a nonlinear static load case in a single step according to G+0.3Q load combination which represents gravity loads, as proposed in TEC 2007. P-Delta effects were taken into account. These settings were defined in SAP2000 as shown in Figure 5.1.

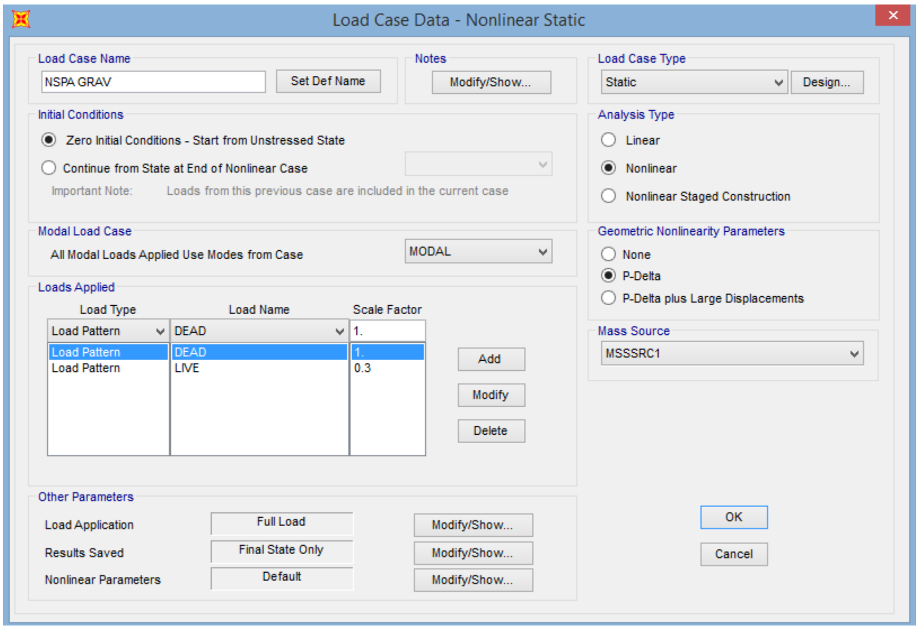


Figure 5.1 *Parameters of "NSPA Grav" load case*

5.2.2. NSPA Y load case

After defining a nonlinear load case for gravity loads, a pushover load case for desired direction can be defined as can be seen in Figure 5.2. Lateral loads are applied to the models according to dominant mode shape in Y direction which is mode-1 in all models. In order to take nonlinear gravity load case "NSPA Grav" into account, "Continue from State at End of Nonlinear Case NSPA Grav" option was chosen. "Displacement Control" option was chosen as "Load Application Control" parameter. Models were loaded to a displacement which is 4% of their total height (4% displacement refers to collapse prevention, CP, limit state according to TEC 2007). Displacements were monitored from the joints that are near the center of floor at the roof. P-Delta effects were taken into account.

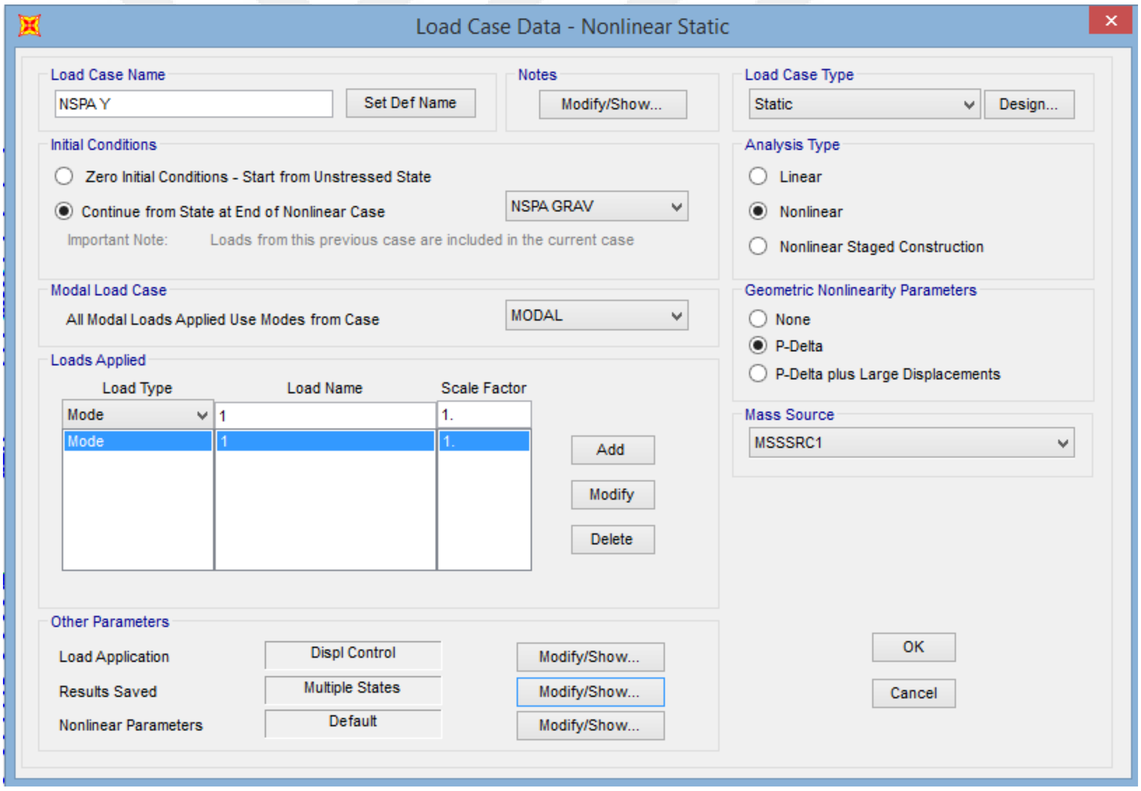


Figure 5.2 *Parameters of "NSPA Y" load case*

5.3. Results of NSPA

The results of NSPA in Y direction are presented herein. Base shear (F) / weight (W) of the building values are shown in vertical axis while roof displacement (D) / height (H) of the building values are shown in the horizontal axis.

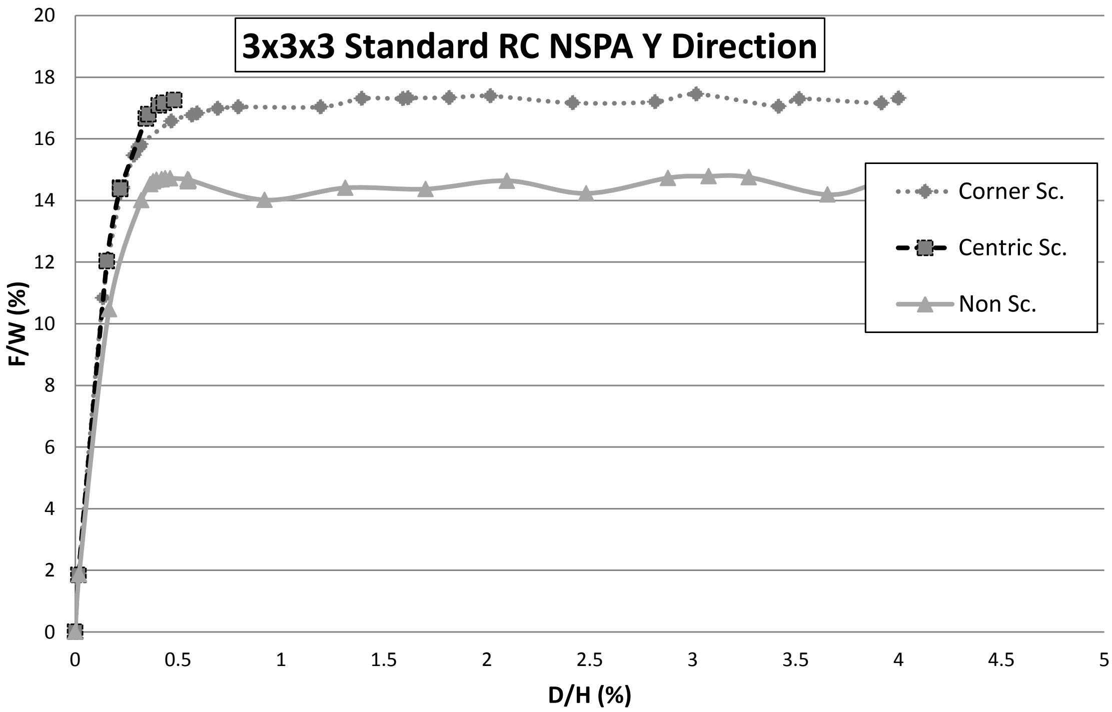


Figure 5.3 *3x3x3 Standard RC NSPA Y Direction*

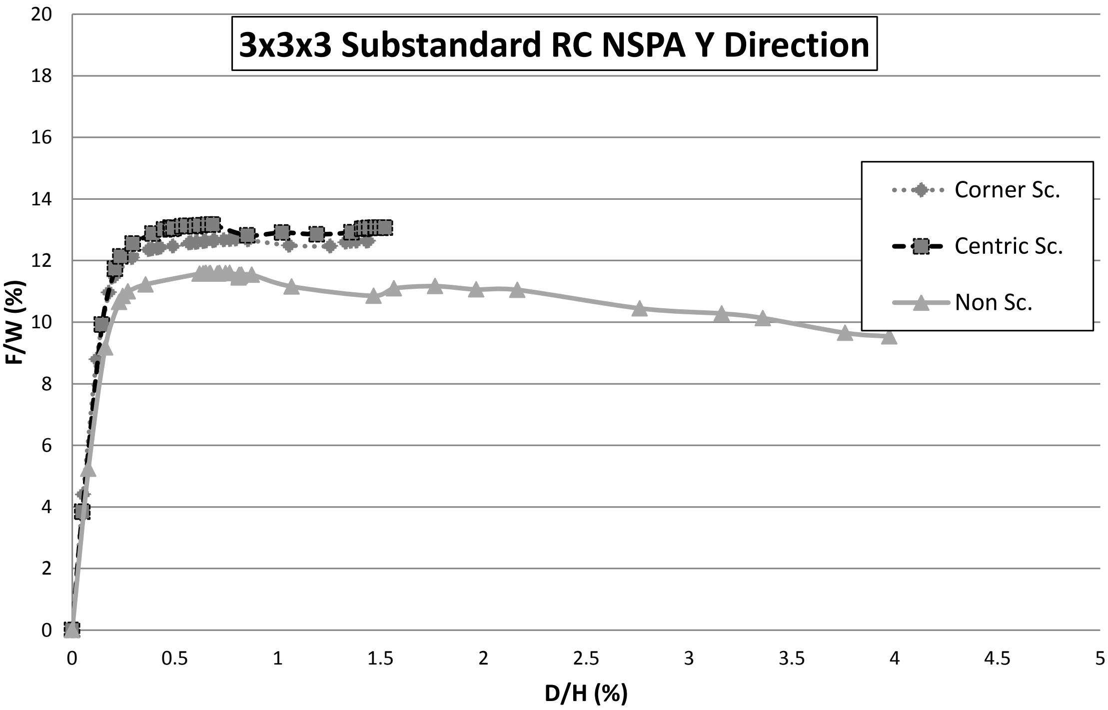


Figure 5.4 *3x3x3 Substandard RC NSPA Y Direction*

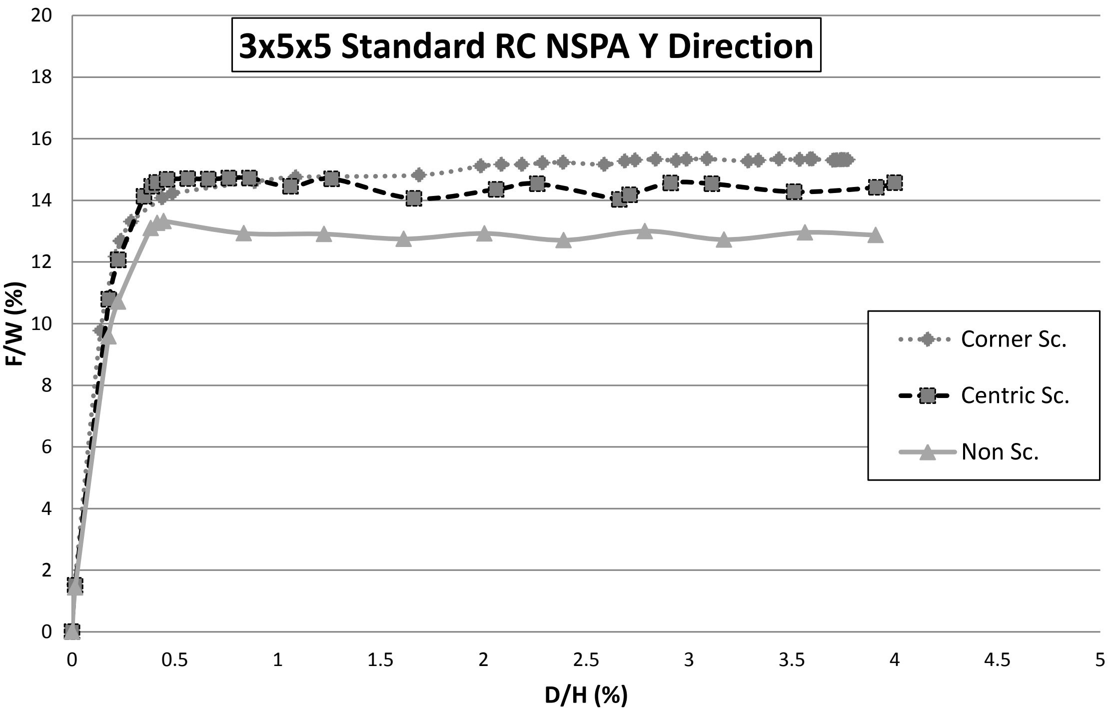


Figure 5.5 *3x5x5 Standard RC NSPA Y Direction*

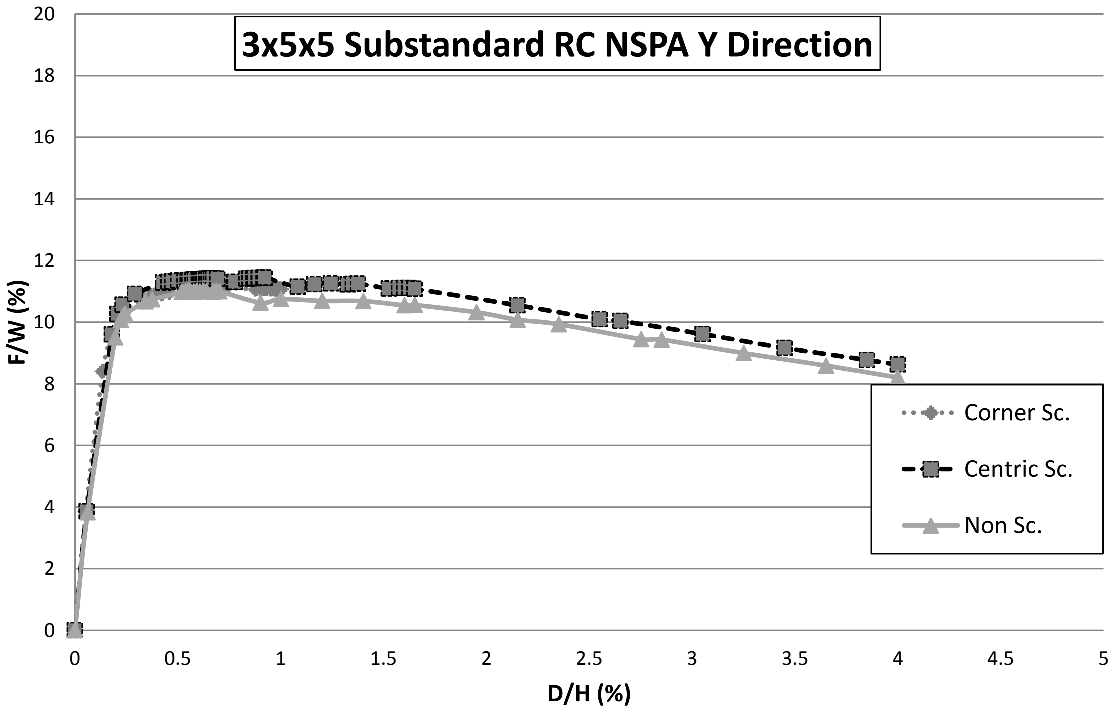


Figure 5.6 *3x5x5 Substandard RC NSPA Y Direction*

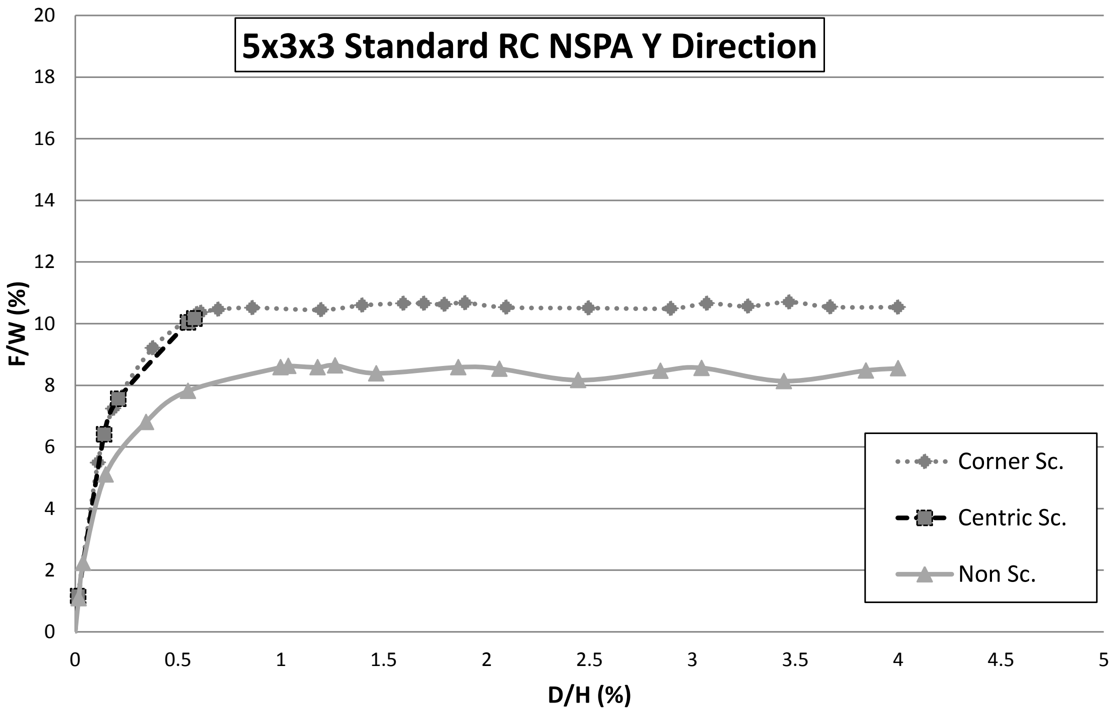


Figure 5.7 *5x3x3 Standard RC NSPA Y Direction*

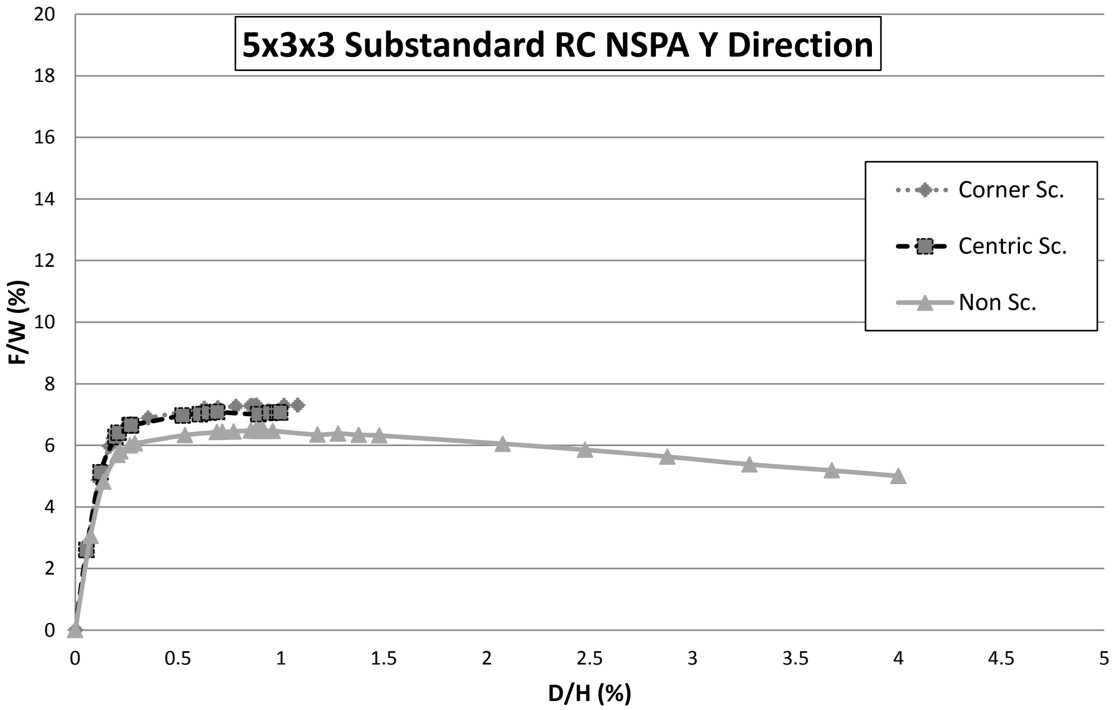


Figure 5.8 *5x3x3 Substandard RC NSPA Y Direction*

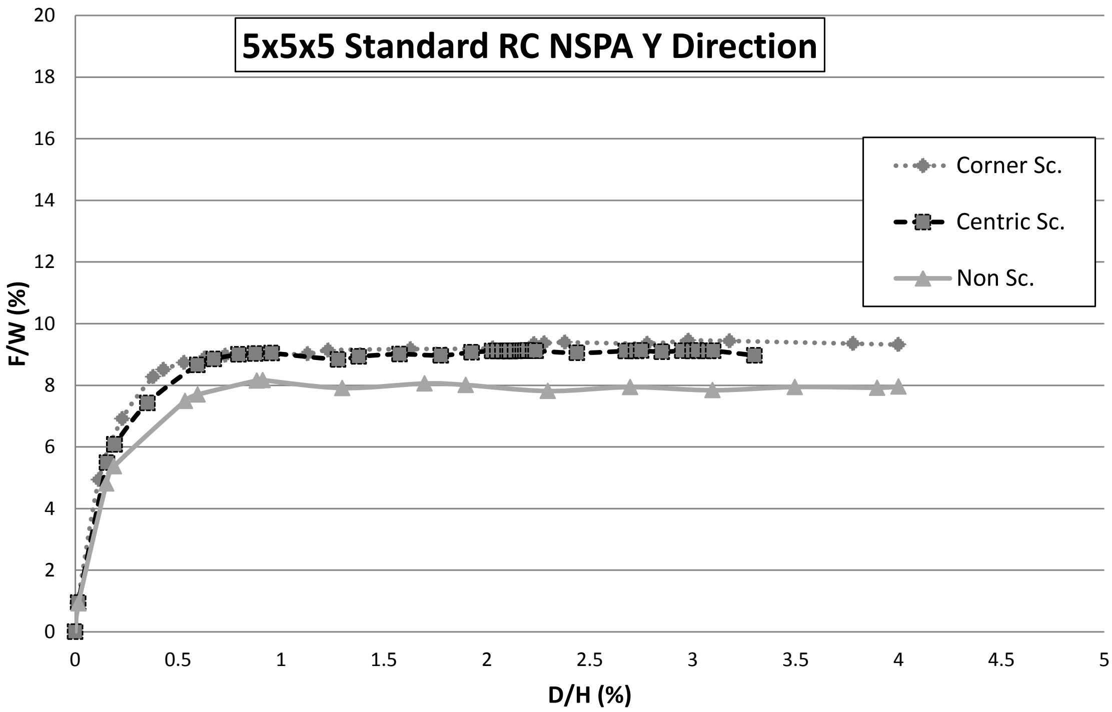


Figure 5.9 *5x5x5 Standard RC NSPA Y Direction*

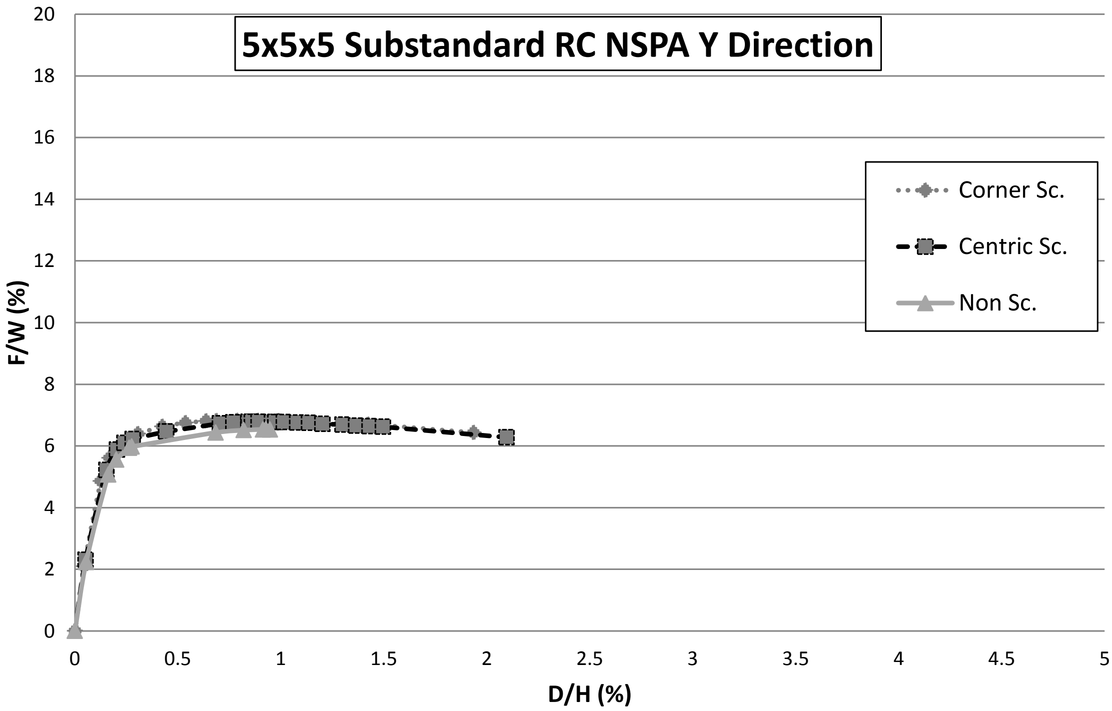


Figure 5.10 *5x5x5 Substandard RC NSPA Y Direction*

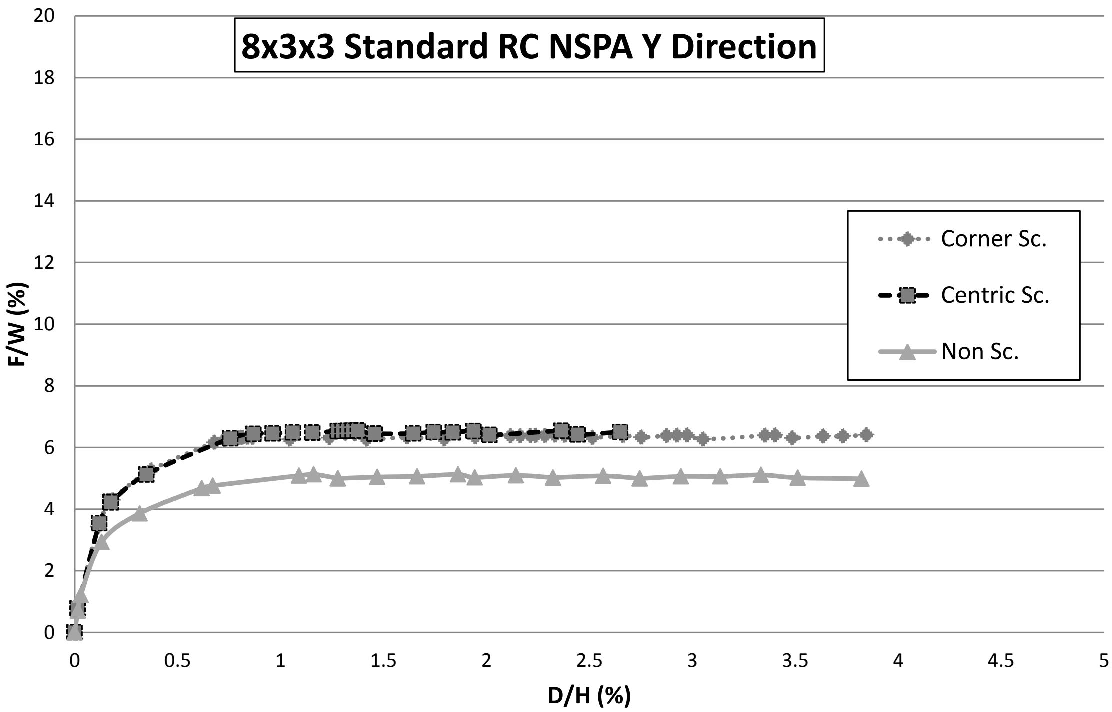


Figure 5.11 *8x3x3 Standard RC NSPA Y Direction*

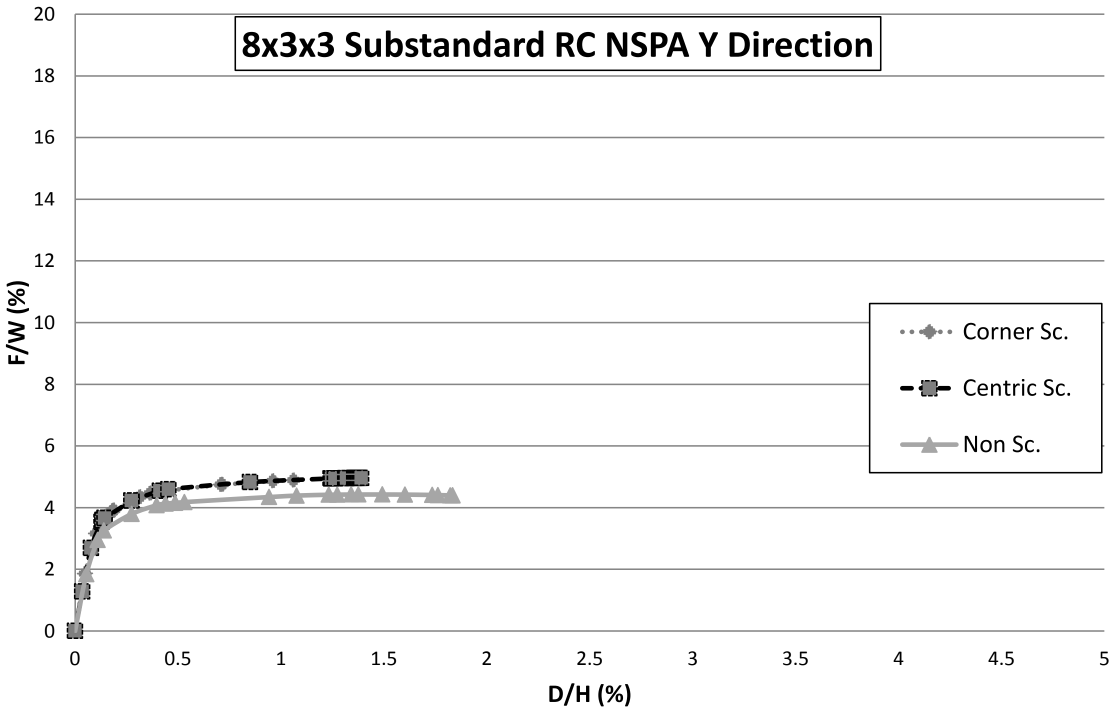


Figure 5.12 *8x3x3 Substandard RC NSPA Y Direction*

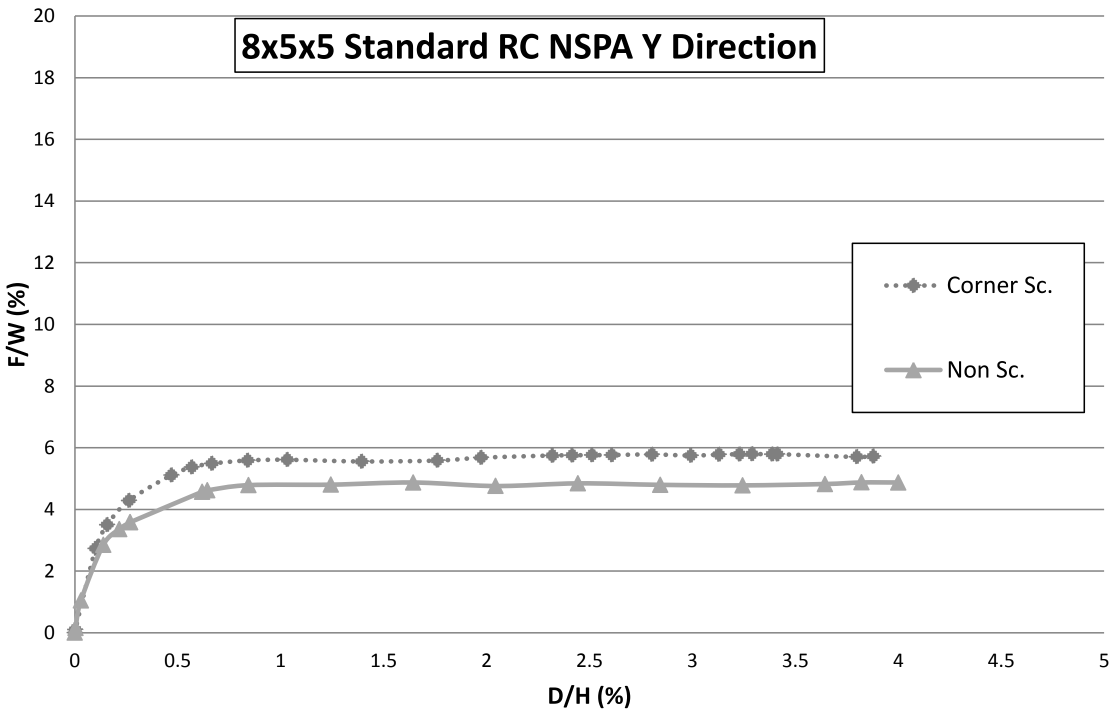


Figure 5.13 *8x5x5 Standard RC NSPA Y Direction*

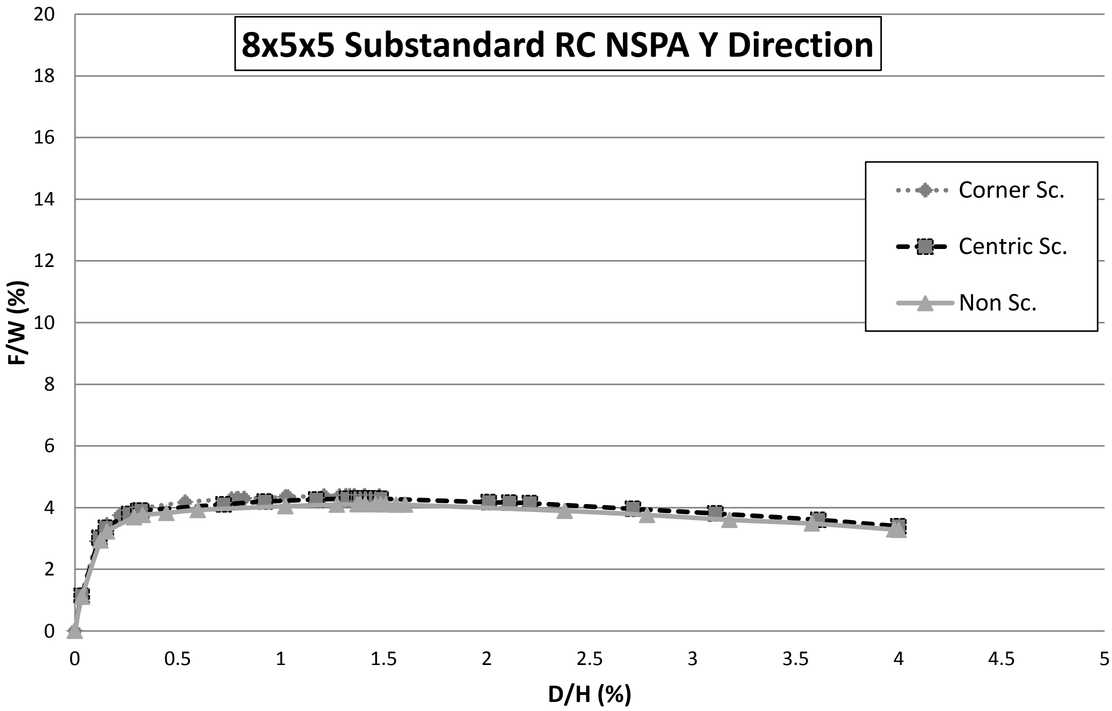


Figure 5.14 *8x5x5 Substandard RC NSPA Y Direction*

By evaluating the Figures 5.3 to 5.14 the following statements can be made;

- Shorter buildings have higher F/W ratio.
- Inclusion of staircases increases the base shear demand of the buildings. However, this may cause staircased buildings to receive more damage and to have lower overall ductility level.
- Substandard buildings have lower D/H ratio due to their lower overall ductility level.
- Increasing number of spans decreased the F/W ratio.
- Changing the position of the staircases did not make a considerable amount of difference.

6. NONLINEAR TIME-HISTORY ANALYSIS AND RESULTS

6.1. Definition

In order to investigate seismic response of the structures, time-history analysis can be employed. In this method, previously recorded or synthesized ground motions are used. Direct integration time-history analysis is chosen as analysis type which is a nonlinear dynamic analysis and it fully integrates equilibrium equations of motion (Equation (6.1)) step by step in every given time interval as a system is subjected to dynamic loading.

$$M\ddot{u}(t) + C\dot{u}(t) + Ku(t) = F(t) \tag{6.1}$$

Where M denotes mass, $\ddot{u}(t)$ denotes the acceleration over time, C denotes damping, $\dot{u}(t)$ denotes the velocity over time, K denotes stiffness, $u(t)$ denotes the displacement over time, $F(t)$ denotes change in force over time (Chopra, 2012).

6.2. TEC 2007 Provisions for Nonlinear Time-History Analysis, Earthquake Selection and Scaling the Earthquakes

According to TEC 2007, in the case where nonlinear time-history analysis is performed, if three ground motions are used the maximum of the results, and if at least seven ground motions are used the mean values of the results shall be considered.

In this study, previously recorded seven earthquake ground motions with their two horizontal components have been selected for bi-axial non-linear time history analysis (NLTHA). These ground motion records were obtained from the study carried out by Aşıkoğlu and Avşar (2017). General characteristics of the selected earthquake ground motions are shown in Table 6.1.

Table 6.1 General informations about the selected earthquakes

Name	Year	Station	M_w	R (km)	Component	PGA (g)	PGV (cm/s)	PGD (cm)	$V_{s\,(m/s)}$	Scale Factor
Kocaeli	1999	Düzce	7.5	13.6	180	0.312	58.9	44.2	276.0	1
					270	0.358	46.4	17.6		
Morgan Hill	1984	Gilroy #3	6.2	13.0	0	0.194	11.2	2.3	349.9	1.3
					90	0.200	12.7	3.4		
Düzce	1999	Bolu	7.1	12.0	0	0.728	56.4	23.1	326.0	1.3
					90	0.822	62.1	13.6		
Landers	1992	North Palm Springs	7.3	26.8	0	0.136	11.0	5.0	345.4	1
					90	0.134	14.5	5.6		
Imperial Valley	1979	Westmorland Fire	6.5	15.2	90	0.074	21.3	16.6	193.7	1
					180	0.110	21.9	10.0		
Superstition	1987	Parachute Test Site	6.5	0.9	225	0.455	112.1	54.0	348.7	1.3
					315	0.377	43.9	15.3		
Kobe	1995	Shin Osaka	6.9	19.1	0	0.243	37.8	8.6	256.0	1
					90	0.212	27.9	7.6		

Scaling was done such that the average SRSS (square roots of sum of squares) of spectral acceleration of the selected earthquakes should be greater than 90% of the design spectral acceleration values in the related response spectrum in given period interval.

Period interval was chosen according to the first mode of all models. The model with the smallest period is 3x3x3_st_cent with a value of 0.482 seconds while 8x5x5_sub_non has the biggest period with a value of 1.440 seconds.

Design spectrum has been created according to the data from the Equation (6.2) (TEC 2007) and Table 6.2. The results were plotted in Figure 6.1. Response spectrum and the SRSS of the average spectral acceleration of the scaled earthquakes are shown in Figure 6.2.

$$S(T) = 1 + 1.5\frac{T}{T_a} \qquad (0 \le T \le T_a)$$

$$S(T) = 2.5 \qquad (T_a \le T \le T_b) \qquad \textbf{(6.2)}$$

$$S(T) = 2.5\left(\frac{T_B}{T}\right)^{0.8} \qquad (T_B < T)$$

Table 6.2 *Spectrum characteristic periods for local site classes according to TEC 2007*

Local Site Class according to Table 6.2	T_A *(second)*	T_B *(second)*
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

Where $S(T)$ denotes the spectrum coefficient depending on the local site conditions and the building natural period, T. Spectrum Characteristic Periods, T_A and T_B, appearing in Equation (6.2) are specified in Table 6.2, depending on local site classes defined in Table 6.2 of Chapter 6 of TEC 2007. In this study, Z3 is chosen as the local site class for all buildings. Seismic zone is taken as "1st class seismic zone".

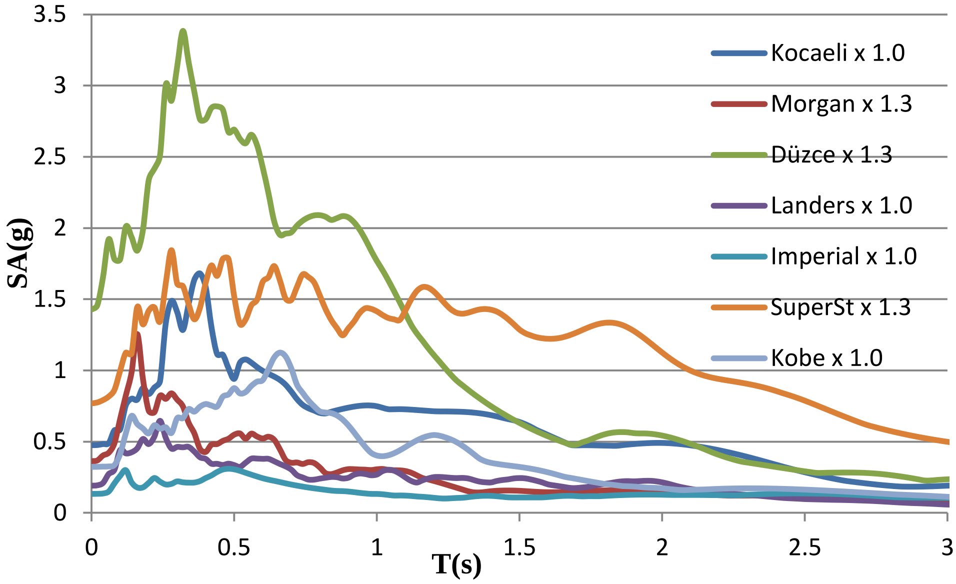


Figure 6.1 *SRSS of the response spectrums of the scaled earthquakes*

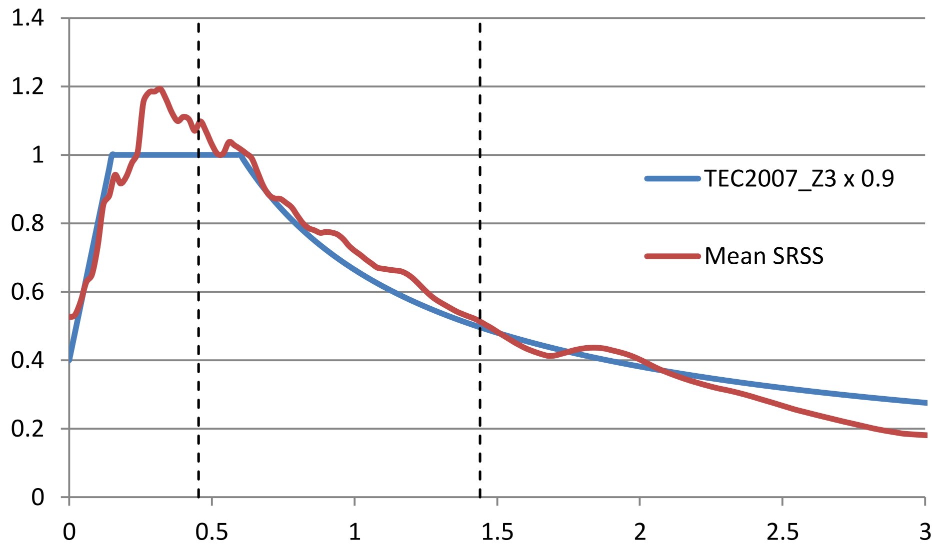


Figure 6.2 *Scaling of the earthquakes according to given conditions*

Non-scaled acceleration-time series of the selected earthquakes are shown in Figure 6.3 to 6.9.

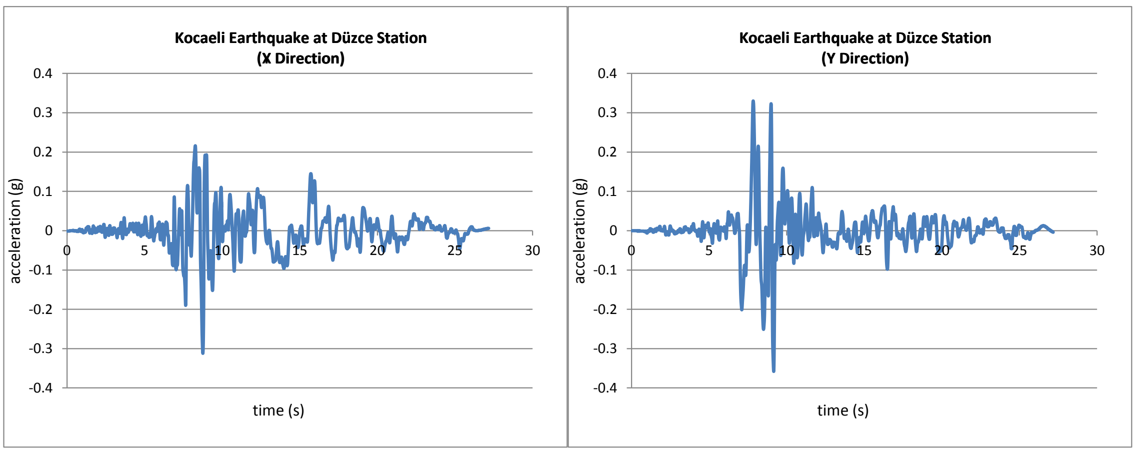


Figure 6.3 *Acceleration-time series of Kocaeli Earthquake*

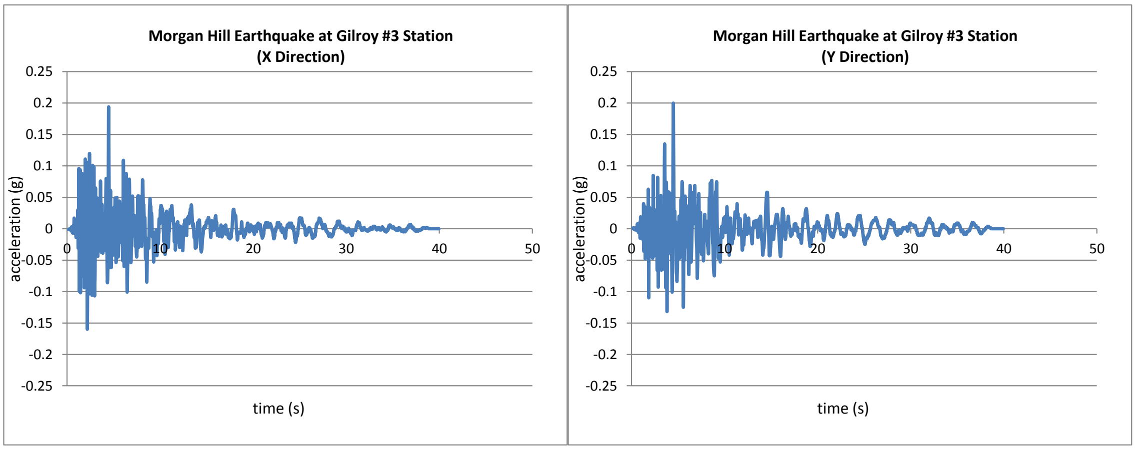


Figure 6.4 *Acceleration-time series of Morgan Hill Earthquake*

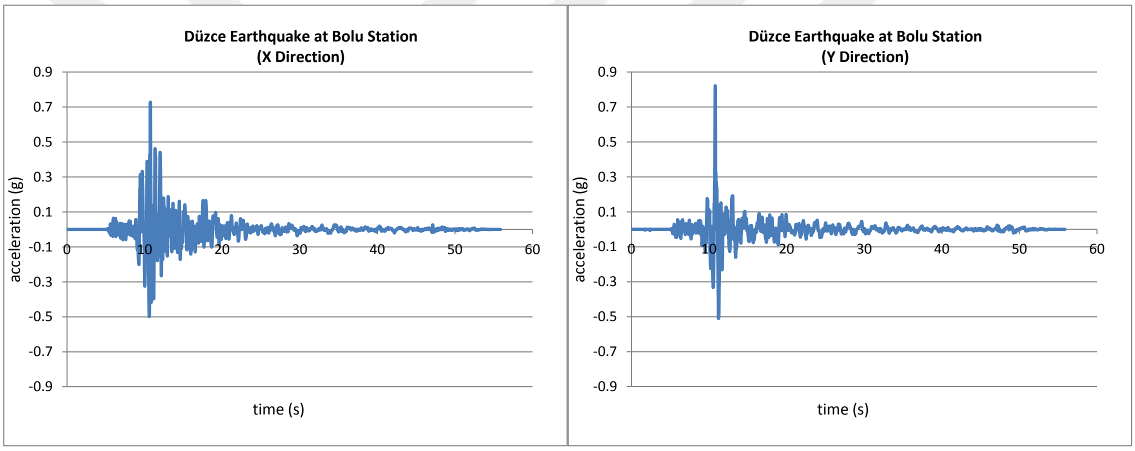


Figure 6.5 *Acceleration-time series of Düzce Earthquake*

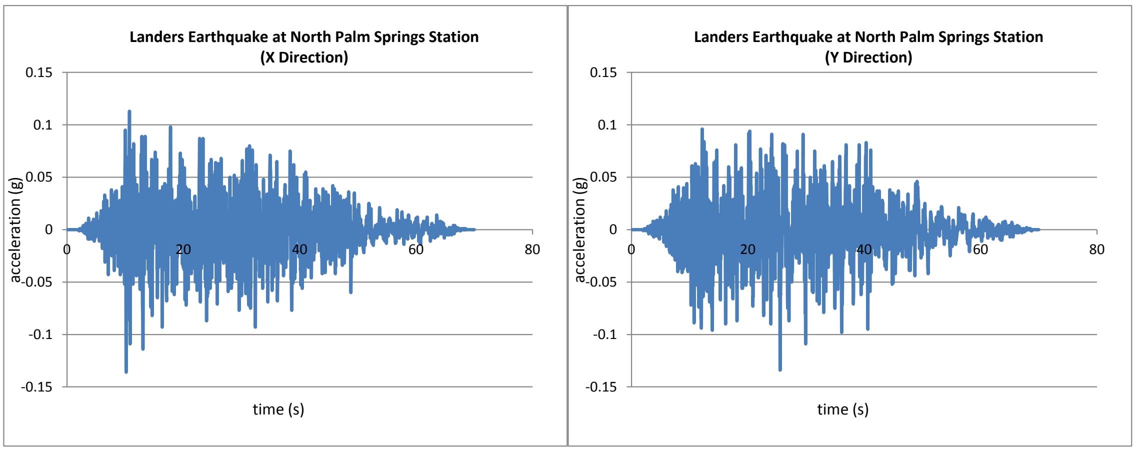


Figure 6.6 *Acceleration-time series of Landers Earthquake*

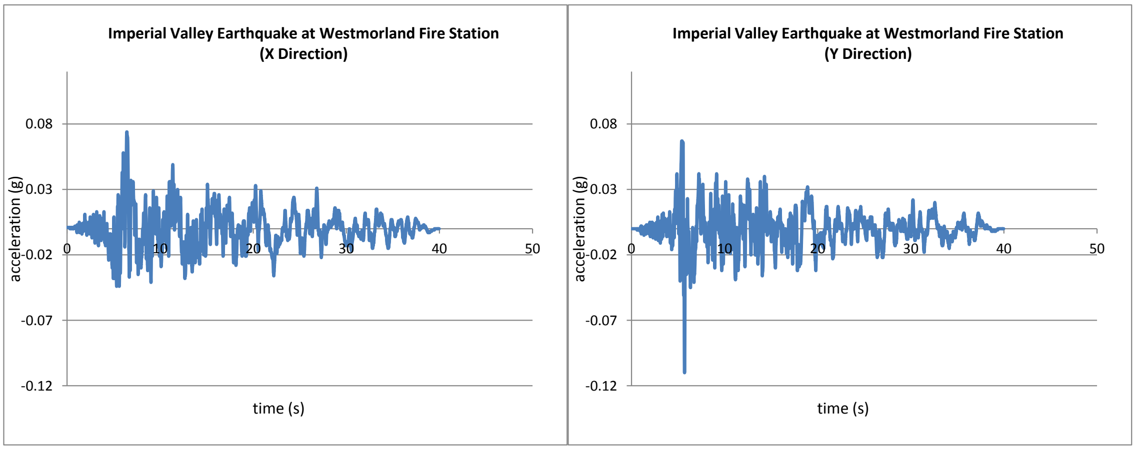


Figure 6.7 *Acceleration-time series of Imperial Valley Earthquake*

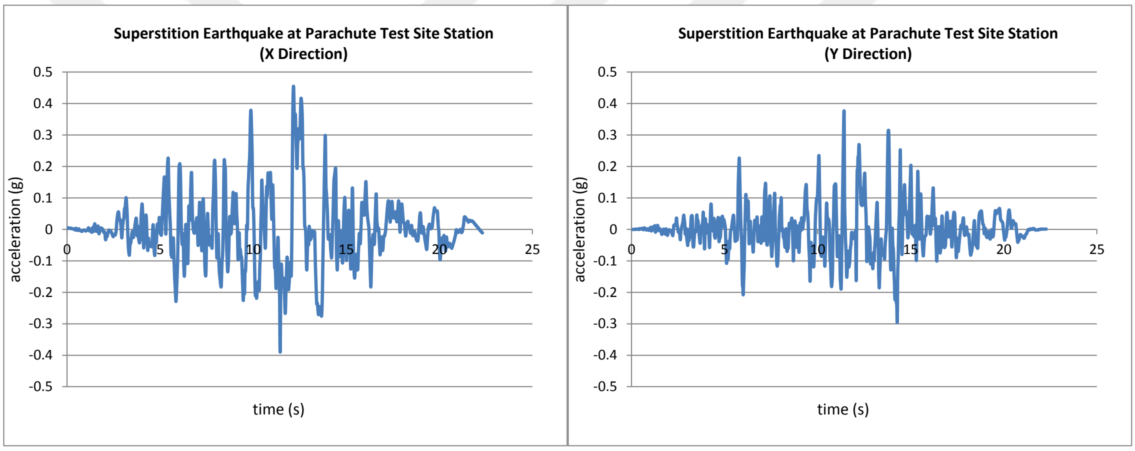


Figure 6.8 *Acceleration-time series of Superstition Earthquake*

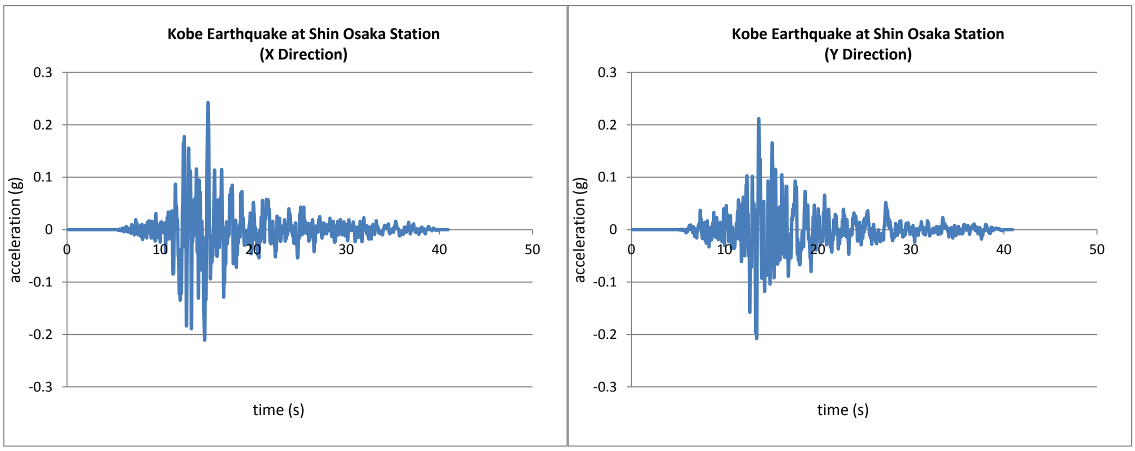


Figure 6.9 *Acceleration-time series of Kobe Earthquake*

6.3. Defining Time-History Functions and Load Cases to SAP2000

As an example, Bolu Earthquake in X direction has been defined in SAP2000 from "Time-History Function Definition" menu. In order to define a time-history function, time interval data should be known in advance. For Bolu Earthquake, time interval value is 0.01 seconds as shown in Figure 6.10.

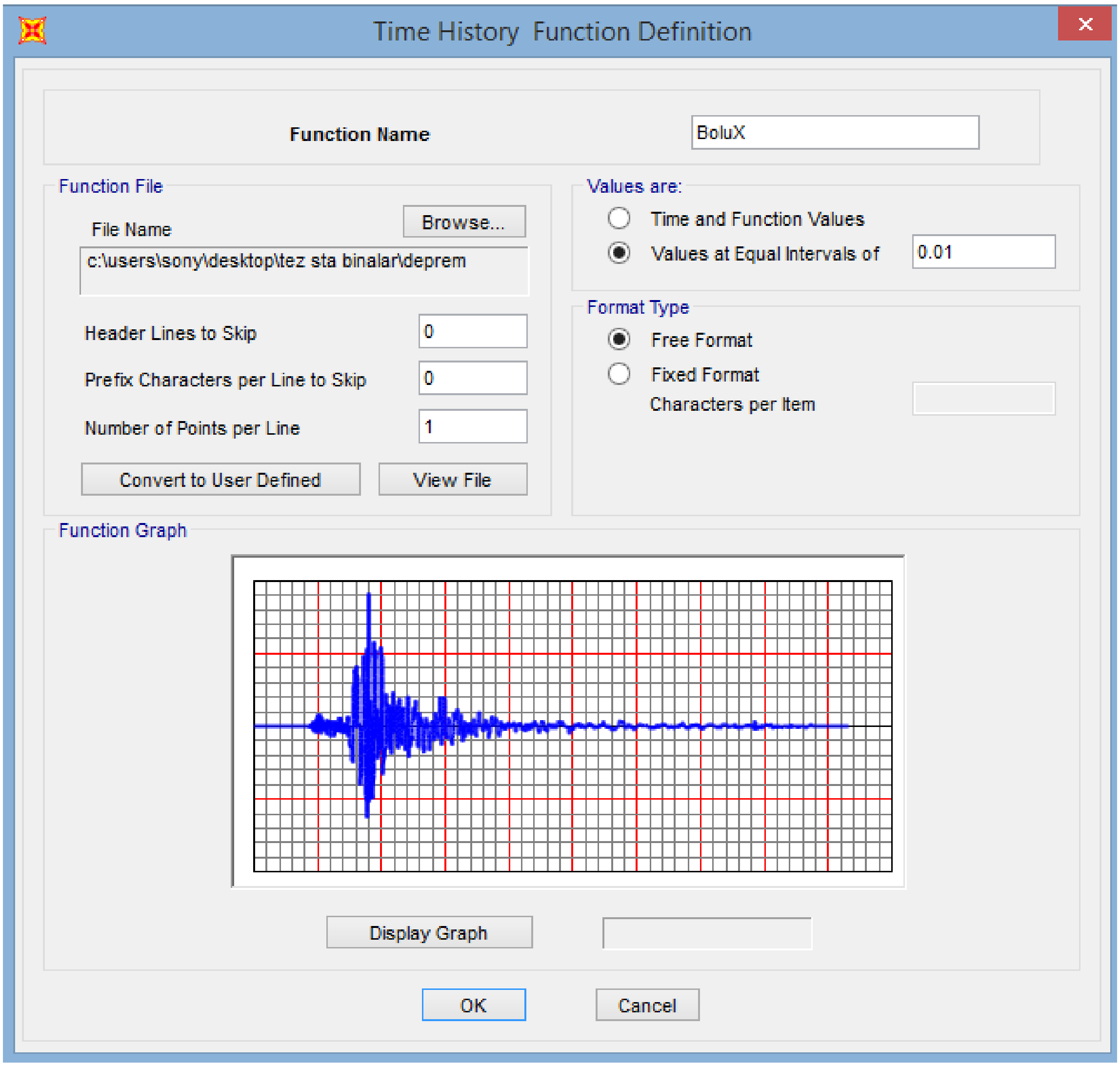


Figure 6.10 *Time-history function data of Bolu Earthquake in X direction*

After defining time-history functions, a time-history load case can be defined. In order to define a time-history load case, time interval, scale factor and number of output time steps should be known in advance as shown in Table 6.3.

Table 6.3 *Earthquake data required for defining time-history load cases*

Earthquake	Scale Factor	Interval (s)	Step
Kocaeli	9.81	0.005	5435
Landers	9.81	0.005	13998
Imperial Valley	9.81	0.005	7995
Kobe	9.81	0.01	4094
Morgan Hill	12.753	0.005	7994
Düzce	12.753	0.01	5588
SuperSt	12.753	0.01	2233

According to the data given above in Table 6.3, a nonlinear time-history load case has been defined for all of the selected earthquakes. In Figure 6.11, parameters of time-history load case for Morgan Hill Earthquake are shown.

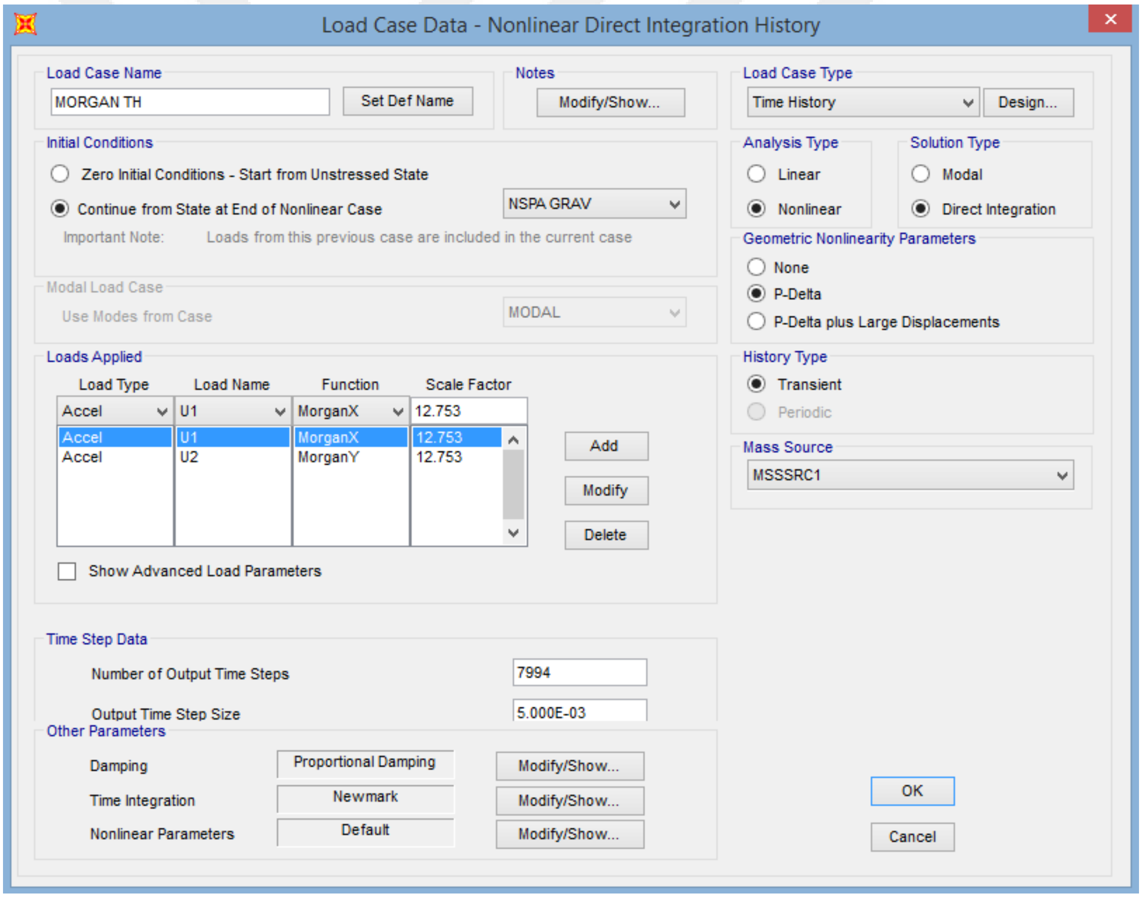


Figure 6.11 *Time-history load case data of Morgan Hill Earthquake*

Note that all earthquake records were defined in SAP2000 in terms of g's. Since in this study the units were chosen as meters and seconds, all earthquake records needed to be multiplied by $9.81 m/s^2$ in order to convert the data series in terms of m/s^2. Additionally, the earthquake records need to be multiplied by the scale factors given in

Table 6.1. As a result, for an earthquake with a scale factor of 1.3 should be multiplied by 12.753 which is the multiplication of 9.81 and 1.3.

6.4. Time-History Analysis Results

6.4.1. Peak floor accelerations

The peak floor acceleration is an important engineering demand parameter for the comfort of the residents of the buildings. Also, it is a significant parameter in the determination of seismic force imposed on the non-structural components on the floors. A building with an infinite amount of stiffness is expected to have the same amount of peak floor acceleration at its all floors. Yet, a building with a certain amount of stiffness is expected to experience different amount of acceleration at its higher floors with respect to its ground floor. If this difference is greater in a building when compared to others, then it can be said that, the subject building is less stiff.

The plotted results were obtained from SRSS of both X and Y directions and are presented in Figures between 6.12 and 6.23. Red line represents the average of the peak floor accelerations at each floor.

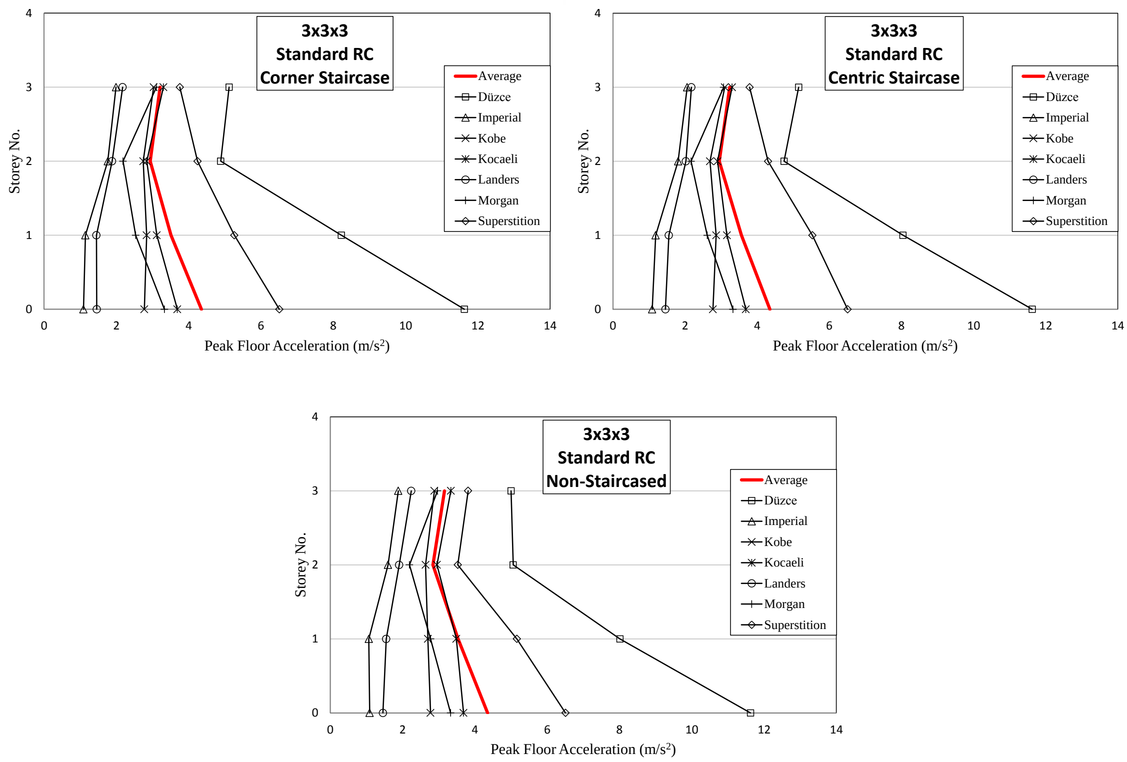


Figure 6.12 *Peak floor accelerations of 3x3x3_st models*

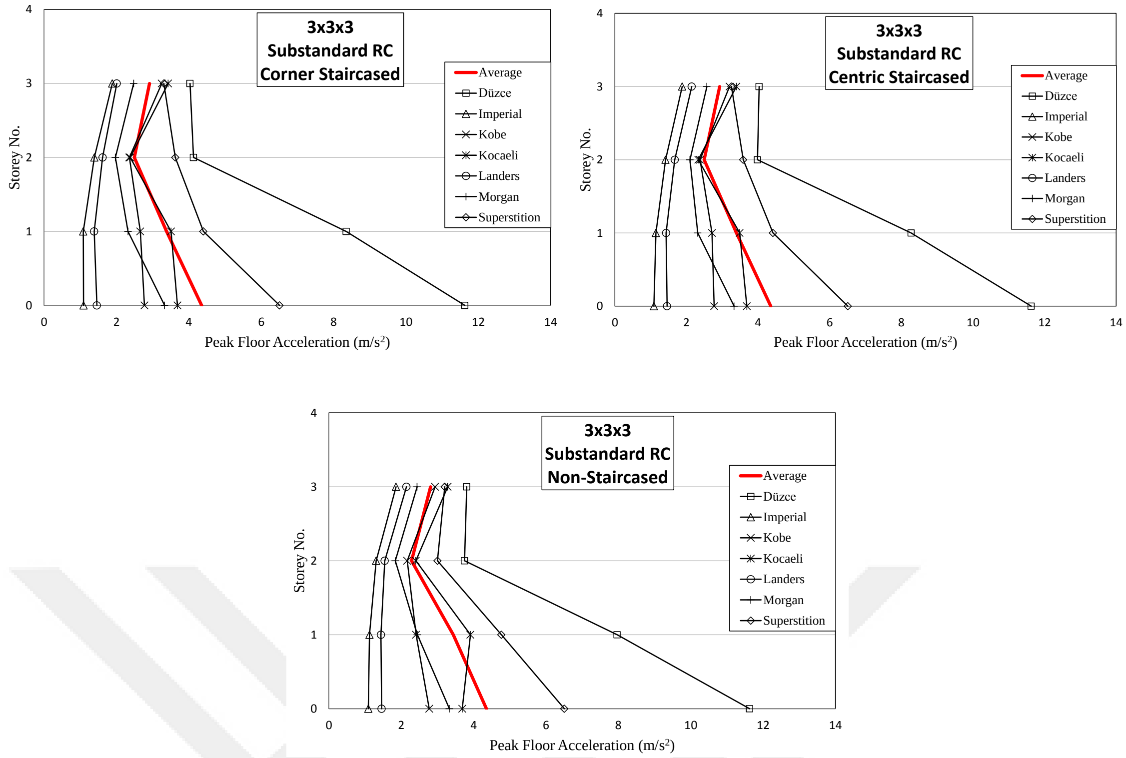


Figure 6.13 *Peak floor accelerations of 3x3x3_sub models*

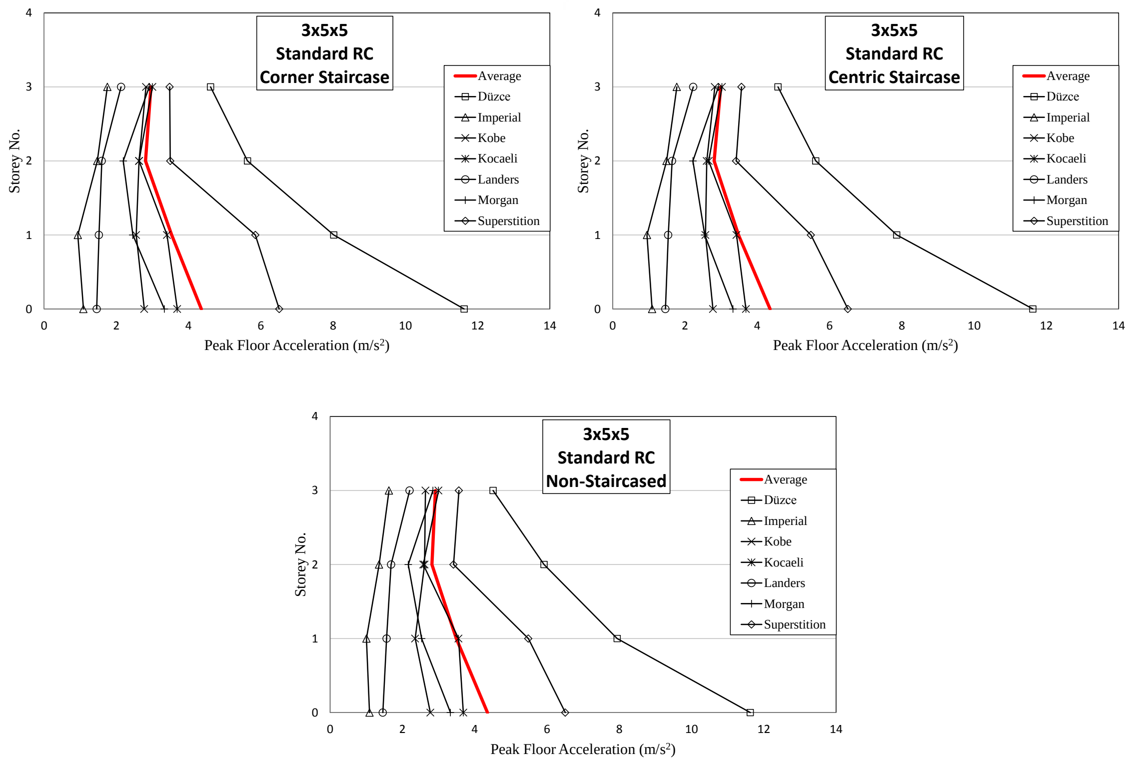


Figure 6.14 *Peak floor accelerations of 3x5x5_st models*

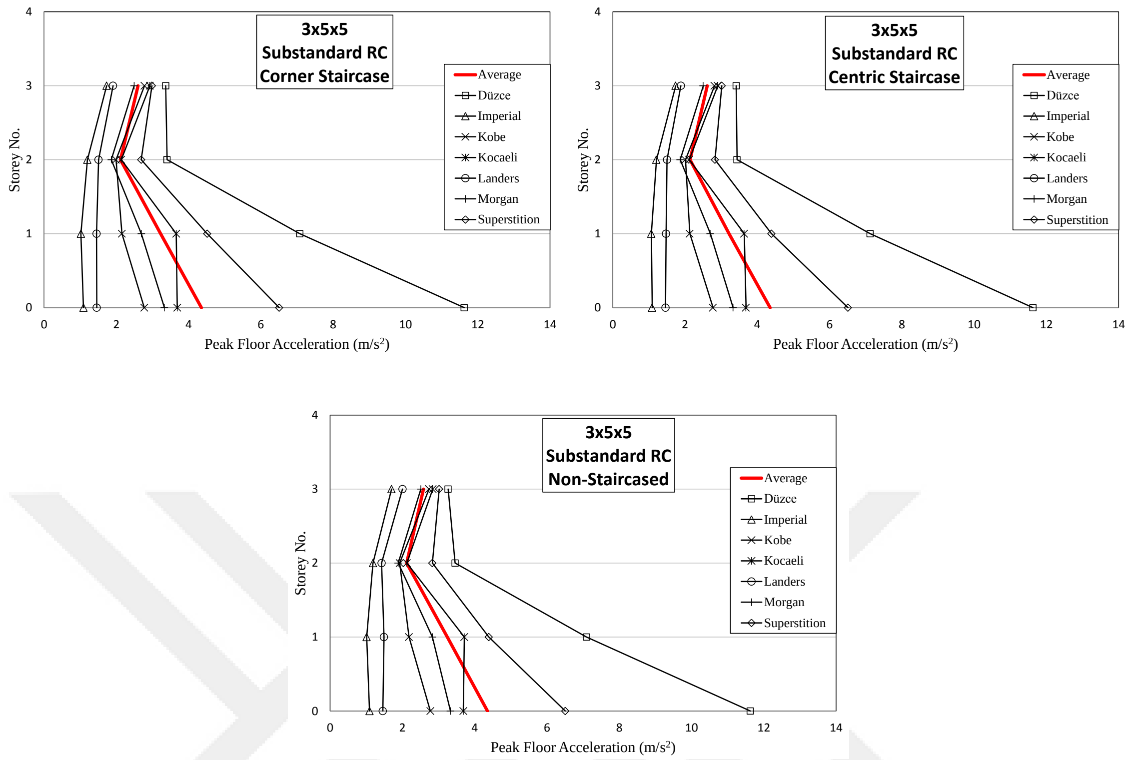


Figure 6.15 *Peak floor accelerations of 3x5x5_sub models*

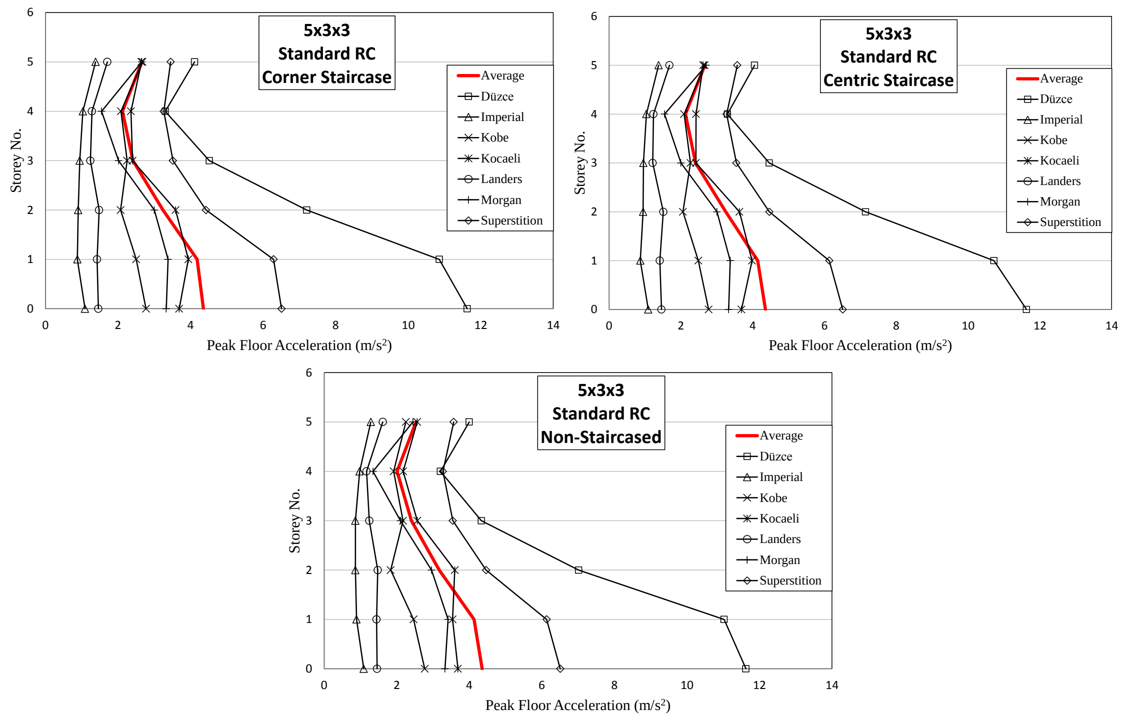


Figure 6.16 *Peak floor accelerations of 5x3x3_st models*

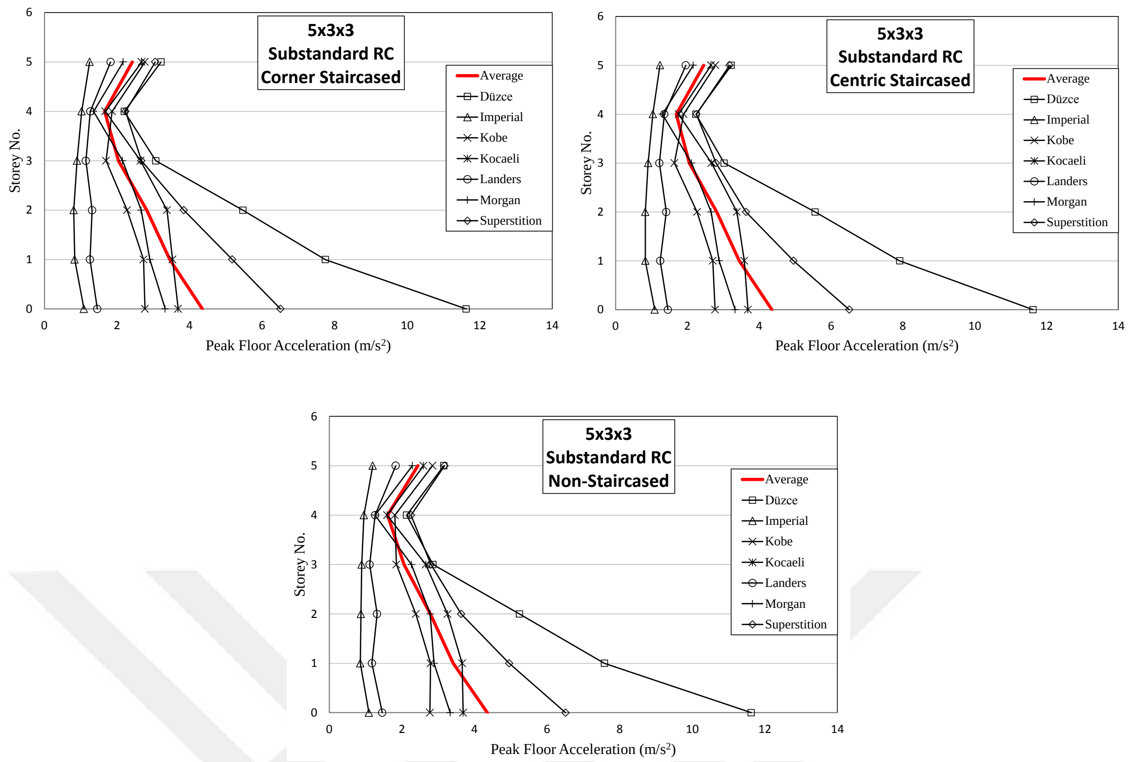


Figure 6.17 *Peak floor accelerations of 5x3x3_sub models*

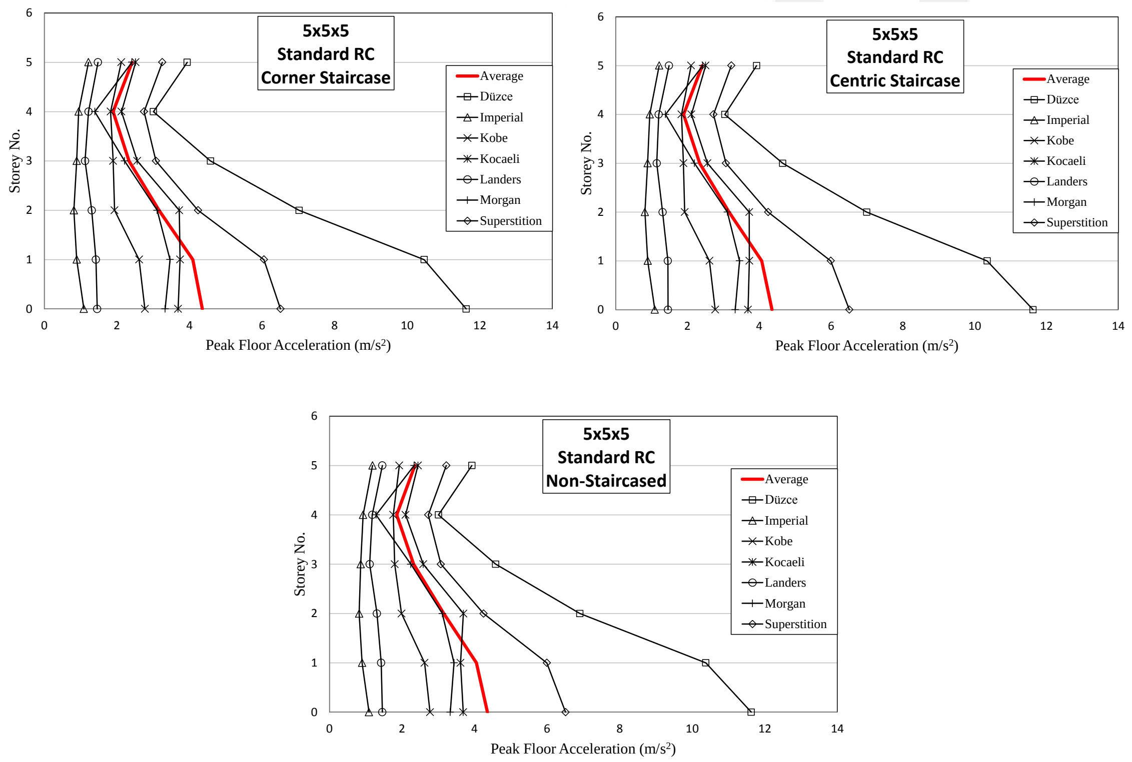


Figure 6.18 *Peak floor accelerations of 5x5x5_st models*

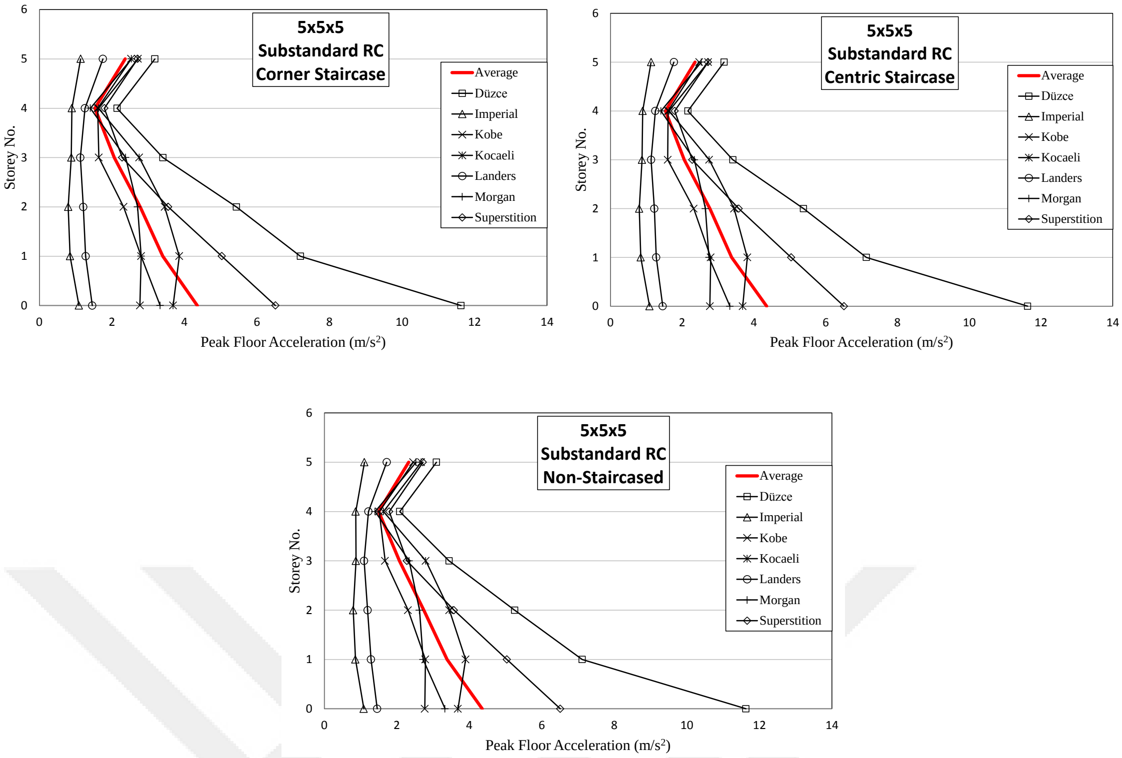


Figure 6.19 *Peak floor accelerations of 5x5x5_sub models*

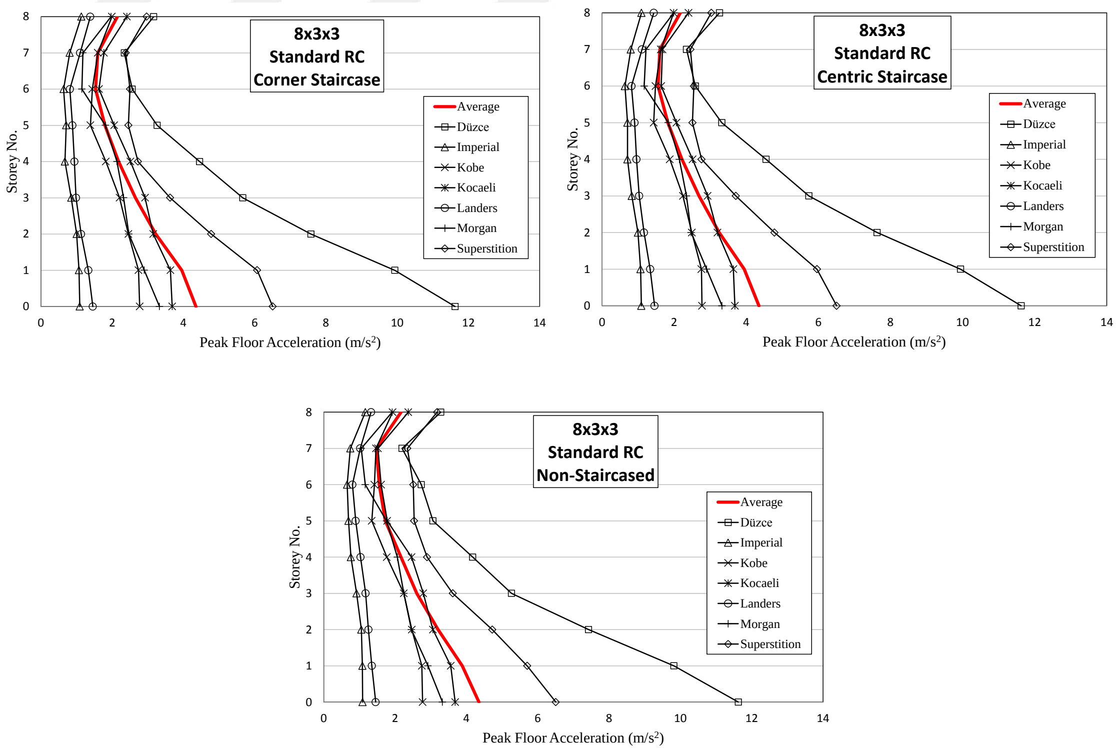


Figure 6.20 *Peak floor accelerations of 8x3x3_st models*

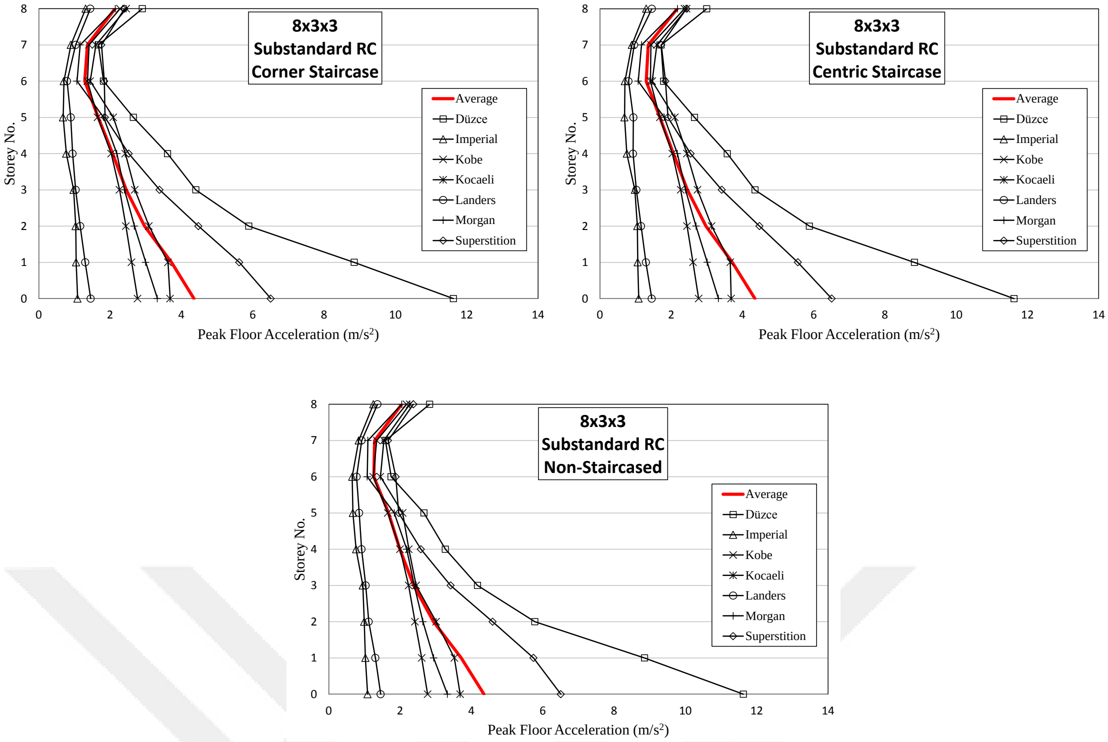


Figure 6.21 *Peak floor accelerations of 8x3x3_sub models*

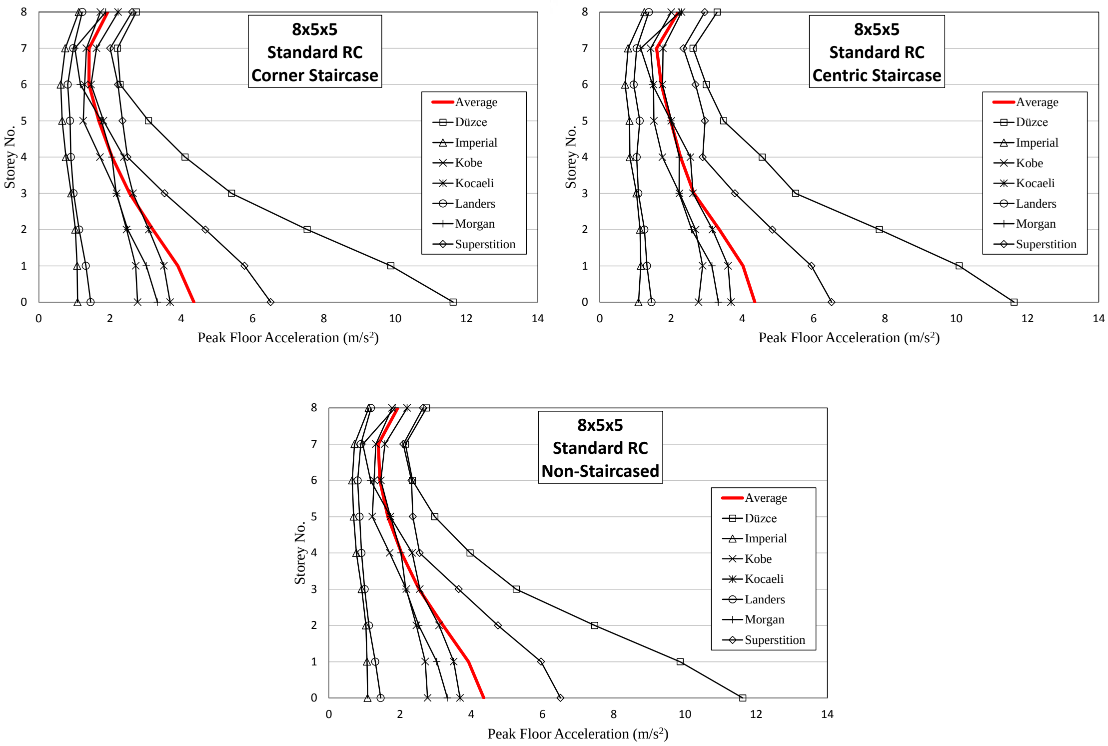


Figure 6.22 *Peak floor accelerations of 8x5x5_st models*

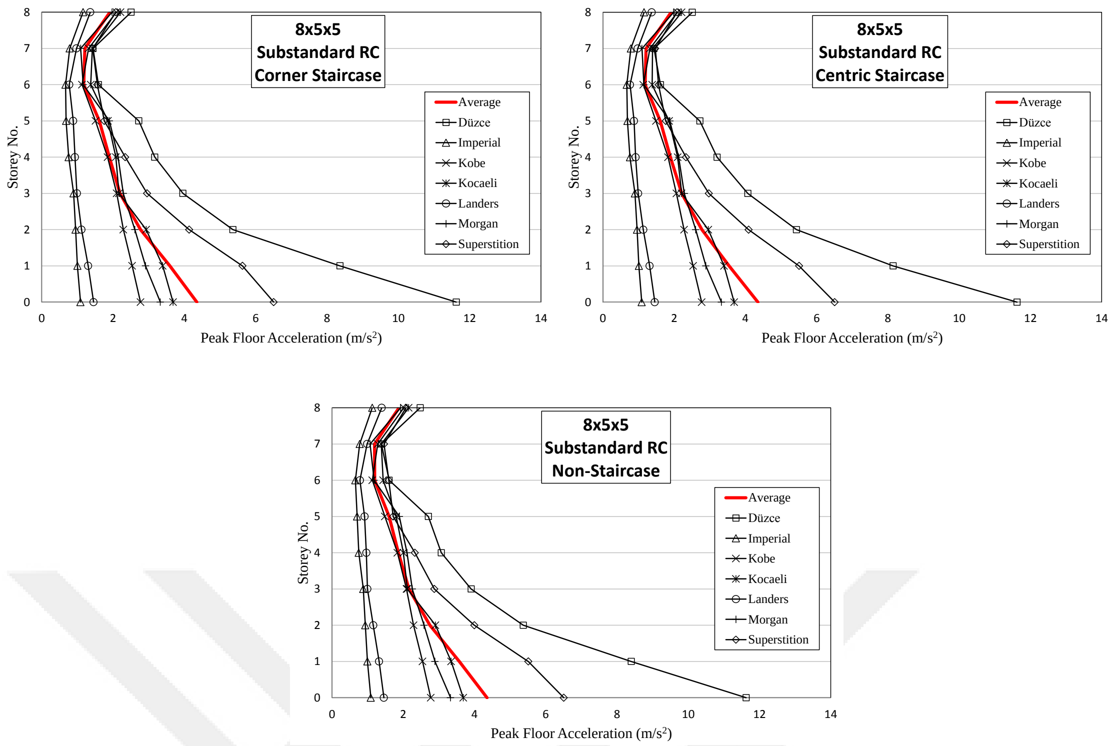


Figure 6.23 *Peak floor accelerations of 8x5x5_sub models*

In order to examine the effect of investigated building parameters on the peak floor acceleration, the average peak floor acceleration results of each 3, 5 and 8 storey buildings were compared and presented in Figure 6.24;

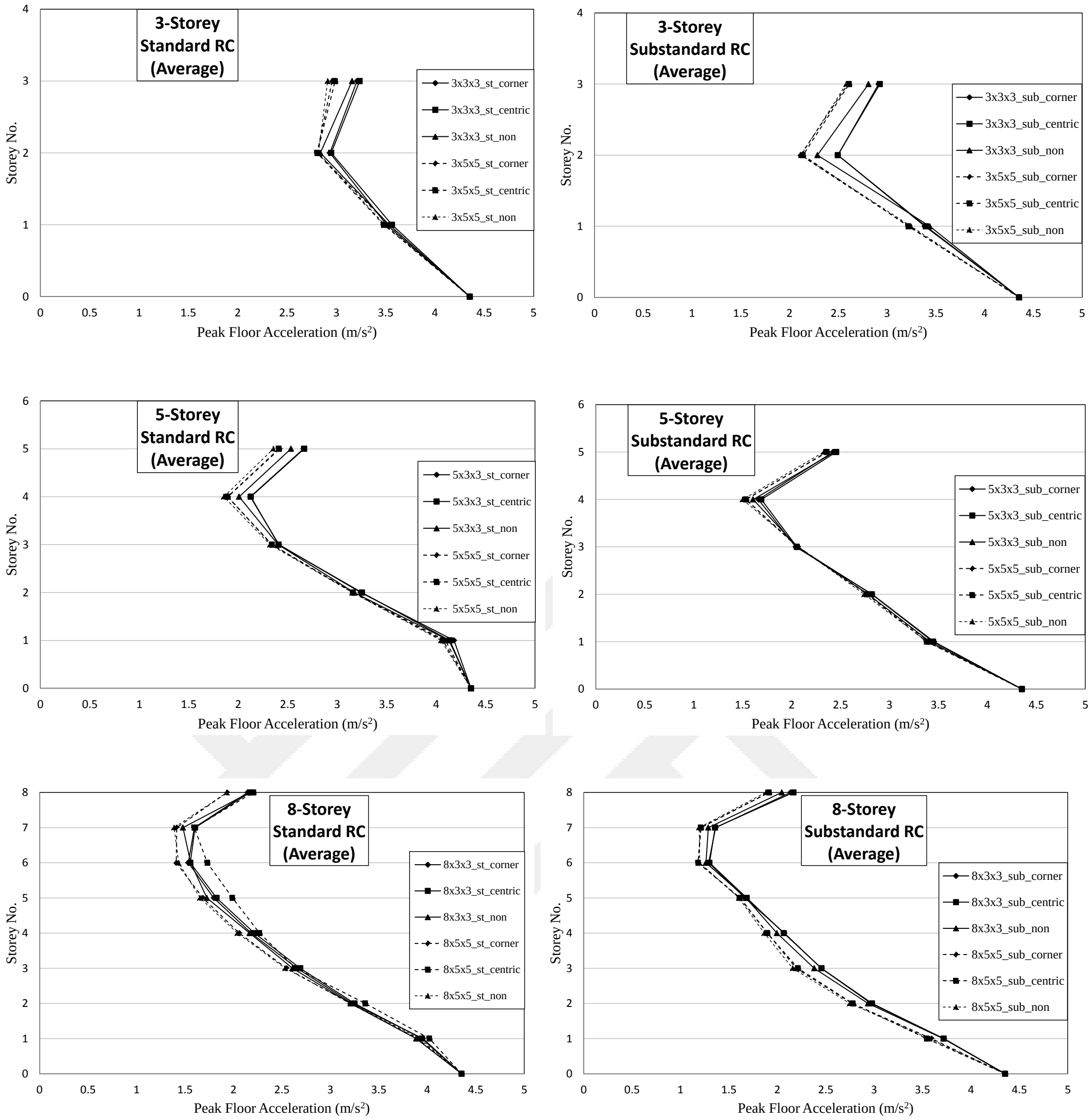


Figure 6.24 *Average peak floor accelerations of all models*

According to the peak floor acceleration graphs given above, the following statements can be made;

- Superstition Hills and Düzce earthquakes have imposed more seismic demands on the buildings. This outcome can be attributed to their high energy content. Their relatively higher PGV values are the indicators for their high energy content.
- The variation in the peak floor acceleration is more pronounced for the taller buildings.
- The change in peak floor acceleration from bottom to the top of the buildings is more for the substandard buildings in an average sense.

- The effect of the staircase and its position in the building did not influence the average results considerably in terms of the peak floor acceleration.
- The difference between minimum and maximum peak floor acceleration is greater in substandard models which indicates that substandard buildings are less stiff and they are exposed to more seismic damage.

6.4.2. Inter-storey drift ratios

Inter-storey drift ratio (ISDR) is an important indicator for assessing the structural response. It is the ratio of the relative displacement between the successive stories with respect to the corresponding story height.

In order to limit the second order effects due to large displacements and to protect the secondary components in the building, modern seismic codes limit the max ISDR.

Inter-storey drift ratio can be found by dividing the relative drift of two consecutive floors to the storey height. In this study, ISDR has only been calculated for Y direction which is parallel to staircase flight running direction.

Note that for 3 storey models, the contribution of Superstition Earthquake to the average results was not included since they cause 3x3x3_sub_non, 3x5x5_sub_cor, 3x5x5_sub_cent and 3x5x5_sub_non to collapse and to have an infinite value of ISDR which affects average results and may provide misleading data.

Note that the red lines in the graphs represent the average peak interstorey drift ratio of the seven selected earthquakes.

ISDR graphs are presented in Figures between 6.25 and 6.37.

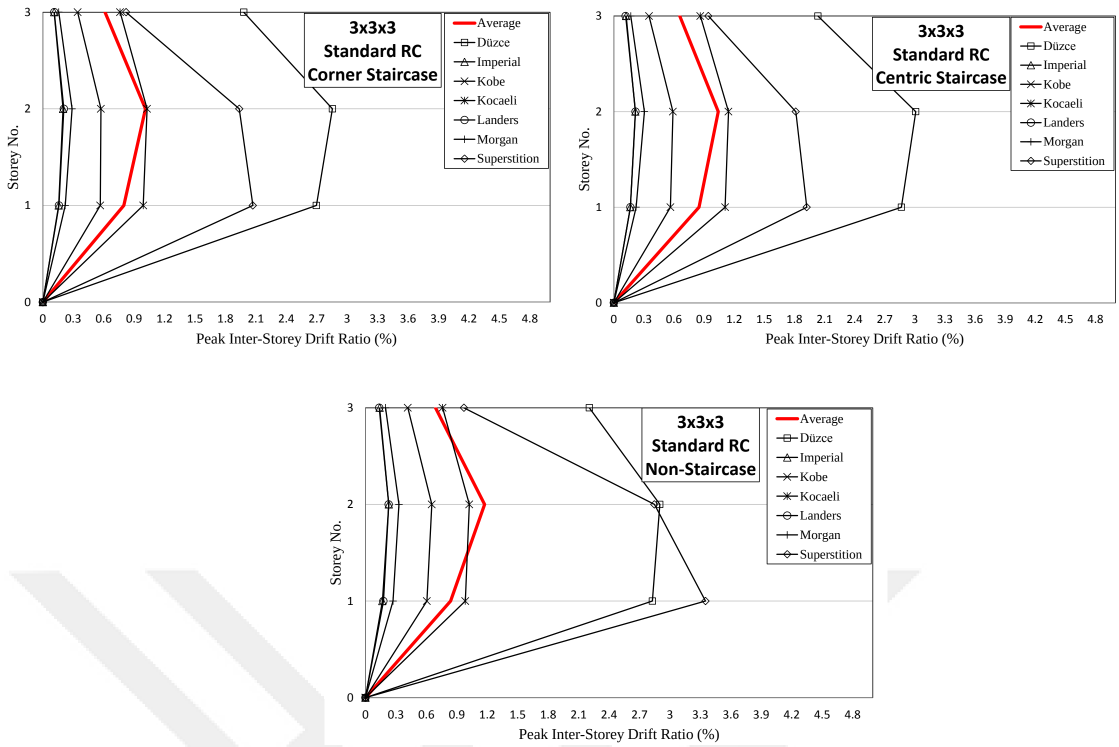


Figure 6.25 *Peak ISDR of 3x3x3_st models*

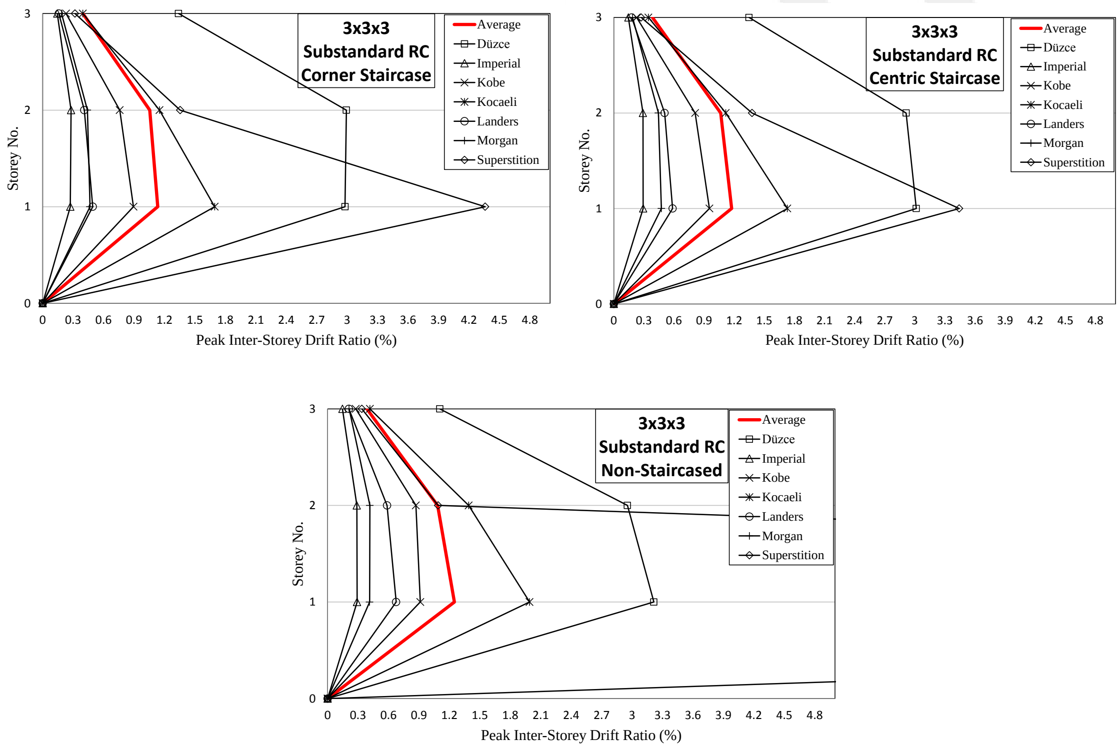


Figure 6.26 *Peak ISDR of 3x3x3_sub models*

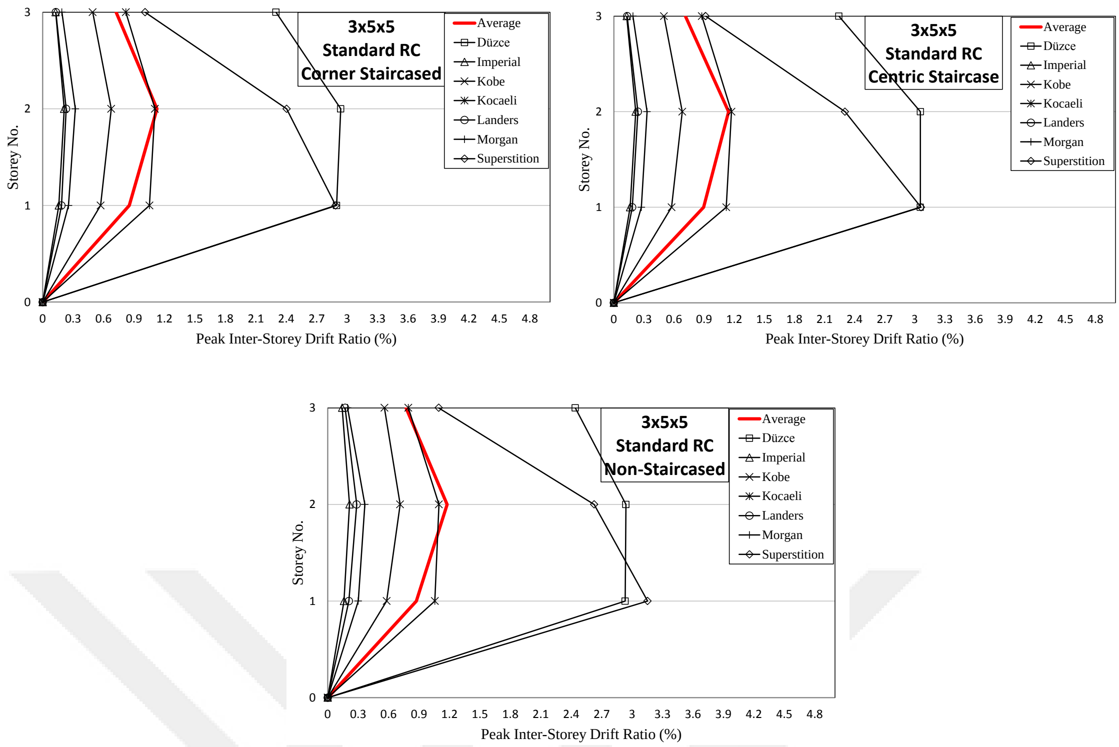


Figure 6.27 *Peak ISDR of 3x5x5_st models*

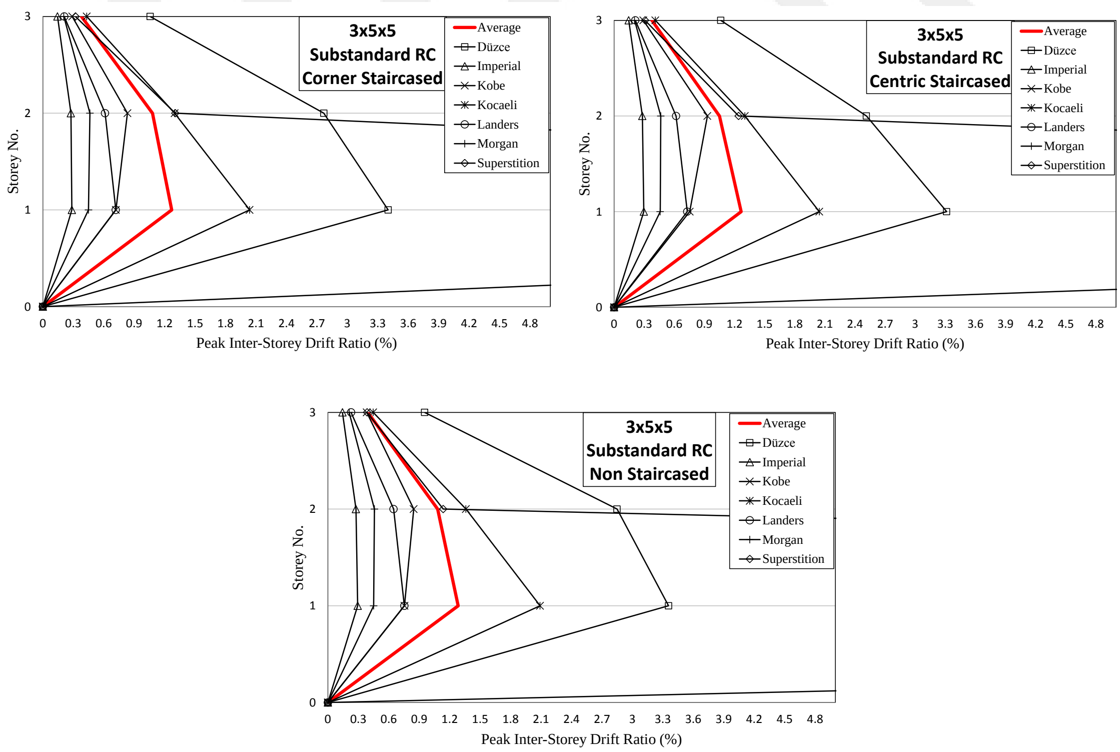


Figure 6.28 *Peak ISDR of 3x5x5_sub models*

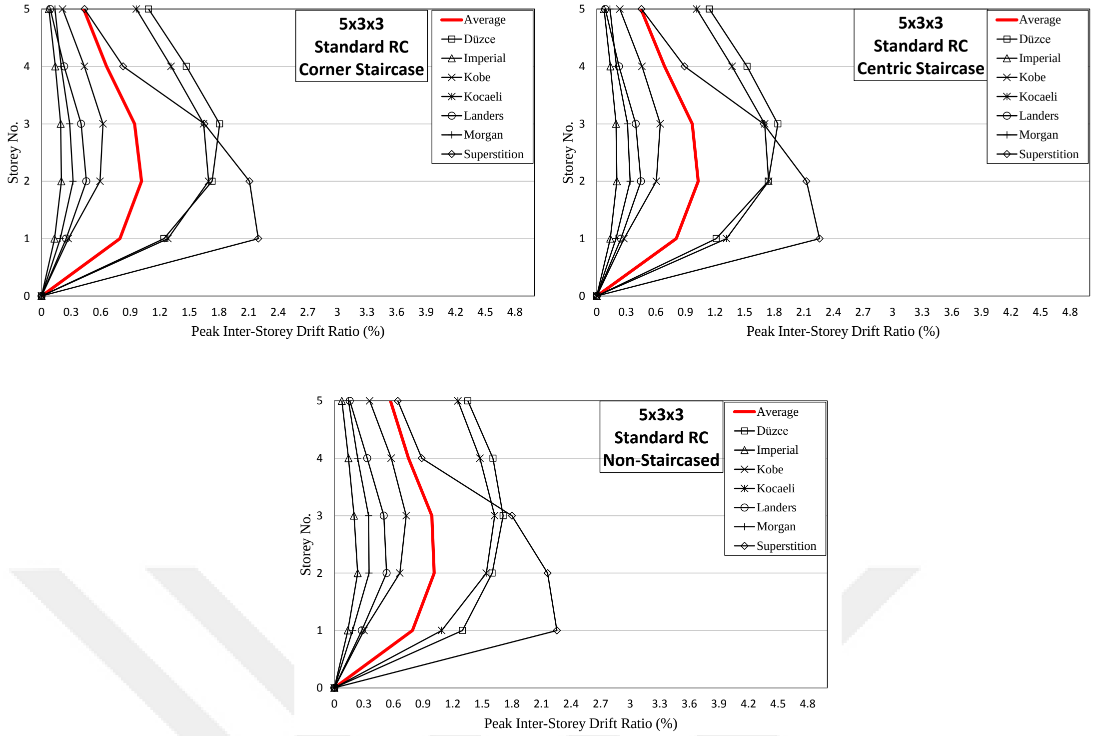


Figure 6.29 *Peak ISDR of 5x3x3_st models*

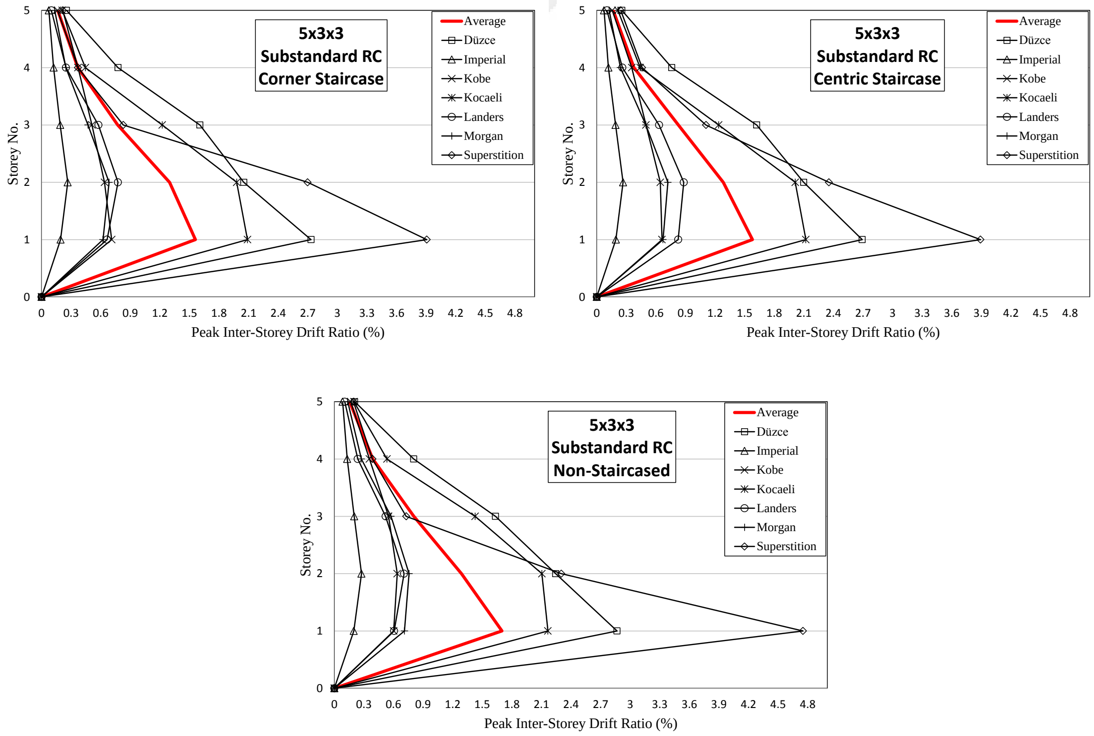


Figure 6.30 *Peak ISDR of 5x3x3_sub models*

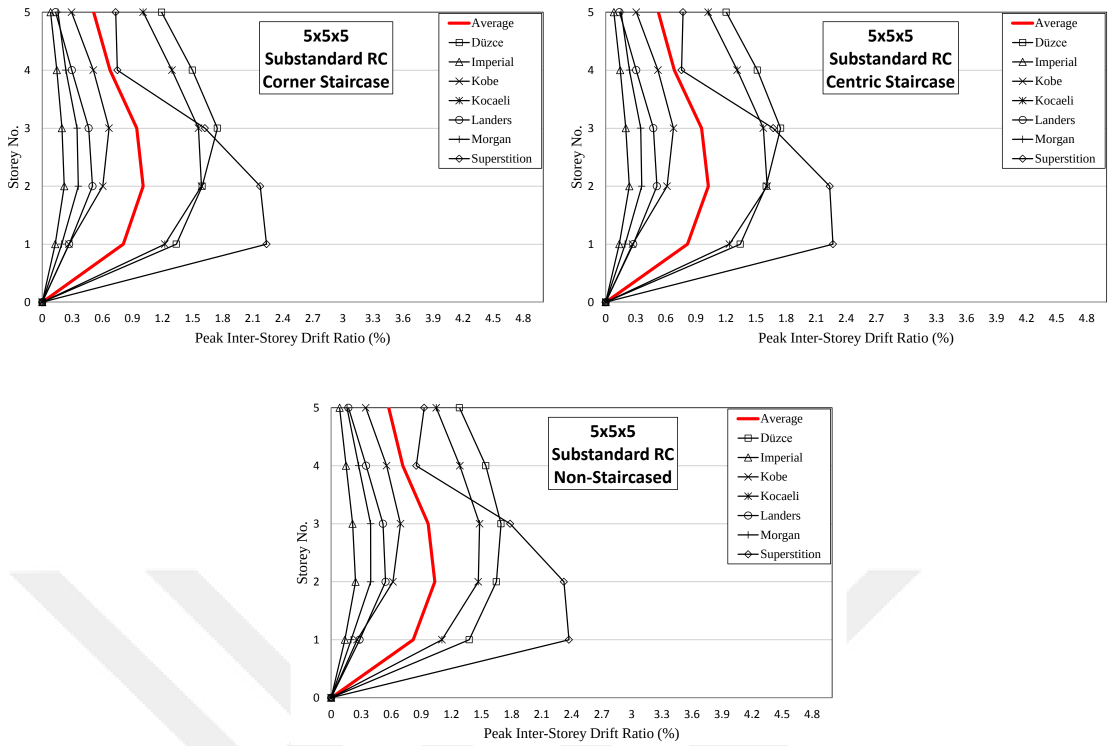


Figure 6.31 *Peak ISDR of 5x5x5_st models*

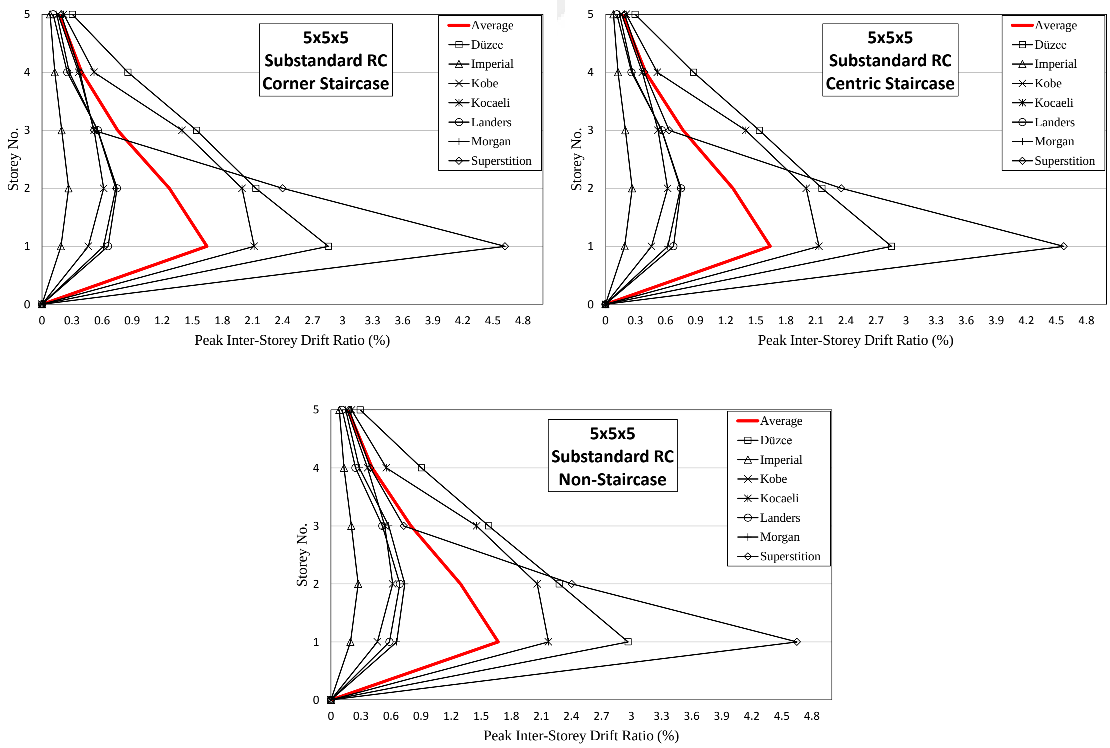


Figure 6.32 *Peak ISDR of 5x5x5_sub models*

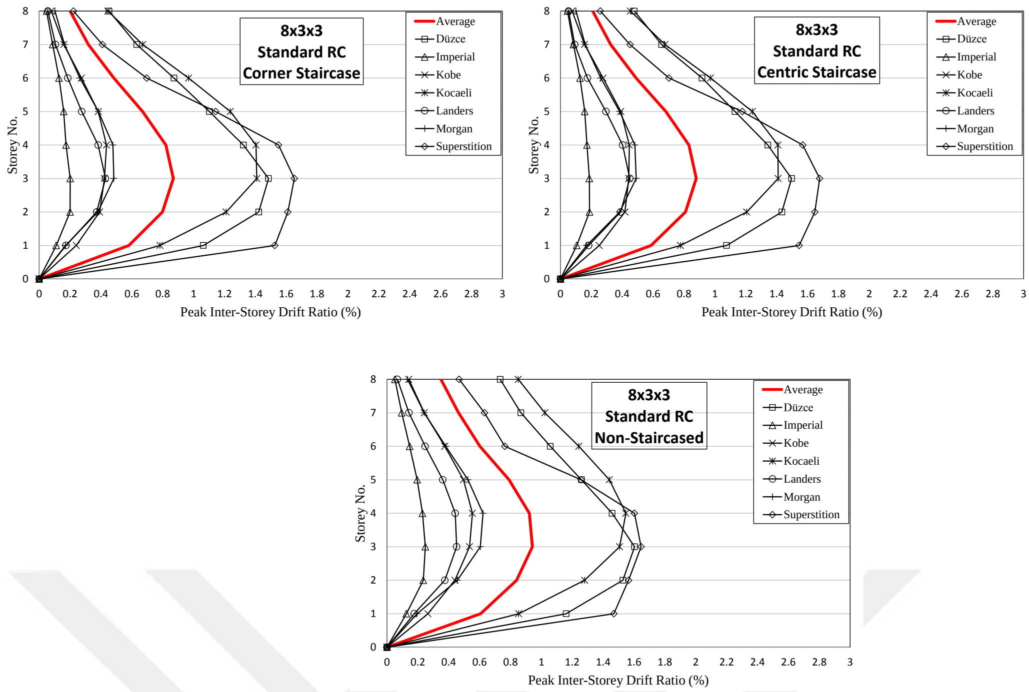


Figure 6.33 *Peak ISDR of 8x3x3_st models*

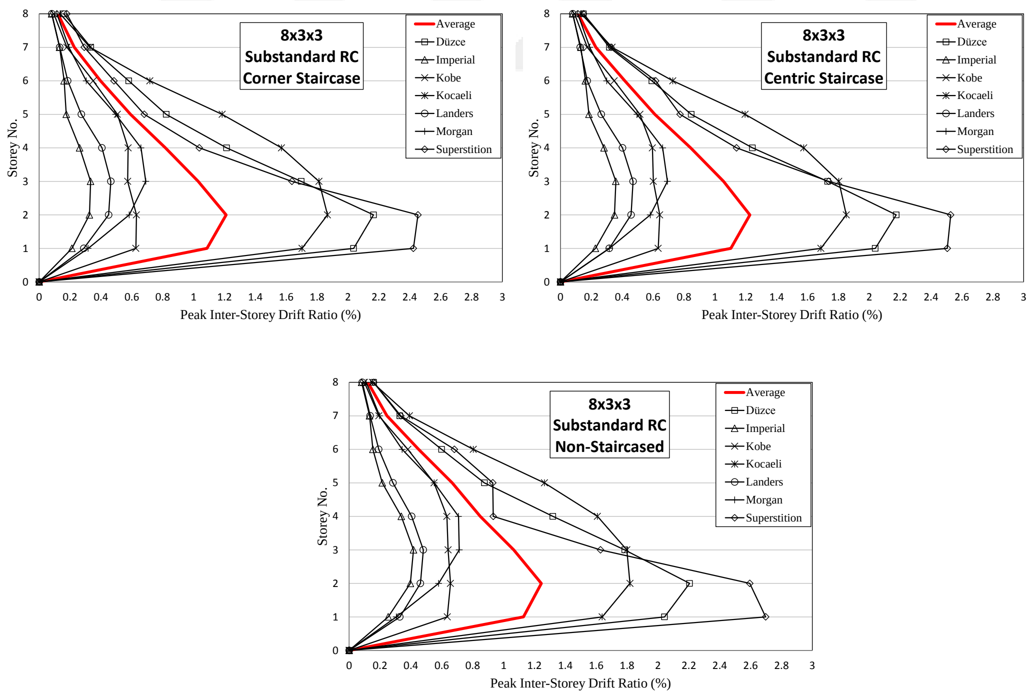


Figure 6.34 *Peak ISDR of 8x3x3_sub models*

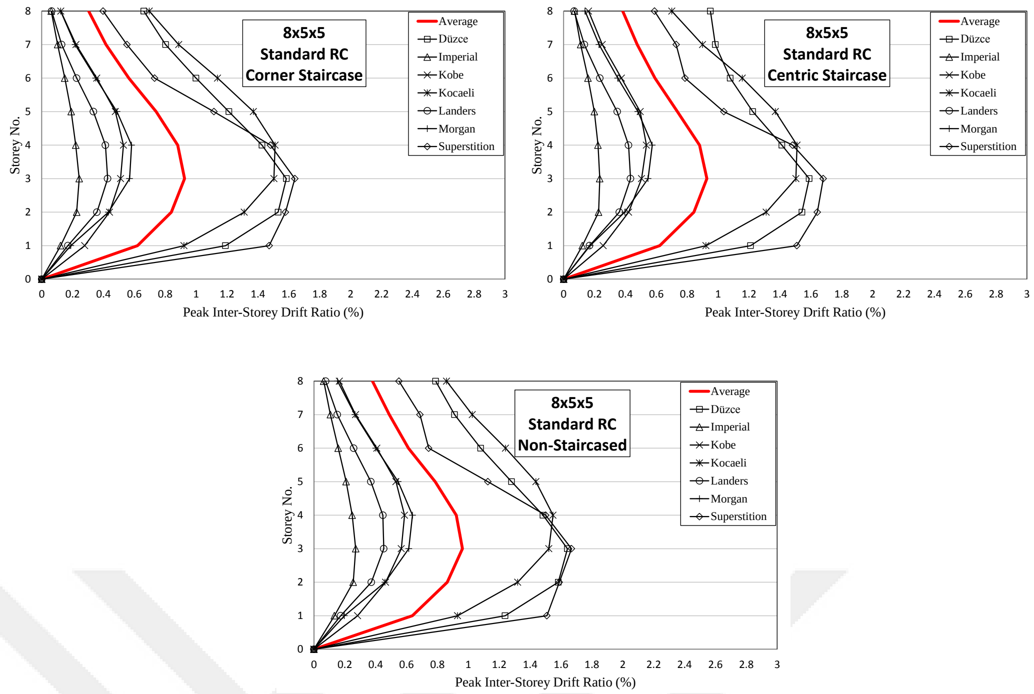


Figure 6.35 *Peak ISDR of 8x5x5_st models*

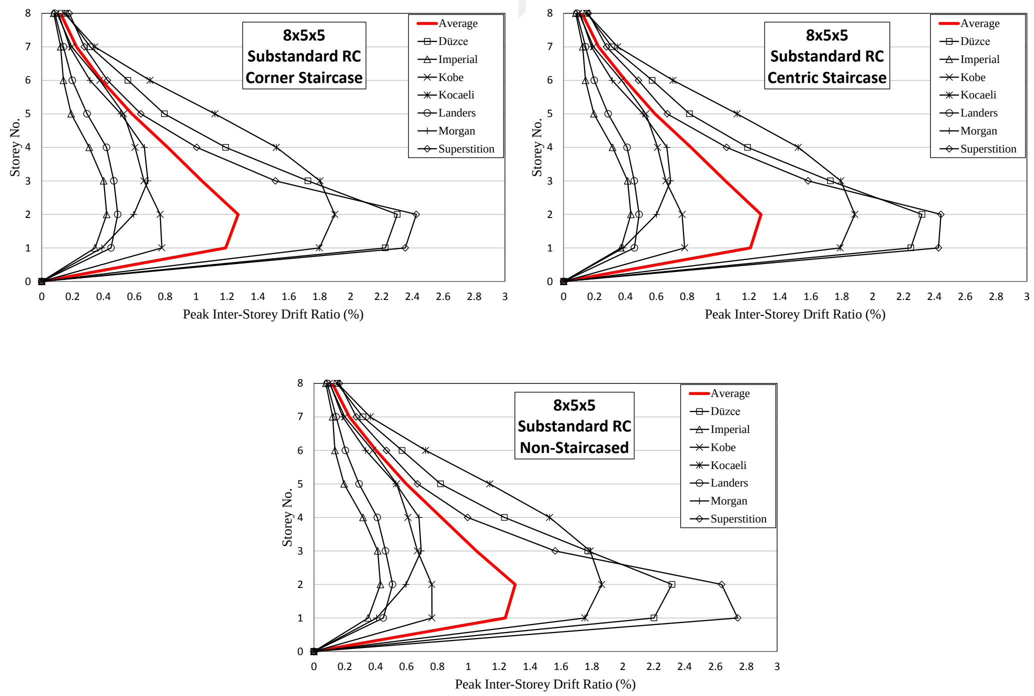


Figure 6.36 *Peak ISDR of 8x5x5_sub models*

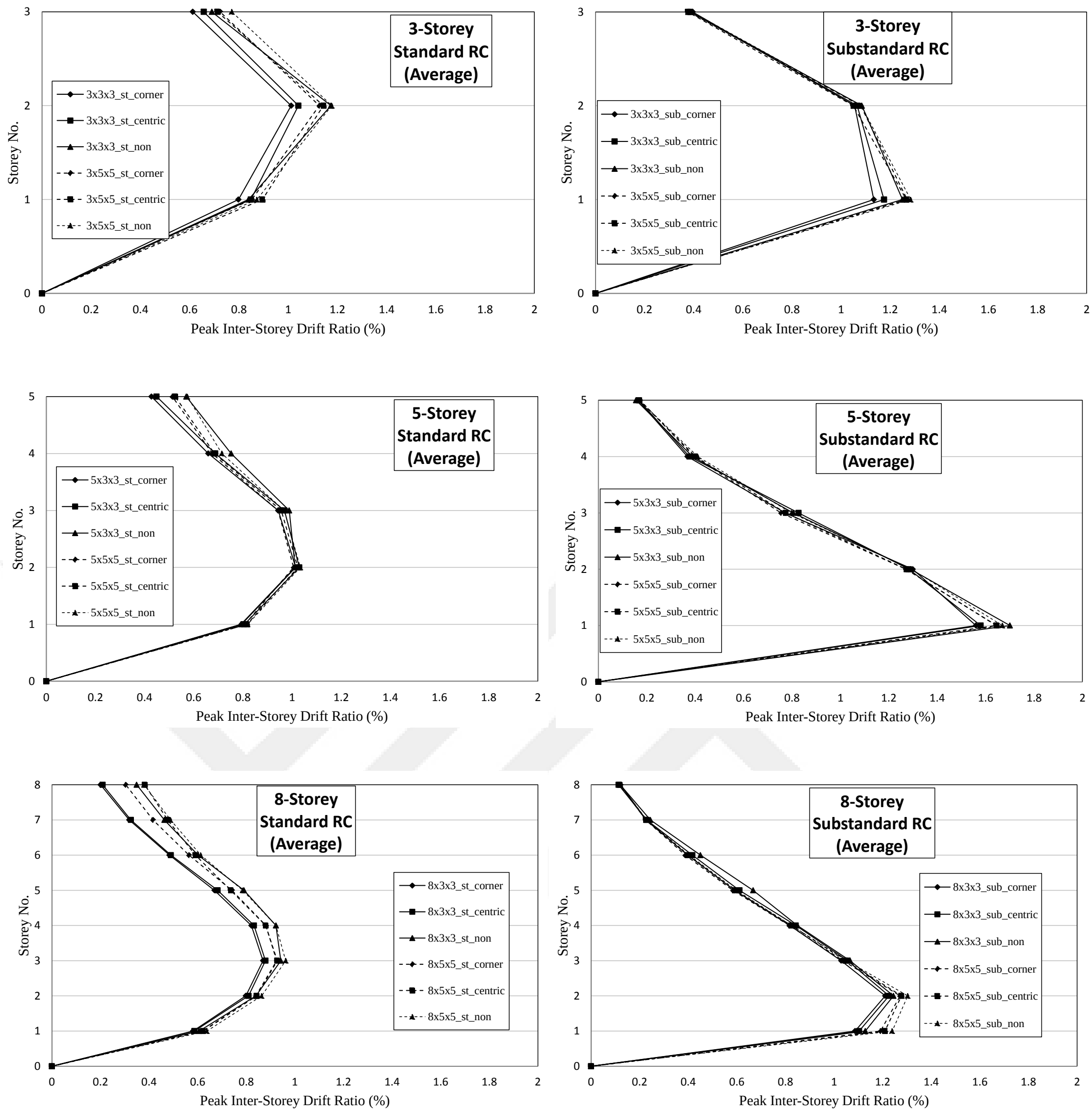


Figure 6.37 *Average ISDR of all models*

According to the results obtained from NLTHA in terms of ISDR, the following statements can be made;

- Similar to the response observed for the peak floor acceleration, Superstition Hills and Düzce Earthquake records caused larger ISDR values compared to the other earthquake records. Superstition Hills record even caused the total collapse of some of the 3-storey substandard RC buildings.

- While the ISDR with the highest values occurred at mid-level floors of the standard models, the ISDR with the highest values occurred at bottom floors of substandard models which can be attributed to existence of soft-storey formation in substandard structures due to the weak column – strong beam phenomenon.

- Presence of staircases decreased ISDR due to their stiffening effect. The highest ISDR is generally observed in the analytical models without staircase.

- 5-storey buildings have resulted higher ISDR values for both standard and substandard cases compared to the 3 and 8 storey buildings.

- As the number of spans increased, the presence of staircases has less impact on the buildings in terms of ISDR. Due to the increased number of columns of five spanned models, the contribution of the staircase to the overall stiffness of the buildings became more negligible.

- As the mass of the building increases -due to the higher number of spans- ISDR increases.

- Just as the results obtained from floor accelerations, presence of staircase did not affect the ISDR results considerably.

6.4.3. Shear force demands in columns

In this section, shear force demands in the columns that are supporting the staircase were investigated for both 2-2 and 3-3 local axes of the columns in centric staircased and non staircased cases. The reference columns chosen for analyses are shown in Figures 6.38 and 6.39. In these figures, blue arrows stand for 3-3 local axis which is parallel to flight running direction, while green arrows stand for 2-2 local axis.

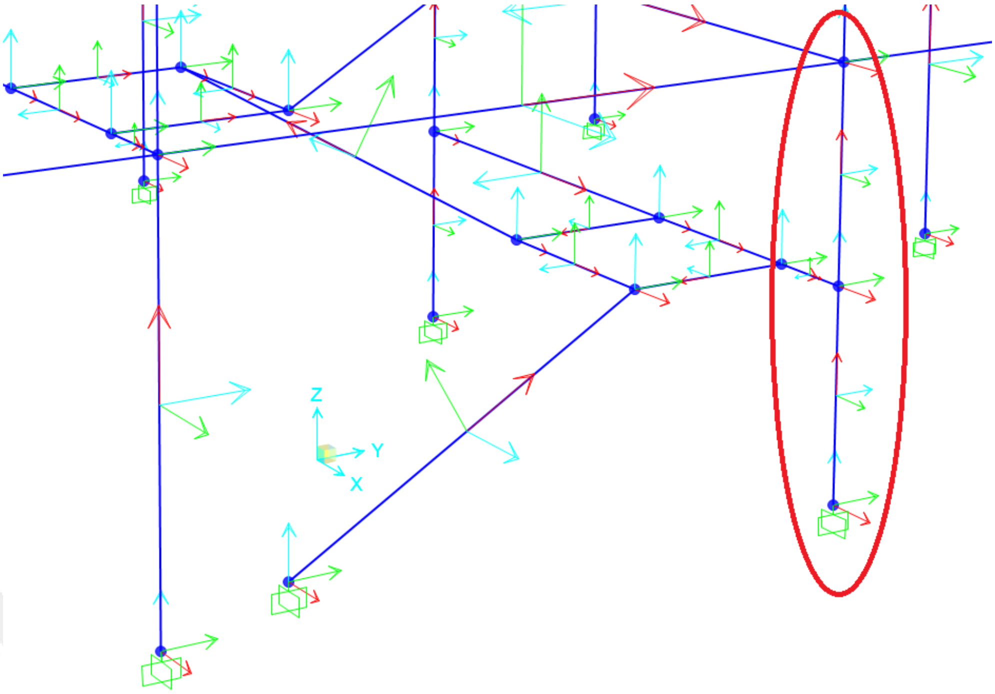


Figure 6.38 *The chosen reference column of centric staircased models*

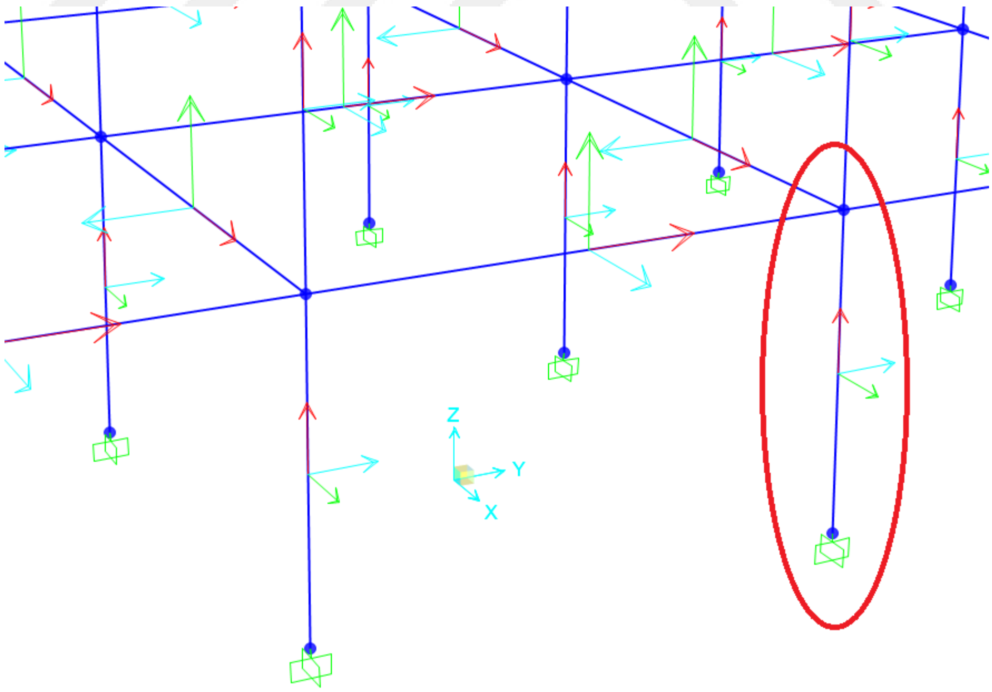


Figure 6.39 *The chosen reference column of non-staircased models*

In Figure 6.40, the shear diagrams of the core frame of a staircased model (on the left) and of a non-staircased model (on the right) are presented. From the figure, it can be seen that the presence of a staircase causes considerable amount of variation in the shear force imposed at the intersection of staircase members and columns. This situation can be attributed to a phenomenon called "short column" and this may end up with the exceedance of the shear force carrying capacity of a column. In order to control whether the members exceeded their shear force carrying capacity or not, shear hinges assigned to all members. From the NLTHA results, the excession of shear force carrying capacity has only been occurred on staircase supporting columns. For that reason, shear force demands of the reference columns which were mentioned in Figures 6.38 and 6.39 were compared in Figures 6.41, 6.42 and 6.43. The red line in Figures 6.41, 6.42 and 6.43 represents the shear force carrying capacity of the columns. The shear force carrying capacity was calculated according to equations (3.4), (3.5), (3.6) and (3.7).

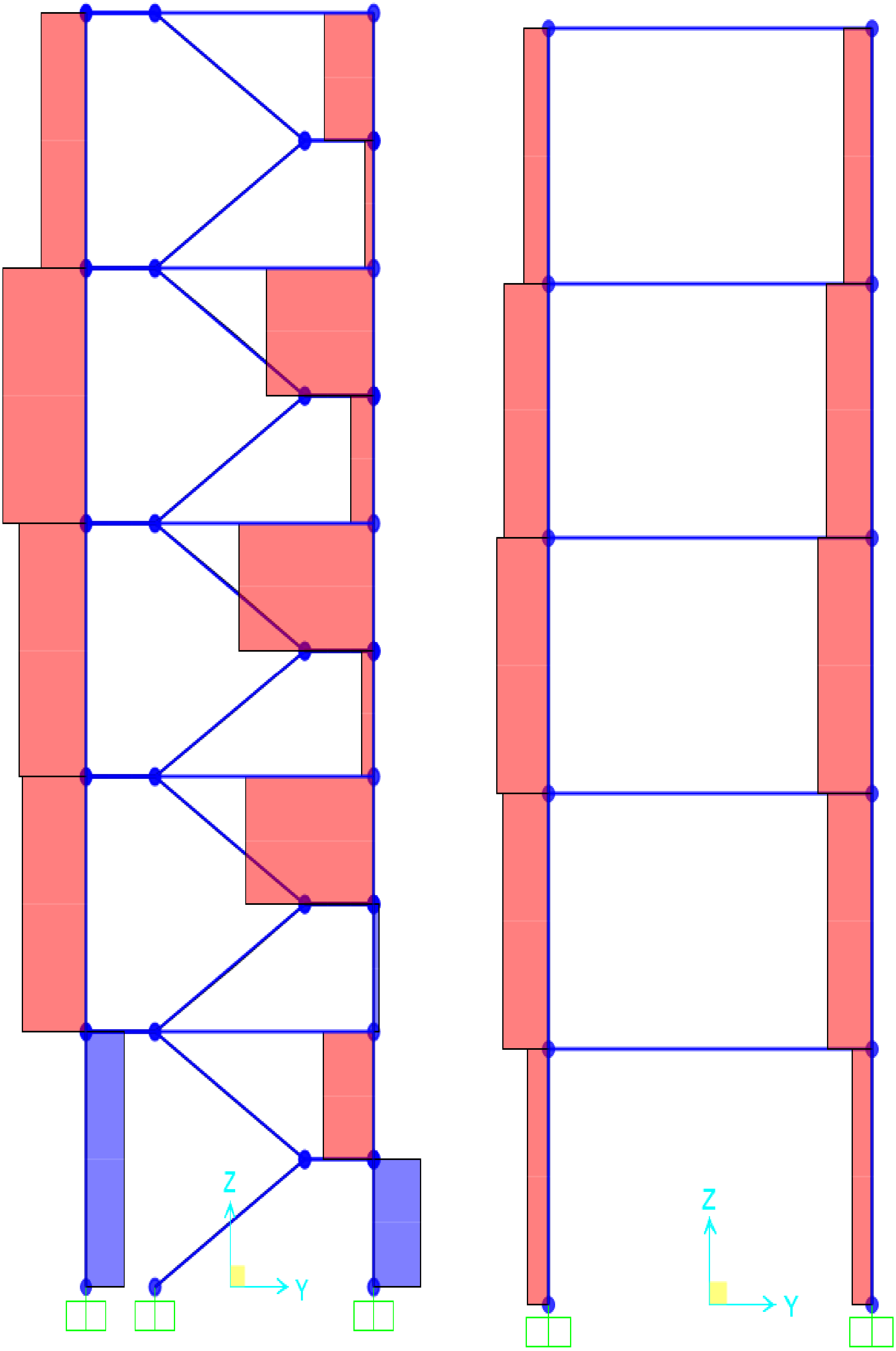


Figure 6.40 *Distribution of shear force along short-column (on the left) and normal column (on the right) during Superstition Hills Earthquake*

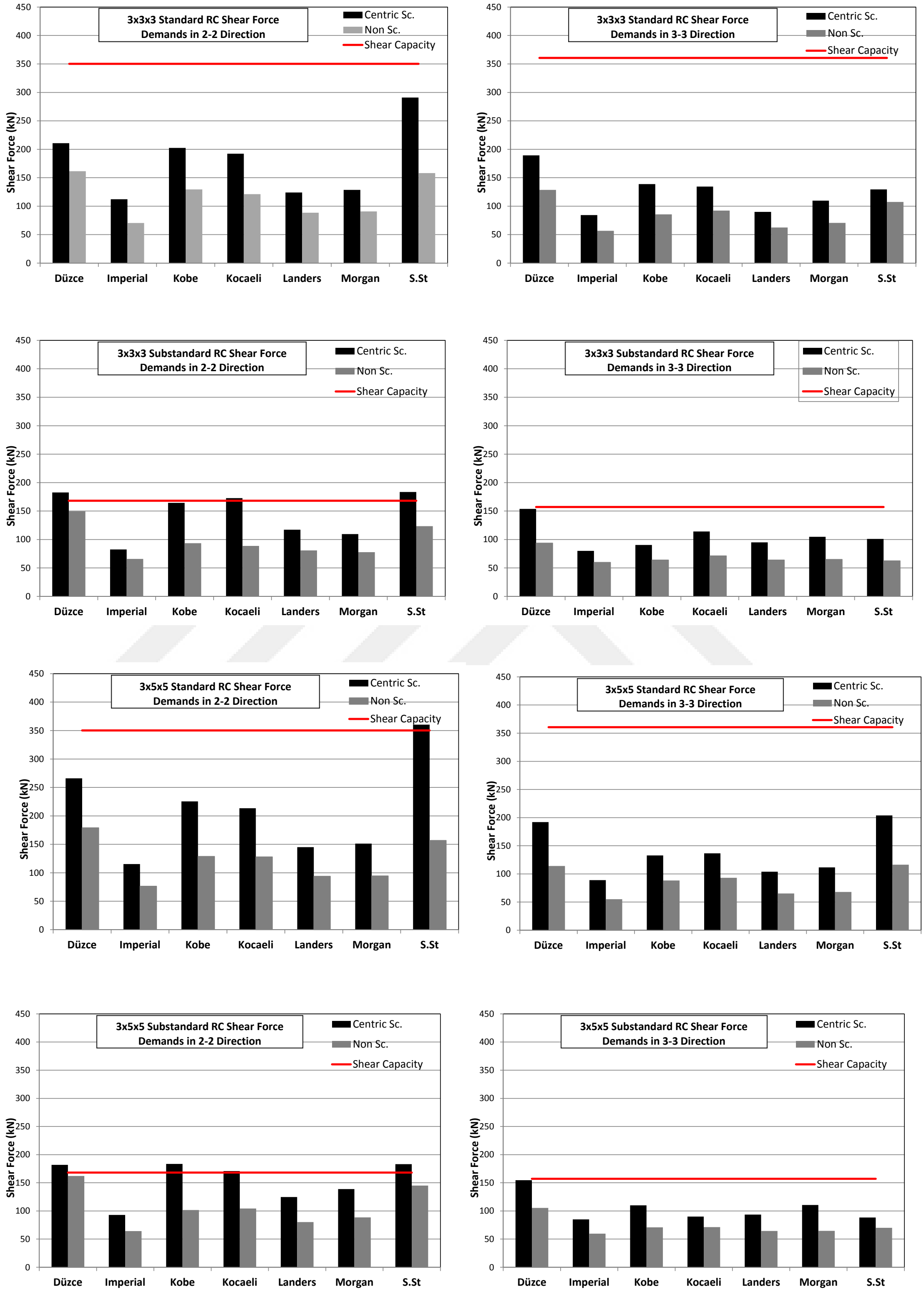


Figure 6.41 *Shear force demands of the reference columns of 3 storey models*

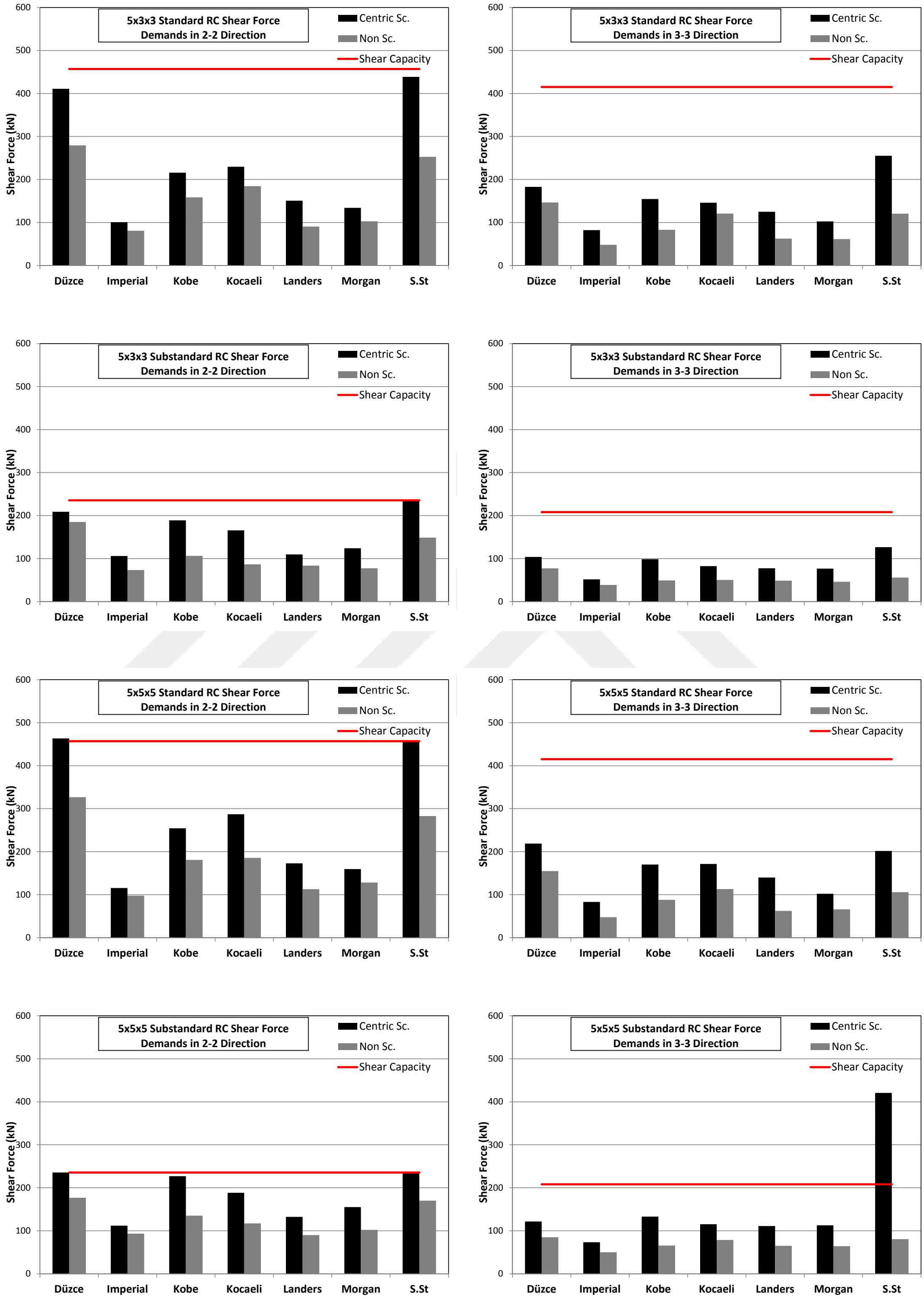


Figure 6.42 *Shear force demands of the reference columns of 5 storey models*

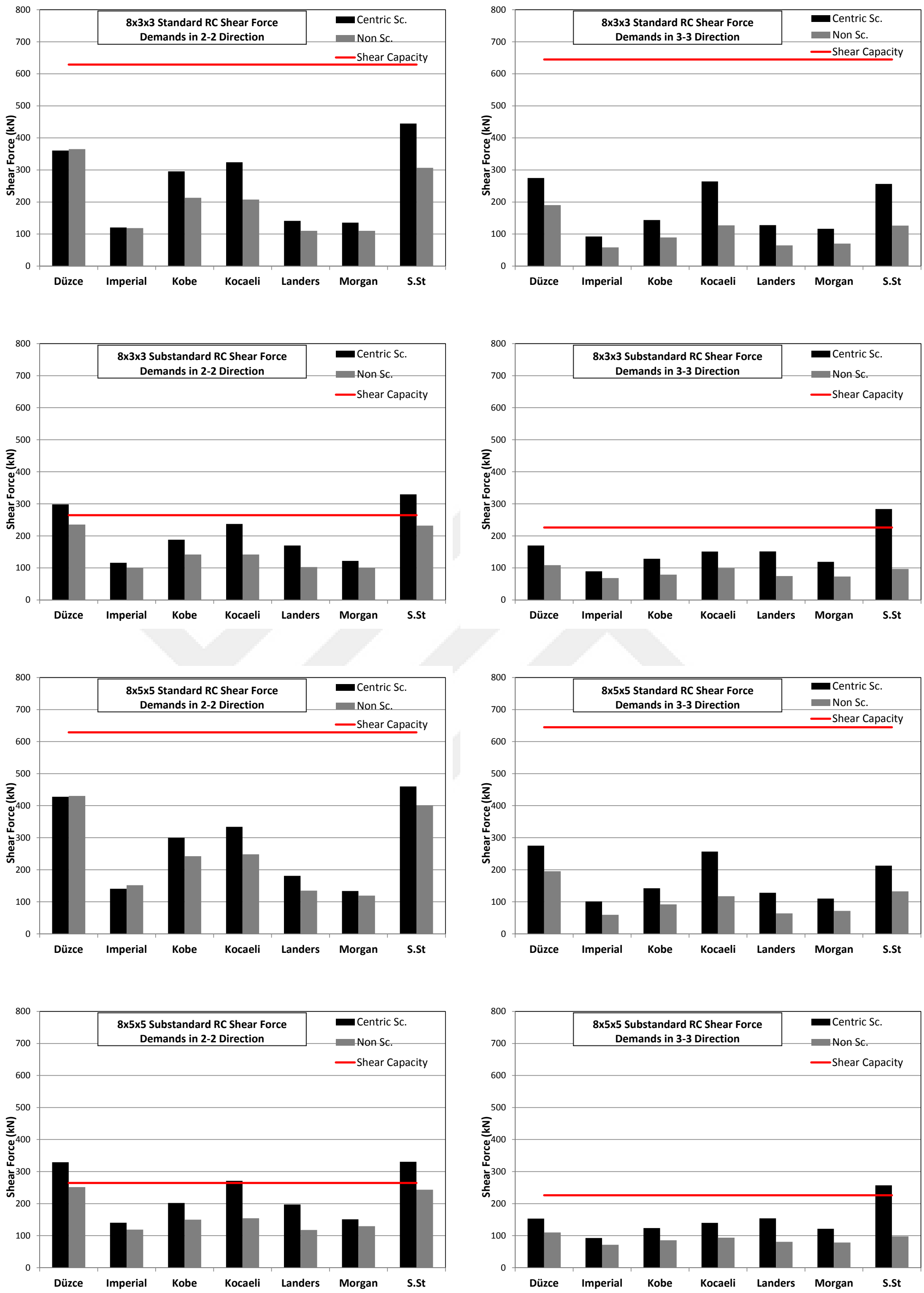


Figure 6.43 *Shear force demands of the reference columns of 8 storey models*

Based on the shear force demands of the reference column emphasized previously, the following statements can be made;

- For each earthquake ground motion analysis, the selected column is exposed to higher shear demands due to the presence of staircase. This is an expected outcome due to the formation of short column with the presence of staircase.

- Consideration of staircase in the analytical models leads to exceedance of shear capacity of the reference columns of all substandard models and for some of the standard models as well. This was observed especially for Superstition Hills and Düzce Earthquake records.

- Since shear failure is a brittle type of failure, it is found that staircases can cause column shear failure, which can even lead to total collapse of the building.

- Even though a few of the standard models failed in terms of exceedance of the shear capacity of the selected column, designing staircased buildings should not be left to chance by not incorporating staircase to the structural analysis model.

6.4.4. Effect of staircase and its position on the base story shear force

Base shear force is the total shear force imposed on the basement columns. Calculating base shear forces gives an insight about seismic demand imposed on the building. In this study base shear forces have only been calculated for Y direction which is parallel to staircase flight running direction. In Figure 6.44, average base shear force demands of each model is presented. In Figure 6.45, average weight normalized base shear graphs of each model is presented in order to investigate the base shear demand of the models with respect to their own weight.

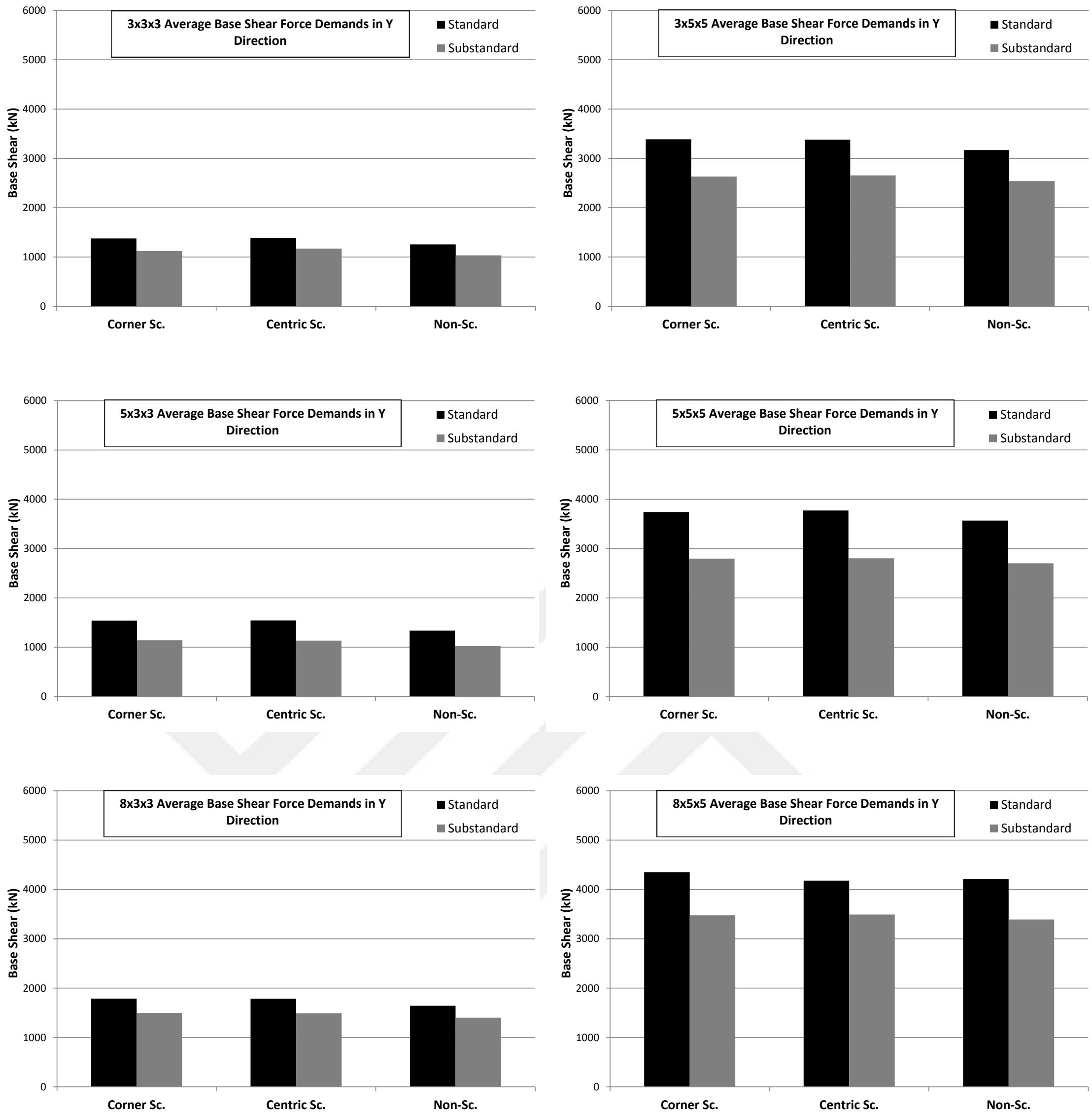


Figure 6.44 *Average base shear force demands of all models*

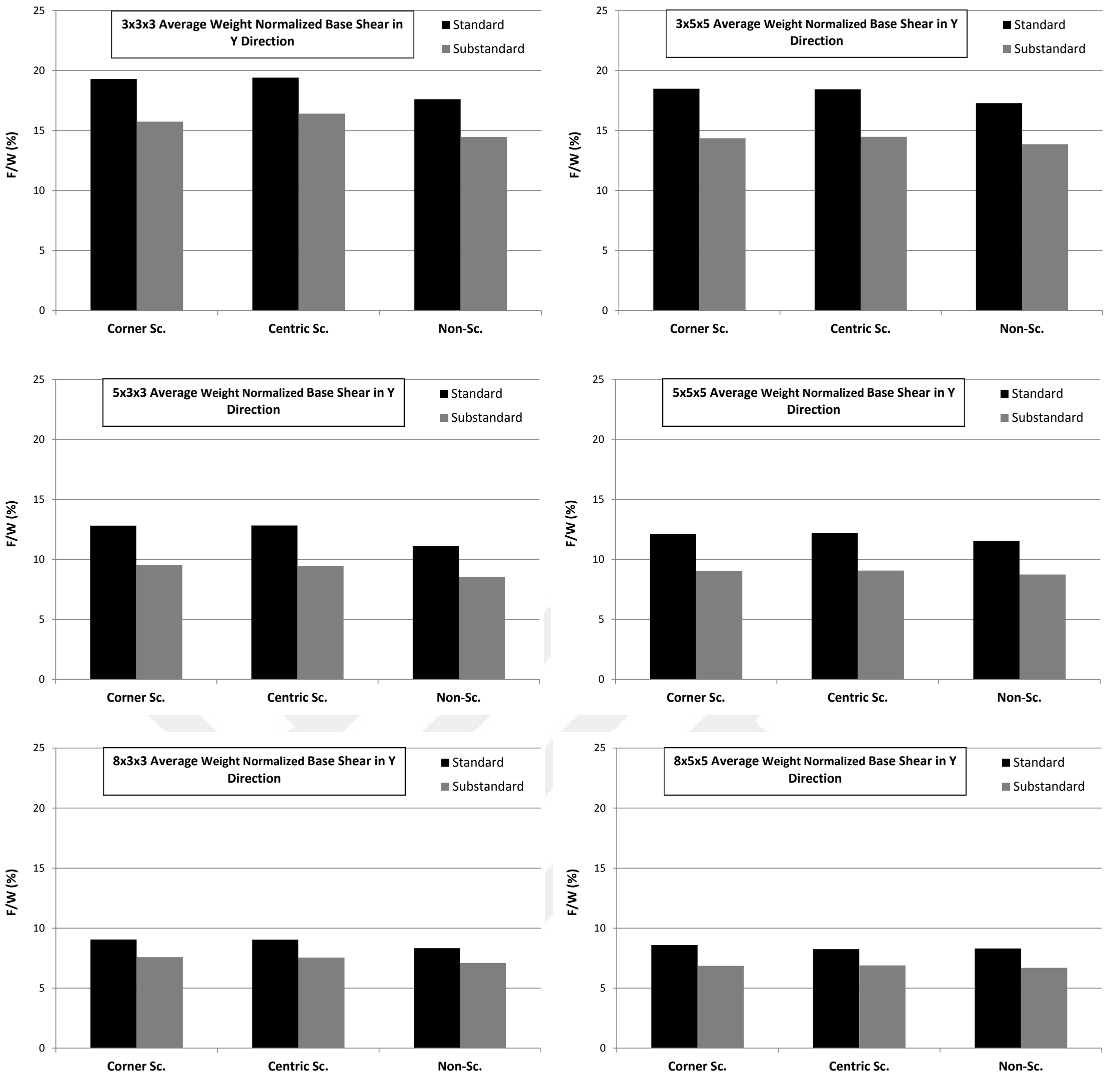


Figure 6.45 *F/W ratios of all models*

From the results shown in average base shear force demands graphs, the following statements can be made;

- Inclusion of staircases increases average base shear forces. However, the increment in the base shear demand is not that much significant.
- The position of staircase has no influence on the base shear force.
- Substandard buildings have exposed to less base shear force compared to standard ones due to their lower stiffness.

- As the number of spans increases, the base shear force increases. For instance, 5x5 span buildings have more than twice as much base shear demand as 3x3 span buildings have.

- Base shear demand increases as the number of storey increases. However, the effect of number of storey on the building base shear is less influential than the number of spans.

- As the buildings get taller, their F/W ratio decreases due to decrement in their stiffness.

6.4.5. Effect of staircase on the plastic hinge distribution

In this section, the distribution of plastic hinges is examined under Superstition Earthquake which is one of the most damaging earthquakes among the other selected earthquakes due to its high spectral acceleration values and high PGV value.

Damage on the plastic hinges is evaluated according to the coloured scale shown in Figure 6.46 (on the right). These colours refer to the conditions of a hinge which is shown as an example in Figure 6.46 (on the left). By this way, the effect of staircase on the distribution of both flexural and shear hinges is investigated. In the coloured scale, "B" stands for the beginning of inelastic behavior, "IO" stands for "immediate occupancy", "LS" stands for "life safety" and "CP" stands for "collapse prevention".

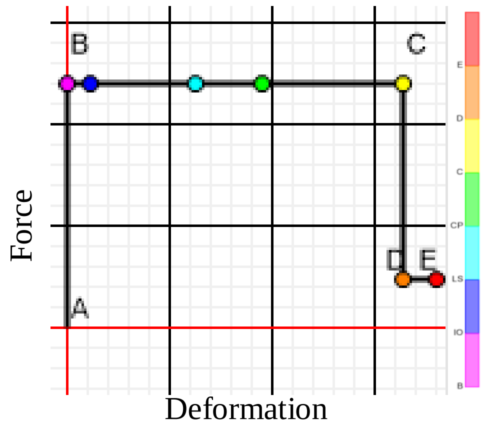


Figure 6.46 *Hinge conditions on a representative force-deformation graph*

The distributions of the plastic hinges are presented in the figures between 6.47 and 6.82. In these figures the presented frames are the critical frames which include staircase components.

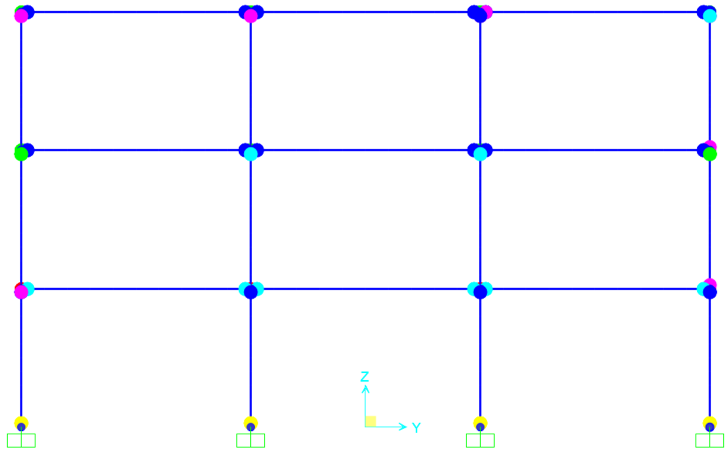


Figure 6.47 *Condition of 3x3x3_st_non*

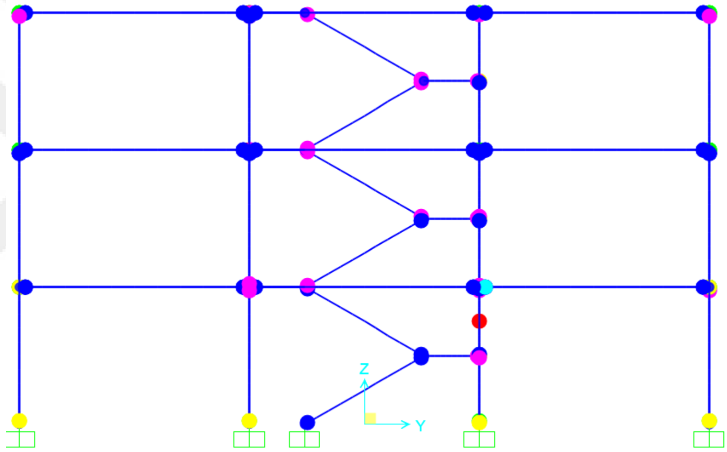


Figure 6.48 *Condition of 3x3x3_st_cent*

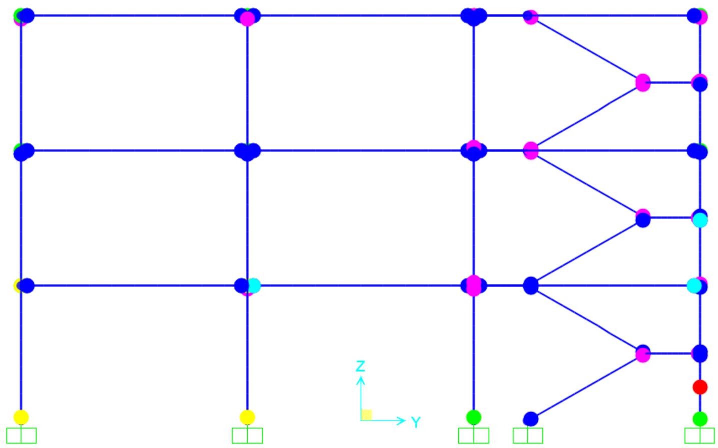


Figure 6.49 *Condition of 3x3x3_st_cor*

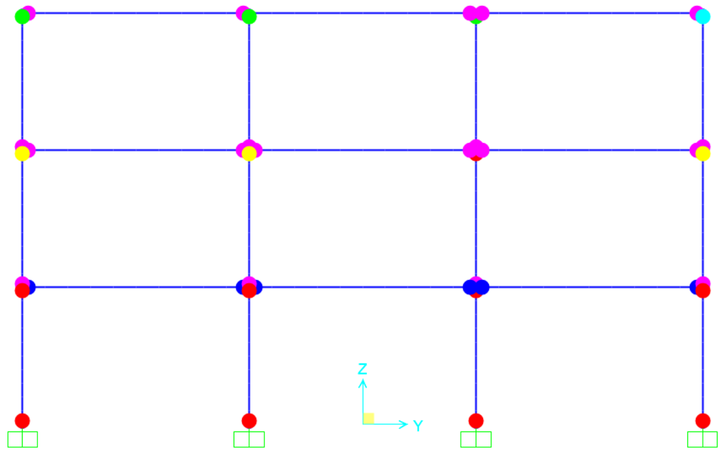


Figure 6.50 *Condition of 3x3x3_sub_non*

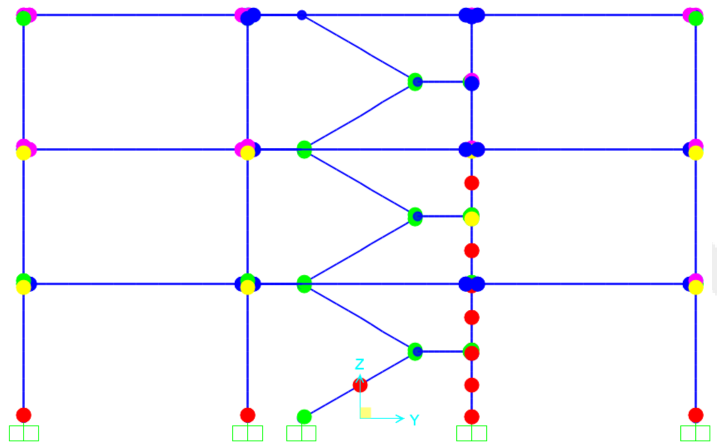


Figure 6.51 *Condition of 3x3x3_sub_cent*

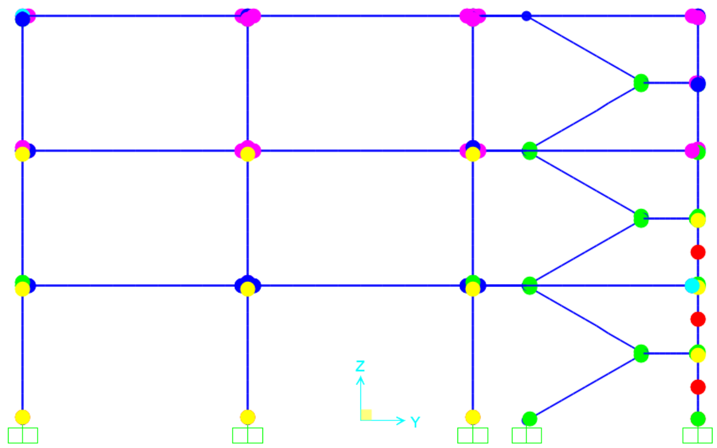


Figure 6.52 *Condition of 3x3x3_sub_cor*

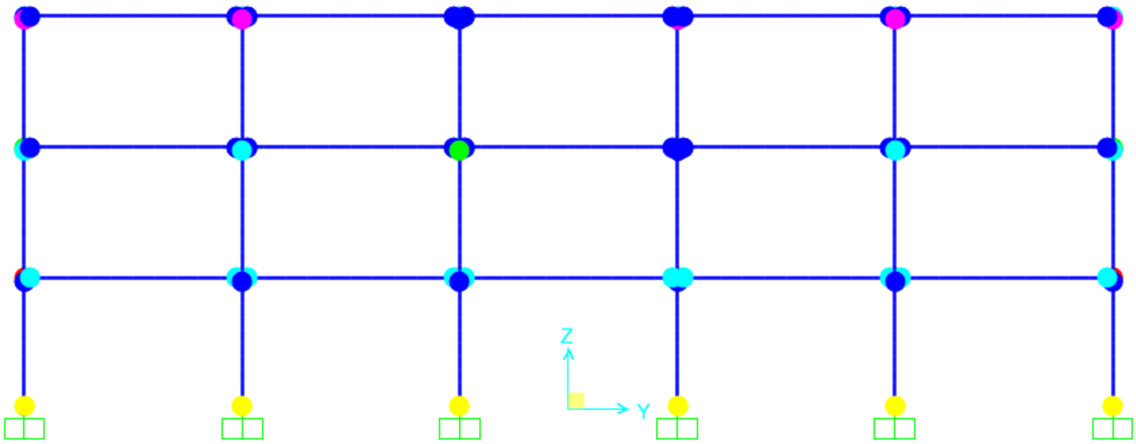


Figure 6.53 *Condition of 3x5x5_st_non*

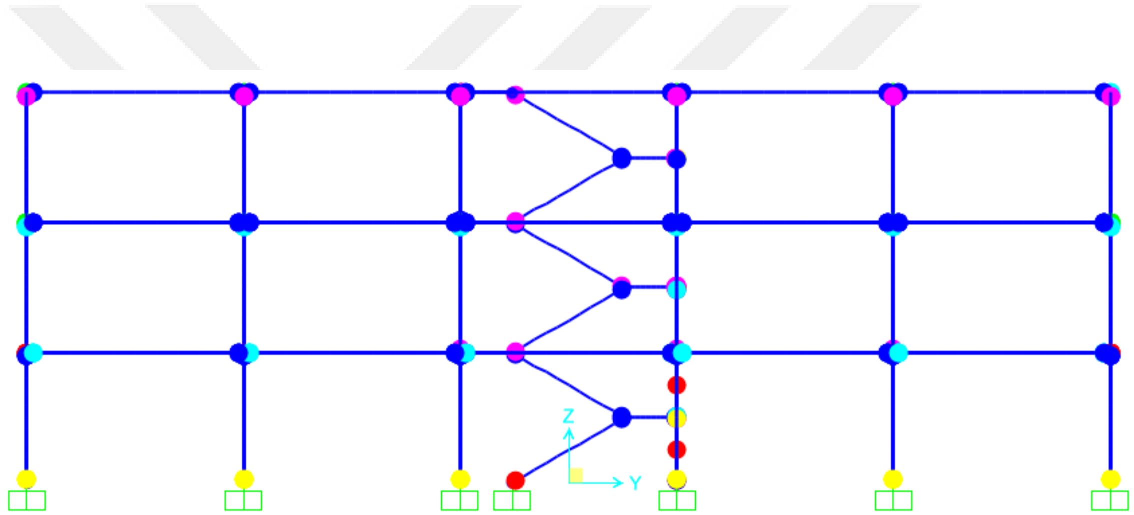


Figure 6.54 *Condition of 3x5x5_st_cent*

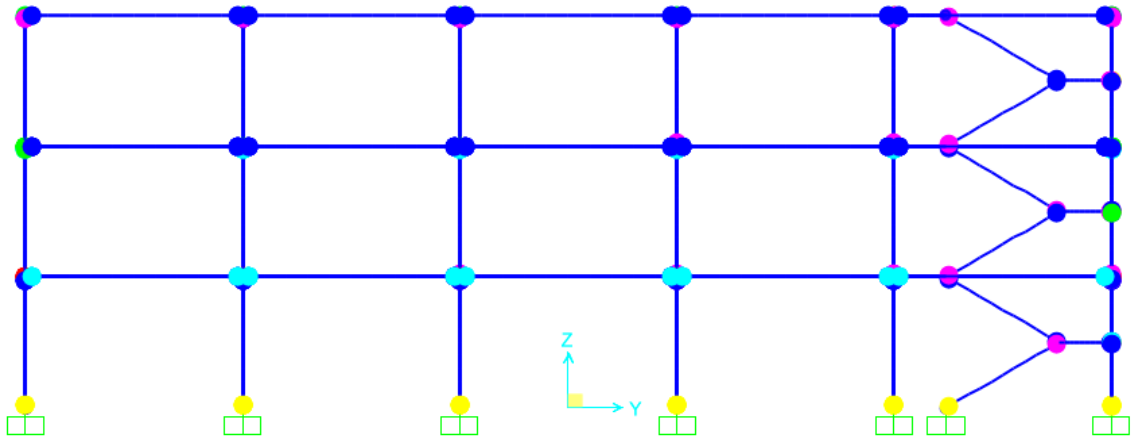


Figure 6.55 *Condition of 3x5x5_st_cor*

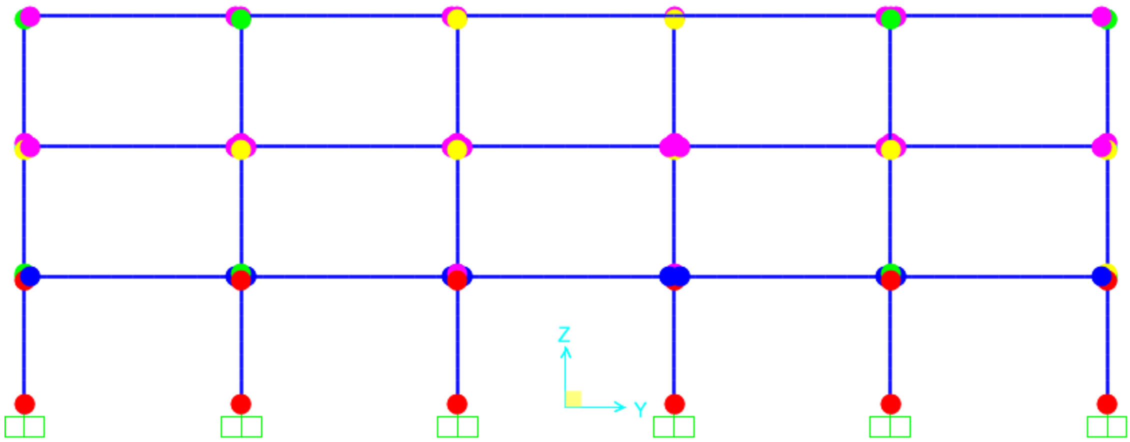


Figure 6.56 *Condition of 3x5x5_sub_non*

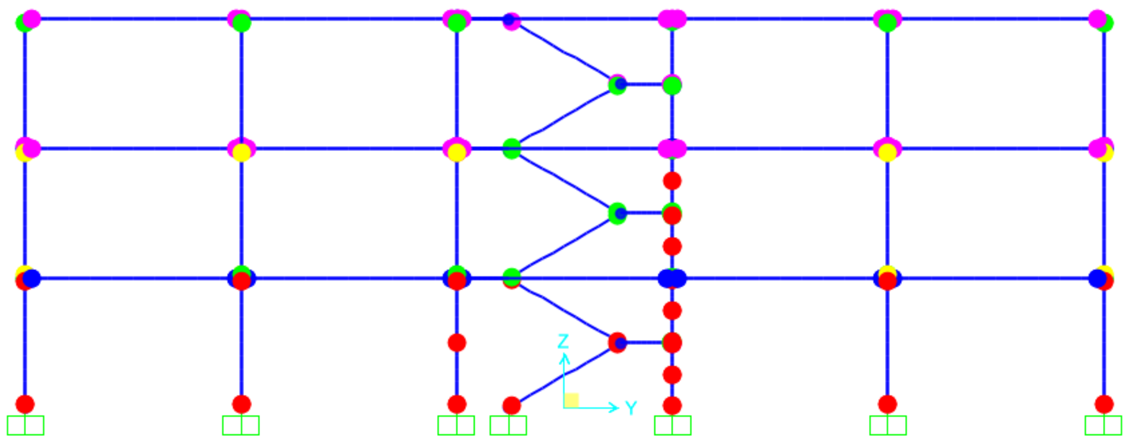


Figure 6.57 *Condition of 3x5x5_sub_cent*

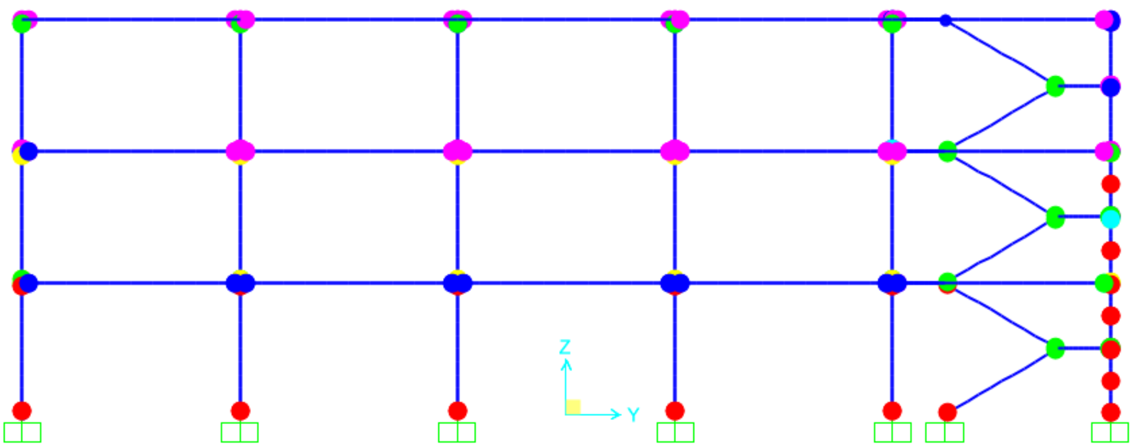


Figure 6.58 *Condition of 3x5x5_sub_cor*

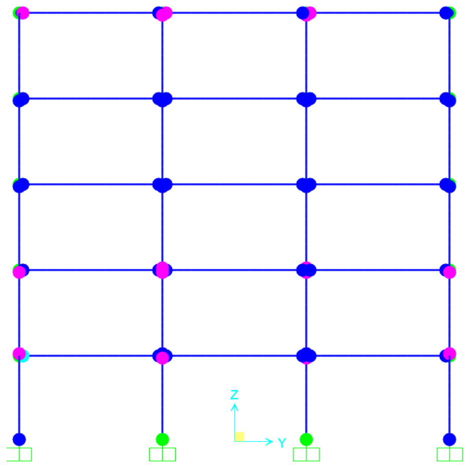


Figure 6.59 *Condition of 5x3x3_st_non*

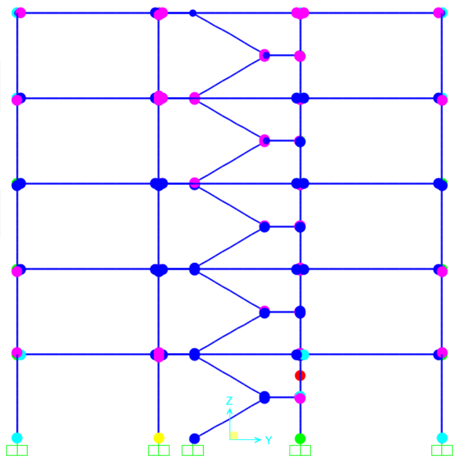


Figure 6.60 *Condition of 5x3x3_st_cent*

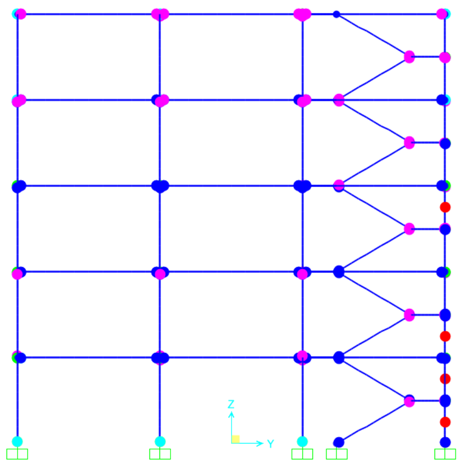


Figure 6.61 *Condition of 5x3x3_st_cor*

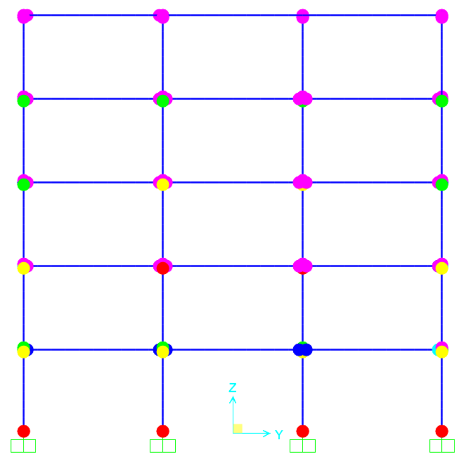


Figure 6.62 *Condition of 5x3x3_sub_non*

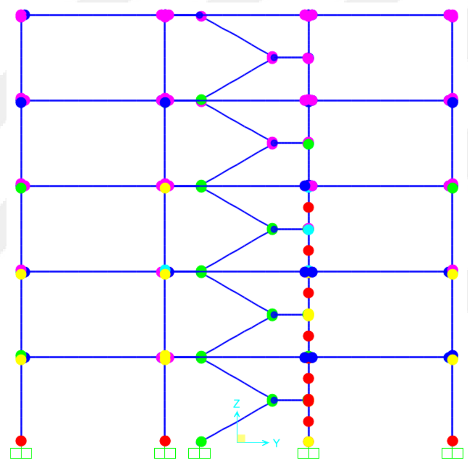


Figure 6.63 *Condition of 5x3x3_sub_cent*

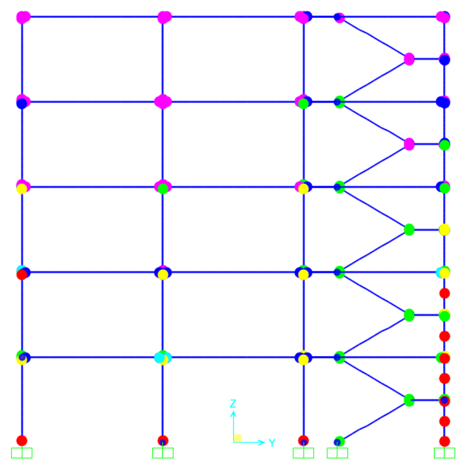


Figure 6.64 *Condition of 5x3x3_sub_cor*

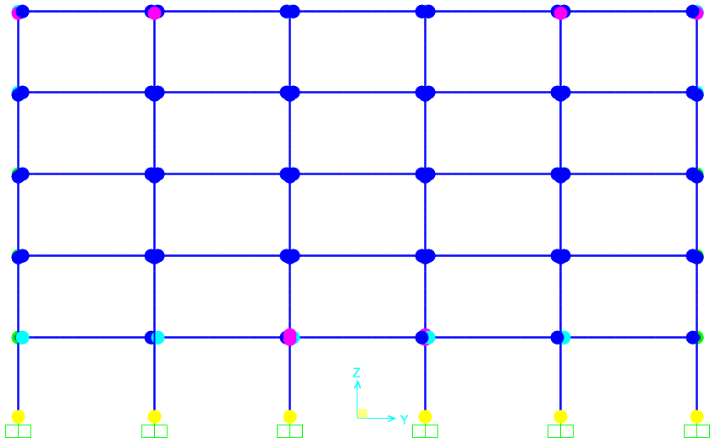


Figure 6.65 *Condition of 5x5x5_st_non*

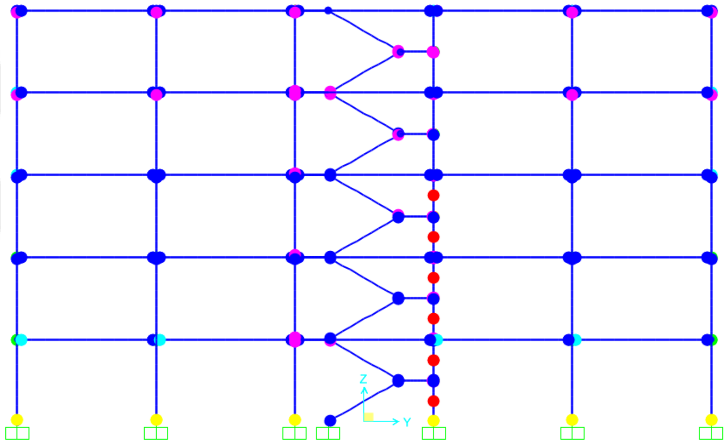


Figure 6.66 *Condition of 5x5x5_st_cent*

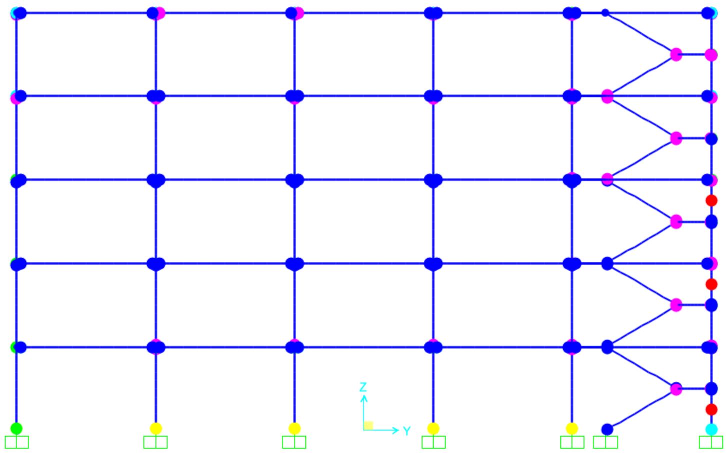


Figure 6.67 *Condition of 5x5x5_st_cor*

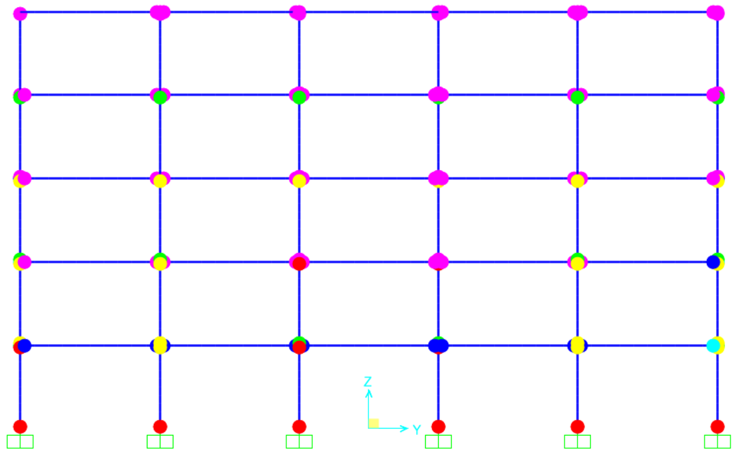


Figure 6.68 *Condition of 5x5x5_sub_non*

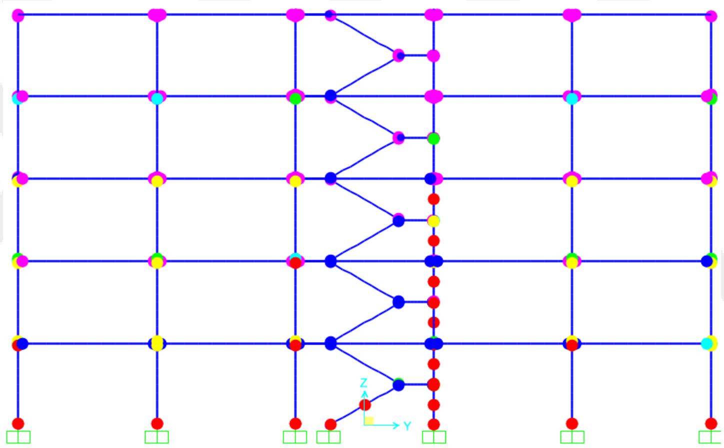


Figure 6.69 *Condition of 5x5x5_sub_cent*

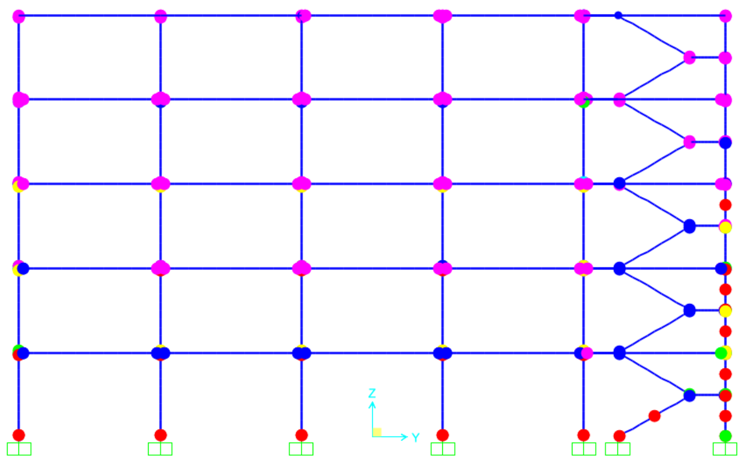


Figure 6.70 *Condition of 5x5x5_sub_cor*

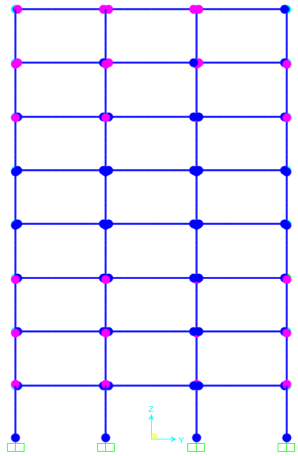


Figure 6.71 *Condition of 8x3x3_st_non*

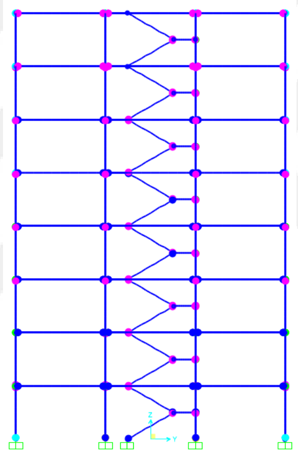


Figure 6.72 *Condition of 8x3x3_st_cent*

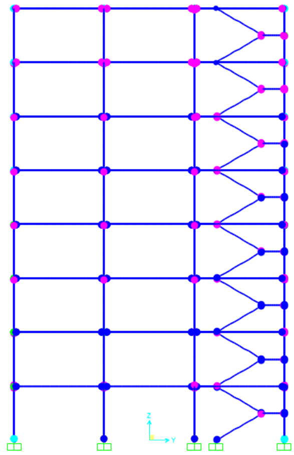


Figure 6.73 *Condition of 8x3x3_st_cor*

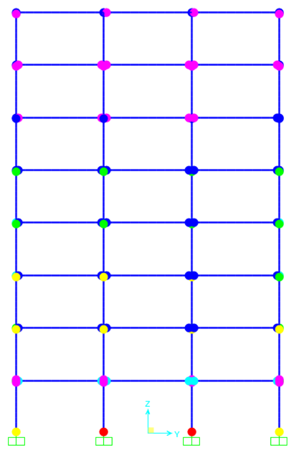


Figure 6.74 *Condition of 8x3x3_sub_non*

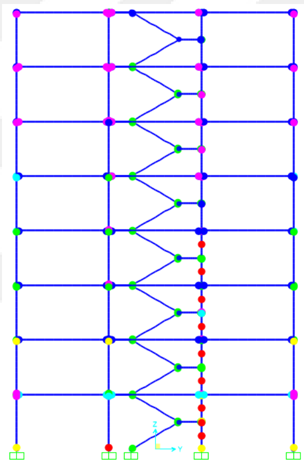


Figure 6.75 *Condition of 8x3x3_sub_cent*

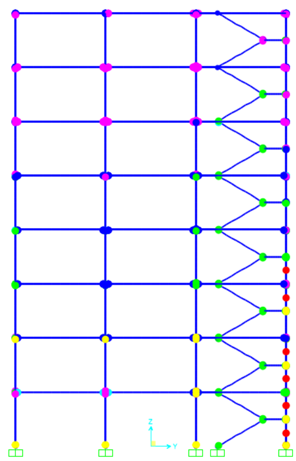


Figure 6.76 *Condition of 8x3x3_sub_cor*

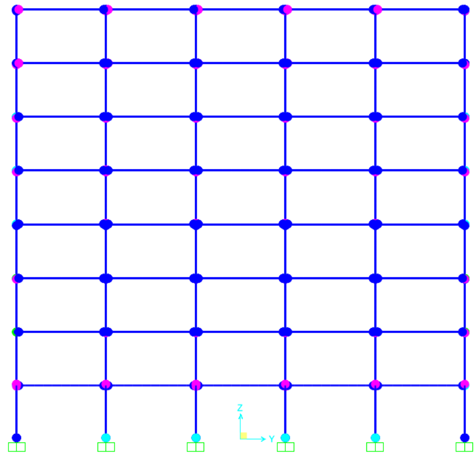


Figure 6.77 *Condition of 8x5x5_st_non*

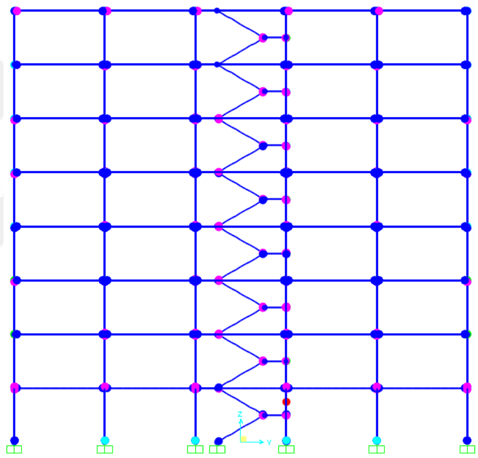


Figure 6.78 *Condition of 8x5x5_st_cent*

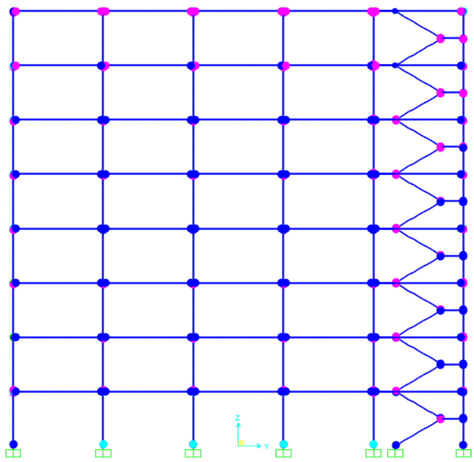


Figure 6.79 *Condition of 8x5x5_st_cor*

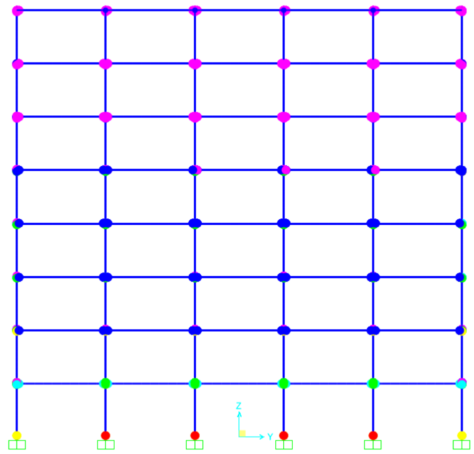


Figure 6.80 *Condition of 8x5x5_sub_non*

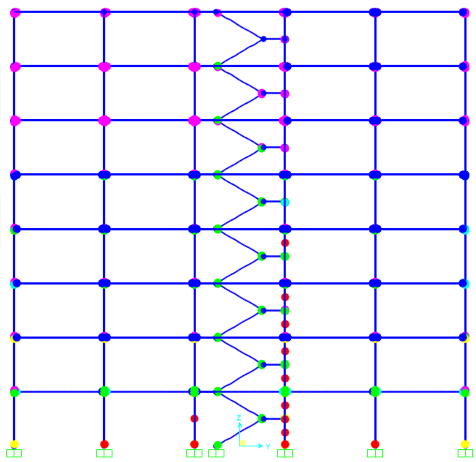


Figure 6.81 *Condition of 8x5x5_sub_cent*

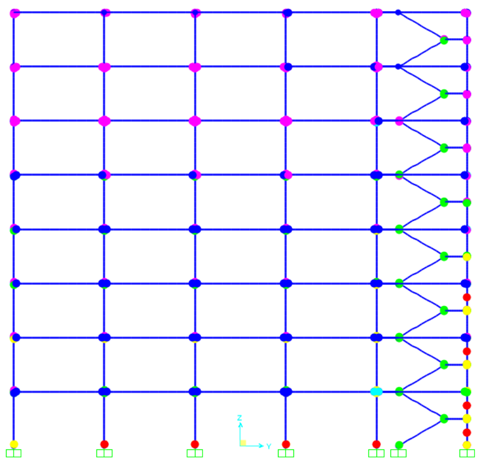


Figure 6.82 *Condition of 8x5x5_sub_cor*

By comparing the figures between 6.47 and 6.82 which show the plastic hinge and shear hinge distributions on the models, the following statements can be made;

- In general, damage was occurred at the bottom ends of the first floor columns of both standard and substandard buildings without staircase.
- Damage level on the columns is higher with respect to the beams of substandard building as expected due to strong beam-weak column phenomenon. However, beams exposed to more damage than columns of standard buildings as it was designed.
- Staircase insertion in the analytical model caused shear hinges to develop due to the short column effect by the interaction between mid-floor staircase landing beams and columns.
- No shear damage has been observed on non-staircased models. Therefore, it is not realistic to design staircased buildings without a staircased analysis model.
- Substandard models were exposed to more damage due to their low ductility.
- In both standard and substandard models, staircases caused damage to adjacent structural members. The observed damage type is both flexural and shear.
- Even though the locations of staircases change, the damage due to short column formation kept concentrating around staircases.

7. CONCLUSIONS

In this parametric study, the impact of staircases on the seismic performance of standard and substandard structures was investigated. Various RC framed buildings were designed which differ in number of storey, number of spans, condition of the building (standard and substandard), location of staircase and existence of staircase. Firstly, specimens were pre-dimensioned according to the non-staircased case and were tested by STA4CAD -for designing substandard specimens and at the same time considering Turkish Earthquake Code 1975- and IDECAD -for designing standard specimens according to Turkish Earthquake Code 2007- structural analysis and design softwares. After the section dimensions and reinforcement details were specified, the specimens were re-modelled by SAP2000 in order to examine the impact of staircases in terms of modal properties such as modal participation factors and fundemental mode periods, inter-storey drift ratio, floor accelerations, member shear forces, base shear force, plastic hinge distribution and formation of short columns. In order to examine them, modal analyses, pushover analyses and nonlinear time-history analyses were conducted. XTRACT was used for determining the moment-curvature data of the cross-sections of beams, columns and staircase frames to be used in modeling the non-linear behavior of the members. The damage limits on those plastic hinges were determined according to TEC 2007 requirements. Shear hinges were defined to observe the exceedance of the shear capacity of the structural members.

According to the data obtained, conclusions of this study can be made as listed below;

- Staircases alter the modal parameters of the buildings such as modal participating mass ratio (MPMR) and natural vibration period. MPMR decreases due to the structural irregularity induced by staircases and natural vibration period decreases due to local stiffening effect of staircases. Moreover, eccentrically placed staircases have a greater effect on decreasing the MPMR by causing the non-uniform distribution of stiffness in building layout. In addition, eccentrically placed staircases caused less decrement in natural vibration period with respect to centric staircases. Further, material quality affects natural vibration periods such that, substandard buildings have longer natural vibration periods.

- As the buildings get taller and larger in the plan view, the difference between the minimum and maximum peak floor acceleration increases.

- The difference between the minimum and maximum peak floor acceleration is greater in substandard models than the one for standard buildings.

- While the ISDR with the highest values occurred at mid stories of the standard models, the ISDR with the highest values occurred at bottom floors of substandard models which can be attributed to existence of soft-storey formation in substandard structures due to the weak column – strong beam phenomenon.

- Shorter substandard buildings were exposed to more damage with respect to taller buildings since four of the three storey substandard buildings collapsed whereas, none of the eight storey specimens collapsed.

- Presence of staircases decreased ISDR due to their stiffening effect.

- As the number of spans increased, the presence of staircases has less impact on the buildings in terms of ISDR. Due to the increased number of columns of five spanned models, the contribution of staircases to the overall stiffness of the buildings became more negligible.

- Staircase insertion to the models leads to an exceedance of shear capacity of the short columns due to staircases in all substandard models and some of the standard models.

- The inclusion of staircases increases base shear force demand of buildings. Besides, substandard buildings have less base shear force demand compared to standard ones. Further, as the buildings get taller and heavier, their base shear force demand increases while their F/W decreases.

- The highest damage occurred at the bottom ends of the first floor columns of both standard and substandard buildings.

- Using relatively stronger beams with respect to columns in substandard models lead the columns to develop plastic hinges earlier than the beams, which is an undesirable behavior in terms of earthquake resistant building design. Besides that, yielded columns could not prevent the lateral load to concentrate around staircase which caused the adjacent elements to the staircase to receive excessive amounts of seismic demand causing severe damage.

- Staircase insertion caused shear hinges to develop due to the short column effect by the interaction between mid-floor staircase landing beams and columns.

- No shear damage has been observed in non-staircased models. Therefore, it is not realistic to design staircased buildings without a staircased analysis model.

- Substandard models received more damage due to their low overall ductility.

- Incidentally, in 3x3x3_sub models, staircases prevented the building from collapsing by limiting relative story drift while non-staircased model collapsed.

- Effect of staircase on the seismic response of both standard and substandard buildings was not that much significant in terms of peak floor acceleration and ISDR. However, the effect of staircase was explicitly observed in the base shear force and critical column shear force distribution. Moreover, the presence of staircase has a significant influence on the plastic hinge distribution of the structural components. Especially, more plastic hinges were developed in the structural members close to staircase. Besides, only the flexural damage was observed in the non-staircased models, but shear failure was also observed in the staircased models, especially in the columns close to staircase due to formation of short columns.

8. RECOMMENDATIONS

According to the results obtained from this study, it has been seen that all existing buildings that are designed without taking staircase elements into account in the structural analysis model are not realistic. Those structures should be subjected to performance analysis by considering their staircases. Otherwise, the analyses results do not represent the actual behavior of the building during seismic events.

As an alternative, staircase elements of the standard buildings can be isolated from the system as shown in Figure 1.2.

For further study, staircase type can be added as an additional parameter. Usage of staircases without cantilever landing slabs (inclined staircase slabs are directly supported by floor beams) may increase the truss action whereas, for staircases with cantilever landing slabs, flexural behavior takes part in the behavior of staircases together with axial behavior.

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