EVALUATION OF A SIMPLE METHOD FOR THE PRELIMINARY DESIGN OF LOW-TO-MEDIUM RISE REINFORCED CONCRETE STRUCTURES

by

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ABSTRACT

EVALUATION OF A SIMPLE METHOD FOR THE PRELIMINARY DESIGN OF LOW-TO-MEDIUM RISE REINFORCED CONCRETE STRUCTURES

A simple method is evaluated in this study for suitability to the preliminary design of low-to-medium rise reinforced concrete building-type structures without significant irregularity problems. The method discussed herein, originally proposed by Ersoy (2013) is based on essential rules and guidelines of earthquake engineering such as member ductility and global ductility. Two main classes of structures are considered: buildings consisting solely of frames and buildings with frames and shear walls (also referred to as dual systems). In frame systems, the seismic loads are resisted entirely by columns, whereas in dual systems it is assumed that the columns take 30% and the shear walls take 100% of the lateral loads. It is also assumed that the locations of the shear walls are well configured on the plan so that the lateral loads are distributed fairly throughout the entire frame and the concentration of forces around a specific region is prevented. In an effort to ensure flexural failure of the members, the dimensions of the vertical members are determined under the action of gravity loads and seismic loads. Simple expressions representing the demand are derived as a function of plan area of the stories, column tributary areas, dead loads and live loads. Once the demands and required capacities are specified, dimensions of the load bearing members are determined by taking into account restrictions regarding certain effects leading to brittle failure such as axial stress, shear stress and interstory drifts. For the flexural and shear reinforcements, certain ratios of minimum reinforcements are adopted for the analysis. The buildings are modeled as per ASCE 41-13 with the aforementioned criteria and nonlinear static pushover analyses are performed to validate the expected performance level.

ÖZET

AZ VE ORTA YÜKSEKLİKLİ BETONARME BİNALARIN ÖNTASARIMI İÇİN BASİT BİR YÖNTEMİN DEĞERLENDİRİLMESİ

Bu çalışmada, büyük düzenlilik sorunu olmayan az ve orta yükseklikli betonarme binaların öntasarımı için basit bir yöntemin uygulanabilirliği irdelenmektedir. Bu kapsamda değerlendirilen ve ilk önce Ersoy (2013) tarafından önerilen yöntem, eleman ve sistem sünekliği gibi deprem mühendisliğinin temel ilkelerine dayanmaktadır. Burada iki bina sınıfı değerlendirilmiştir: salt çerçeveden oluşan sistemler ve çerçeve ve perde duvarların birlikte çalıştığı sistemler (bir diğer deyişle karma sistemler). Çerçeve sistemlerde deprem yüklerinin tümüyle kolonlar tarafından karşılandığı kabul edilmiştir; karma sistemlerde ise yükün %30'u kadarının çerçevelere, %100'ü kadarı ise perdelere etkitilmiştir. Bir diğer varsayım, perde duvarların planda görece düzgün biçimde yerleştirildiği ve dolayısıyla belirli bölgelerde yük yığılması olmadığıdır. Elemanların eğilme nedeniyle güç yitimine uğramasını sağlamak amacıyla eleman boyutları belirlenirken düşey yüklerin ve deprem yüklerinin etkileri dikkate alınır. Deprem talebini belirlemekte kullanılan baistleştirilmiş kestirimler, katların plan alanına, kolonların alan paylarına, sabit yüklere ve hareketli yüklere dayandırılmıştır. Talep ve gerekli kapasiteler belirlendikten sonra yük taşıyıcı elemanlar boyutlanırken, eksenel gerilme, kayma gerilmeleri ve göreli kat ötelemesi gibi gevrek kırılmaya yol açabilecek etkilerle ilgili kısıtlar gözetilir, Eğilme ve kesme donatıları için çözümlemelerde çeşitli asgari oranlar kullanılarak çalışma kapsamında binalar ASCE 41-13'e göre modellenmiş, doğrusaldışı statik itme analizi yapılmış ve beklenen perfomans düzeyleri değerlendirilerek yöntemin güvenilirliği irdelenmiştir.

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LIST OF SYMBOLS

A_0	Effective ground acceleration factor
A_c	Gross cross sectional area of the column section
A_{ce}	Effective cross sectional area of the base columns
A_{ch}	Gross cross sectional area of the wall
A_{ci}	Gross cross sectional area of the column
A_{ck}	Area of the confined concrete core
A_{ft}	Total floor area above the base of the building
A_{oi}	Column tributary area
A_{pi}	Floor area
A_{pt}	Plan area of the first story level
A_{sh}	Total cross sectional area of ties and cross ties
A_{sw}	Total cross sectional area of tie legs for a specified direction
A_{tf}	Total floor area of the building
A_{w}	Total area of infill walls
A_{wi}	Wall area
A_{wt}	Effective cross sectional area of the walls in a given direction
b	Smaller of the column cross sectional dimensions
b_k	Dimension of the confined core depending on the direction
b_{max}	Larger of the column dimensions
b_w	Web width of the beam
C_0	Modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the MDOF system.
C_1	Modification factor to relate the expected maximum inelastic displacements to the displacements calculated for linear elastic response
C_2	Modification factor tto represent the effect of pinched hysteresis shape, cyclic degradation, and strength degradation on the maximum displacement response
C_A	Multiplier involving architectural features
C_M	Multiplier involving construction quality

d	Effective height of the cross section
E	Young's modulus
E_C	Young's modulus of concrete
f_{ck}	Characteristic concrete compressive strength
f_{ctd}	Design tensile strength of concrete
f_{ctk}	Characteristic concrete compressive strength
f_i	Coefficients reflecting structural irregularities
f_{ywd}	Design yield strength of ties
f_{ywk}	Characteristic yield strength of the ties
g	Dead load (*page 22*), gravitational acceleration (*page 37*)
h	Larger of the column cross sectional dimensions
H	Story height
H_{ij}	The vertical distance between i^{th} and j^{th} diaphragm
I	Structure importance factor (*page 20*), moment of inertia (*page 29*)
I_g	Gross moment of inertia of the section
K_e	Effective lateral stiffness
K_i	Elastic lateral stiffness
l_w	Length of the wall cross section
n	Live load reduction factor
N	Number of stories
N_{d1}	Axial load acting on the column
N_{d2}	Axial load obtained from estimated loads
P_d	Design load
P_i	Evaluation scores for different failure modes
P_{min}	The minimum evaluation score
P_w	Weighted score
q	Live load
$R_a(T)$	Behaviour factor of the structural system
s	Tie spacing
$S(T)$	Spectrum coefficient
S_a	Spectral acceleration
S_d	Spectral displacement
t	Thickness of the member

T	Fundamental period of vibration
T_e	Effective fundamental period of vibration
T_i	Elastic fundamental period of vibration
Ux	Mass participation ratio of the fundamental mode in X direction
Uy	Mass participation ratio of the fundamental mode in Y direction
V_c	Contribution of concrete to the shear strength of the member
V_{code}	Required base shear as per the code
V_{cr}	Cracking strength of the column subjected to shear
V_d	Base shear force
V_r	Shear capacity of the member
V_w	Contribution of transverse reinforcement to the shear strength
V_y	Yield base shear of bare frame (*page 12*), effective yield strength (*page 36*)
V_{yw}	Yield base shear of the infilled frame
w	Uniform distributed load per area
W	Seismic weight of the building
w_i	Weighting factor
α	Correction factor
α_1	A parameter for describing positive post-yield slope
α_2	A parameter for describing negative post-yield slope
β	Correction factor
δ	Lateral displacement
δ_{avg}	Average displacement of a given diaphragm level
δ_{ij}	Relative lateral displacement of the two ends of a vertical member
δ_{max}	Maximum displacement at any point for a given diaphragm leve
Δ_d	Greater of the target displacement or the displacement corresponding to the maximum base shear
δ_t	Target displacement
Δ_y	Effective yield displacement
η	Torsional amplification multiplier
μ_{max}	A parameter measuring system degradation

μ_{strength}	A parameter measuring the extent of the nonlinearity
ρ_h	Transverse reinforcement ratio of the wall
ρl	Longitudinal reinforcement ratio
ρs_h	Transverse reinforcement ratio of the wall web
ρt	Ratio of the longitudinal reinforcement in the boundary region

LIST OF ACRONYMS/ABBREVIATONS

ASCE	American Society of Civil Engineers
BCPI	Basic Capacity Index
CI	Column Index
CP	Collapse Prevention
CPI	Capacity Index
FEMA	Federal Emergency Management Agency
IO	Immediate Occupancy
IOPC	Immediate Occupancy Performance Classification
LS	Life Safety
LSPC	Life Safety Performance Classification
MDOF	Multi Degree of Freedom System
MNLSI	Minimum Lateral Stiffness Index
MNLSTFI	Minimum Lateral Strength Index
NSP	Nonlinear Static Procedure
OR	Overhang Ratio
PGV	Peak Ground Acceleration
PI	Priority Index
SDOF	Single Degree of Freedom System
SSI	Soft Story Index
PGV	Peak Ground Acceleration
TSC	Turkish Seismic Code
WI	Wall Index

1. INTRODUCTION

1.1. General

For the last couple of decades, significant efforts have been devoted to earthquake mitigation studies, and both municipal and national authorities have extensively increased the awareness of the earthquake risks, with increasing population and investments in earthquake-prone areas. Comprehensive studies have been conducted and the codes have been updated, leading generally to more restrictive and/or conservative prescriptions. All such activities, however, do not necessarily lead to the conclusion that expected casualties and monetary losses have been decreased sufficiently to reach ‘acceptable’ levels; on the contrary, the 1992 Erzincan, 1995 Dinar, 1998 Ceyhan, 1999 Marmara, 1999 Düzce, 2002 Afyon Sultandağı, 2003 Bingöl, and 2011 Van earthquakes provide ample and most certainly undesired counter evidence. The main question arising here is: why do earthquakes still cause unacceptable losses? Some answers may be obtained within the scope of earthquake reports, post-earthquake assessment guidelines and rapid seismic performance assessment studies (rapid screening procedures) which generally focus on the statistical implications of the physical parameters of the buildings causing damage and partial or total collapse.

Based on post-earthquake examinations of the buildings, the following factors have been identified as causes of failures and deficiencies: soft stories (often leading to pancake-type collapse), weak stories, short columns, violation of the strong column-weak beam principle, excessive interstory drifts, discontinuity in plan and/or along the height, heavy overhangs, poor concrete quality, lack of redundancy, torsional irregularity, mass irregularity, pounding, liquefaction, improper detailing, debonding of reinforcements (especially in undeformed rebars), and poor construction practice.

Soft story is a type of stiffness irregularity along the height. Most of the times this type of irregularity is observed in buildings located near main streets or busy city centers. The first story often has no infill walls while the higher stories do or the height of the first story is greater than the other stories due to commercial reasons. During an earthquake, the soft story is exposed to excessive deformations while the other stories act as a rigid body system and remain far away from inelastic activity. The concentrated deformation demand in the soft story causes yielding in the columns of that story, thereby the soft story becomes unstable and serves as a mechanism. It is important to emphasize that the soft story phenomenon is almost the most frequent irregularity observed in the collapsed buildings during the past earthquakes.

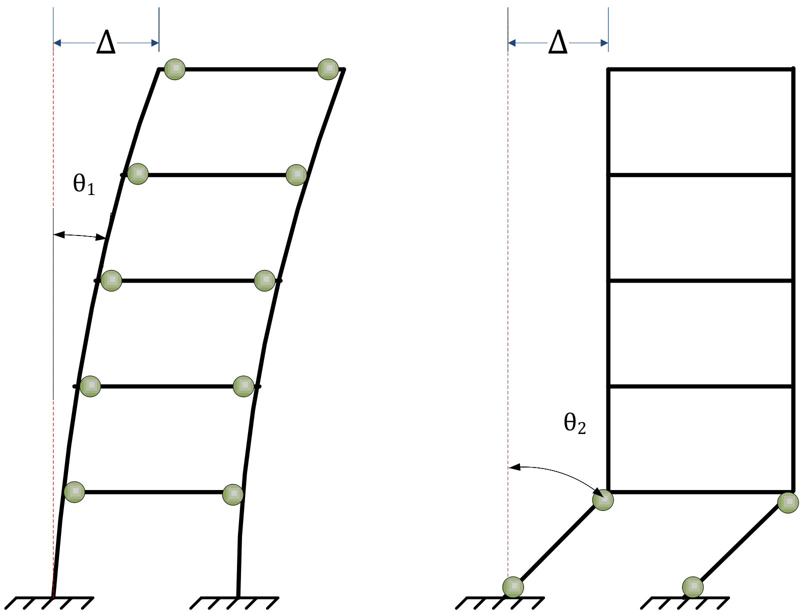

Figure 1.1. Soft story mechanism.

Weak story is very similar to the soft story case except that weak story is a strength irregularity. It is defined as a story having 80% less strength than any adjacent stories. The result of a weak story is the concentration of inelastic activity at that particular floor, which may result in the partial or total collapse of the story.

The term short column or captive column is used when the infill walls are not continuous along the height of the story and the column is partially braced. Lessening the unsupported height of the column causes a dramatic increase (in cubic order) in stiffness of the column. Due to increase in stiffness relative to other columns, captive columns attract large shear forces. Partial infill walls will also develop compression struts with horizontal components that are highly eccentric to the beam column joints. These high shear demands generally will cause non-ductile shear failure and partial collapse in structure. Another point detected in studies is that short or captive columns reduce structural periods, and this reduction generally increases the seismic force demand. In the Bingöl Earthquake, it was observed that the masonry infills accompanying captive columns either had no damage or only vertical narrow cracks near the column faces. This type of behavior is indicative of the brittle behavior of captive columns without exhibiting any ductile deformations.

Strong column-weak beam principle ensures ductile systemic behavior and a stable energy dissipation mechanism. Of particular interest in the strong column-weak beam principle is to enforce that flexural yielding occurs in beams rather than columns. Since beams do not carry axial forces, their energy absorption capacity is much higher than that of the columns. Furthermore, flexural yielding in columns may result in a story mechanism and partial or total collapse.

Excessive interstory drifts can cause second order effects (P-Δ Effect) and result in increased demands. Moreover, high interstory drifts lead to damage in the infill walls, causing an abrupt change of stiffness in the structure. According to the FEMA 306, experiments show that diagonal cracking begins with the onset of inelastic behavior at interstory drift ratios of 0.25% and is essentially complete (from corner to corner) in a panel by about 0.5% drift. It should be kept in mind that non-structural elements such as facades, which are brittle in most cases, are also sensitive to interstory drifts.

Vertical or in-plan irregularities prevent transfer of earthquake loads and results in concentration of forces in a particular region. Heavy overhangs are usually the cantilever balconies, in which case the area of the ground floor is noticeably smaller than the area of a typical intermediate story; the center of the mass of the buildings is therefore shifted in the upward direction (elevated) and the moment arm of the lateral earthquake forces are increased. According to the Middle East Technical University Van Earthquake Report (2011), only 17 of the 95 buildings with overhangs had not experienced any damage in the Van Earthquake of 2011. The remaining 75 buildings suffered various levels of damage, with 21 buildings heavily damaged and 4 that collapsed. When significant torsion exists in the structure due to plan irregularity, additional shear and lateral drift demands are imposed on the vertical elements. Mass irregularity may trigger higher-mode effects and lead to bigger demands than those that would be taken into account if the building is designed considering the first mode only, which is, in fact, the case for most simple low-medium rise structures.

When enough space is not provided between adjacent buildings, pounding may take place due to asynchronous movement of the structures. Pounding may pose a serious risk especially for the building at the end of a series of adjacent structures or when the floor levels of the two adjacent buildings are different.

Poor construction practice, which has been a significant issue especially in rural areas, violates the assumptions and computations that are taken into account during the design process of the building. Improper detailing of reinforcement, which may be partly due to insufficient design and partly due to poor construction practice, may lead to significant damage and even total collapse of the structure in an earthquake.

1.2. Literature Review

Simple methods to enforce designs that would ensure some basic performance level (such as “no collapse”) and to estimate damage for the existing stock are generally developed based on past experiences and building performances observed during earthquakes. There are numerous proposals for fast screening, and some of the most well-known of such methods are described below. The aim here is to highlight the common deficiencies that have been identified by various authors, based on different experiences. It will be observed that such commonly identified deficiencies, or rather the effort to avoid them, play an important role in design procedures.

1.2.1. Rapid Screening Procedures

In recent years, rapid screening procedures has been a research topic in focus, since detailed assessment of existing buildings located in urbanized seismic zones requires substantial amount of time and money. These methods generally focus on detecting those buildings that will perform poorly in an earthquake. For a given building and seismic hazard, basic parameters representing the capacity and the demand are evaluated and calibrated in a statistical way based on building performances observed in past earthquakes. Contribution of each parameter, interaction of parameters and some threshold values for the performance levels are specified in terms of coefficients and equations.

Hassan and Sozen (1997) has noticed a valuable relationship between area of base columns and walls and floor area. These choices may basically be interpreted as simple demand and capacity parameters. While the area of base columns and walls hint at the capacity for base shear (and maybe the embedded stiffness which is correlated with the dimensions) and the floor area represents the seismic weight and corresponding earthquake forces. This might seem a force-based assessment and it is the real case for the structures having poor seismic performances that are expected to respond in a brittle (force controlled) fashion. Two parameters, called the wall index and column index, are defined as follows:

$$WI = \frac{A_{wt}}{A_{ft}} \times 100 \tag{1.1}$$

$$CI = \frac{A_{ce}}{A_{ft}} \times 100 \tag{1.2}$$

where A_{wt} is the effective cross-sectional area of walls in a given horizontal direction, including masonry infill walls with a certain reduction, and A_{ce} is the effective cross sectional area at base. A_{ce} is the half of the total cross sectional area of the base columns due to effect of stiffness on the attraction of relevant shear forces. A_{ft} is the total floor area above the base in a building. The *CI* and *WI* values are added to obtain the priority index (*PI*). The priority index is aimed for use as a descriptor of the priority for removal or strengthening. Erzincan 1992 earthquake was utilized as the database for calibration, and via a plot including the damage states of the buildings experienced in the aforementioned earthquake and their corresponding *CI* and *WI* values, it is observed that an apparent correlation exists between the *CI* and *WI* values and the damage state. As the priority index decreases, the damage on a building is expected to increase.

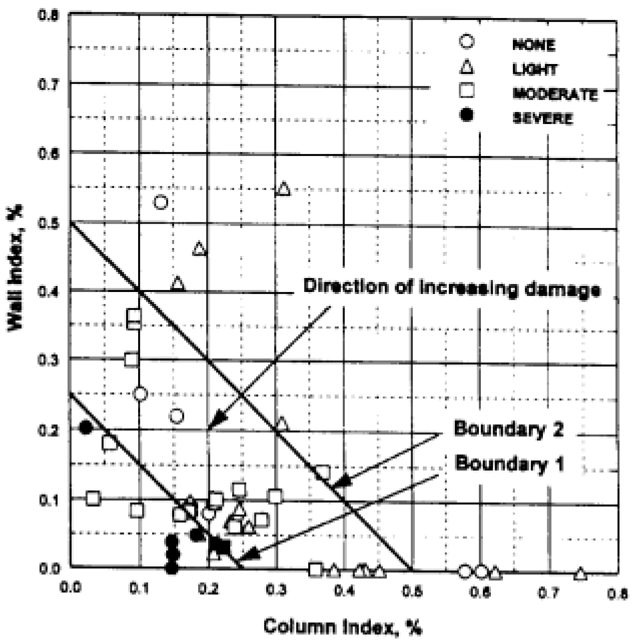

Figure 1.2. Damage states of the buildings and corresponding CI & WI values (Hassan & Sozen, 1997).

This method requires a minimum of information and computation for filtering a large inventory of low-rise monolithic reinforced concrete buildings to identify the fraction that should have priority for remedial action.

Özcebe *et al.* (2003) proposed a method to identify the buildings' expected performance levels by damage scores. The damage scores are estimated by using a discriminant function derived using data from the Düzce 1999 earthquake inventory containing 484 low-to-medium rise reinforced concrete buildings. The damage inducing parameters are; number of stories (*N*), minimum normalized lateral stiffness index (*MNLSTFI*), minimum normal lateral strength index (*MNLSI*), normalized redundancy score (*NRS*), soft story index (*SSI*), and overhang ratio (*OR*). The *MNLSTFI* is the indication of the lateral rigidity of the ground story, which is usually the most critical story. For each direction, total normalized moments of inertias of the vertical structural members are computed and their minimum is taken as the *MNLSTFI*. The *MNLSI* is an indicator for the base shear capacity of the critical story. In the calculation of this index, for each direction, total cross sectional areas of the vertical structural members including masonry infills are computed and their minimum is taken as the *MNLSI*. *NRS* indicates the degree of continuity between multiple frame lines and the structure's ability to distribute lateral forces throughout the structural system. *SSI* is the ratio of the height of first story to the height of the second story and provides a measure for the soft story risk. The overhang ratio *OR* is the summation of the overhang area of each story divided by the area of the ground story. In general, damage states are classified as none, light, moderate, severe, and collapse. Due to limited nature of the Düzce database and concerns regarding the rapid screening procedures, the five distinct damage states are reduced to two expected damage classifications: “The Life Safety Performance Classification” (LSPC) and the “Immediate Occupancy Performance Classification” (IOPC). In the LSPC, none, light, and moderate damage are considered as one group; severe and collapse are considered as another group since the main concern in LSPC is to identify buildings under risk. In the IOPC, none and light damage are considered as one group; moderate, severe, and collapse are considered as another group since the main concern is to identify buildings that can be occupied after a strong ground motion. In the proposed classification procedure, firstly the damage scores are obtained by the two equations below for the cases of LSPC and IOPC, respectively:

$$\begin{aligned} DI_{LS} &= 0.620N - 0.246MNLSTFI - 0.182MNLSI - 0.699NRS + 3.269SSI + 2.728OR - 4.905 \\ DI_{IO} &= 0.808N - 0.334MNLSTFI - 0.107MNLSI - 0.687NRS + 0.508SSI + 3.884OR - 2.868 \end{aligned} \tag{1.3}$$

These damage scores are compared with the number-of-stories dependent cutoff values calculated from the equations below:

$$\begin{aligned} CF_{LS} &= -0.090N^3 + 1.498N^2 - 7.518N + 11.885 \\ CF_{IO} &= -0.085N^3 + 1.416N^2 - 6.951N + 9.979 \end{aligned} \tag{1.4}$$

Based on the comparisons between calculated and cutoff values, the building under evaluation is assigned an indicator variable of “0” or “1”. The indicator variable “0” corresponds to {none, light or moderate damage} in the case of LSPC and {none or light damage} in the case of IOPC. Similarly, the indicator variable “1” corresponds to {severe damage or collapse} in the case of LSPC and {moderate or severe damage or collapse} in the case of IOPC. In the final stage of the classification procedure, the building under evaluation is rated as “safe” or “unsafe” or “intermediate” depending on the values of the indicator variables obtained from both classification types according to the combinations listed in Table 1.1.

Table 1.1. Relationships among different classification criteria (Ozcebe *et.al.*, 2003).

Classification	Indicator Variable		Indicator Variable in Classification
	LSPS	IOPC	
SAFE (None or Light Damage)	0	0	0
UNSAFE (Severe Damage or Collapse)	1	1	1
INTERMEDIATE	1	0	2
INTERMEDIATE	0	1	2

It is essential to say that, number of stories above the ground level (N) is the most significant variable for both LSPC and IOPC. In the case of IOPC, normalized redundancy score (NRS) is the second best discriminant variable.

Yakut *et al.* (2003) developed a method to apply the aforementioned approach by Özcebe *et al.* (2003), to sites having different soil and distance-to-source characteristics than Düzce. The spectral displacement S_d is selected as the damage inducing ground motion parameter. According to the exponential change of the damage with the spectral displacement, the cutoff relations of Equation 1.4 are modified. The spectral displacement is computed by available attenuation relations. The method relies on several stated assumptions:

- The earthquake magnitude in the region to which the method is applied is similar to the one that affected the reference site, i.e. Düzce.
- Attenuation relations are believed to represent the variation of the ground motion adequately.
- Construction practice does not show regional variations.
- Damage pattern observed in the reference site would be the same for the other sites that have same distance to source and soil type.

The method is implemented beginning with the site-specific response spectra using an appropriate attenuation model. Most of the attenuation models (except the new generation attenuation models) require three parameters as input, which are distance, shear wave velocity (representing soil type) and earthquake magnitude. The magnitude is already assumed as the magnitude of the Düzce earthquake and hence the remaining unknowns are the distance and the shear wave velocity. If the attenuation model does not consider the soil type and yields a response spectrum for rock, appropriate amplification functions can be utilized to obtain site specific response spectra. The fundamental period T of interest is obtained from Table 1.2 based on the number of stories N. From the period T and spectral acceleration S_a, the spectral displacement S_d is computed and employed in Equation 1.5 to obtain the damage index (cutoff value) by assuming an exponential relation. All damage indexes at different sites and distances with the damage index obtained for the reference site, i.e. Düzce. As a last step, Düzce cutoff values are modified by multiplying the cutoff modification coefficients, i.e. normalized values.

$$CV = \left[\frac{1}{1 - e^{-S_d/N}} \right] \tag{1.5}$$

Table 1.2. Period vs. number of stories for Düzce seismic damage database.

Number of Stories	Period (sec.)
2	0.275
3	0.355
4	0.433
5	0.504
6	0.529

Sucuoğlu and Yazgan (2003) proposed a two-level risk assessment procedure. The first level is based on recording building parameters by a street survey. In the second level, these are extended with structural parameters measured by entering the building and investigating the ground story. Statistical correlations are obtained by employing a database of 477 damaged buildings surveyed after the 1999 Düzce Earthquake. The proposed method shows some similarities with FEMA 154, but requires more parameters than FEMA 154 does. The method is initiated by assigning a base score (initial vulnerability score) to each building by taking into account the number of stories and the seismic zone, since in the 1999 Kocaeli and Düzce Earthquakes, the damage observed in buildings was strongly correlated with the number of stories. This is especially the case for buildings that are non-conforming to modern seismic design codes. The seismic forces increase linearly with the number of stories, while in non-conforming buildings the resistance does not follow a proportional trend. For both of the assessment levels, the assigned base score is then penalized based upon certain parameters (vulnerability parameters) representing deficiencies or drawbacks. For the Level 1 survey (street survey), only qualitative parameters are required, such as number of stories above the ground, presence of a soft story (yes or no), presence of heavy overhangs such as balconies with concrete parapets (yes or no), apparent building quality (good, moderate, poor), presence of short columns (yes or no), possible pounding between adjacent buildings (yes or no), local soil conditions (stiff or soft), topographic effects (yes or no). For the Level 2 survey process, in addition the parameters used in the first level, 2 quantitative parameters and 1 qualitative parameter are required: plan irregularity (yes or no), redundancy, and strength index. The corresponding parameters are given in Tables 1.3 through 1.5.

Table 1.3. Initial vulnerability scores and the vulnerability parameters for Level 1 Survey (Sucuoğlu and Yazgan, 2003).

Story #	Zone-I 60<PGV	Zone-II 40<PGV	Zone-III 20<PGV	Soft Storey	Heavy Overhang	Apparent Quality	Short Column	Pounding	Topographic Effects
1,2	90	125	160	0	-5	-5	-5	0	0
3	90	125	160	-10	-10	-10	-5	-2	0
4	80	100	130	-15	-10	-10	-5	-3	-2
5	80	90	115	-15	-15	-15	-5	-3	-2
6,7	70	80	95	-20	-15	-15	-5	-3	-2

Table 1.4. Initial vulnerability scores for Level 2 Survey (Sucuoğlu & Yazgan, 2003).

Story #	Zone-I 60<PGV	Zone-II 40<PGV	Zone-III 20<PGV
1,2	95	130	170
3	90	125	160
4	90	115	145
5	90	105	130
6,7	80	90	105

Table 1.5. Vulnerability parameters for Level 2 Survey (Sucuoğlu & Yazgan, 2003).

Soft Storey	Heavy Overhang	Apparent Quality	Short Column	Pounding	Topographic Effects	Plan Irregularities	Redundancy	Strength Index
0	-5	-5	-5	0	0	0	0	-5
-10	-5	-10	-5	-2	0	-2	0	-5
-15	-10	-10	-5	-3	-2	-2	-5	-5
-15	-15	-15	-5	-3	-2	-5	-10	-10
-20	-15	-15	-5	-3	-2	-5	-10	-10

Yakut (2004) proposed a preliminary assessment procedure for existing reinforced concrete buildings. A capacity index is computed considering the orientation, size and material properties of the components comprising the lateral load resisting structural system. This index is then modified by several coefficients that reflect the quality of workmanship and materials, and architectural features. The procedure has been tested and calibrated based on the data compiled from damage surveys conducted after the earthquakes that occurred within the last decade in Turkey. As the first step, the total concrete shear capacity of the ground story columns and shear walls are computed by incorporating the effect of orientation of the members. The total concrete shear capacity obtained in the preceding step is then employed in Equation 1.6 to estimate the yield base shear of bare frame, V_y (without infill walls) where N is the number of stories:

$$V_y = \frac{V_c}{0.95e^{0.125N}} \tag{1.6}$$

Equation 1.6 is derived based on pushover analyses of forty low and medium rise reinforced concrete buildings as per FEMA 273 and the Turkish Design Code. In a general sense, it is accepted that infill walls increase the lateral load capacity of the buildings and hence the relation between yield base shear of the bare frame V_y and the yield base shear of the infilled frame V_{yw} is established through Equation 1.7 where A_w stands for the total area of infill walls and A_{tf} is the total floor area of the building:

$$V_{yw} = V_y\left(46\frac{A_w}{A_{tf}} + 1\right) \tag{1.7}$$

The code required base shear V_{code} is computed as per the seismic design code and the Basic Capacity Index (BCPI) is computed as shown in Equation 1.8:

$$BCPI = \frac{V_{yw}}{V_{code}} \tag{1.8}$$

The BCPI, which is also called the yield overstrength ratio, is then multiplied by C_A and C_M to obtain the Capacity Index (CPI). The C_A and C_M can be considered as penalties arising from architectural features and construction quality, respectively. The last step is to identify the building either “safe” or “unsafe”. A demarcation line or cutoff value can be determined to do so, and case specific cutoff values are recommended to be more reliable. It is stated that the proposed method is more suitable when a group of buildings need to be rapidly evaluated rather than a single building, so the CPI limit becomes specific to the available inventory.

Bal *et al.* (2007) conducted a study referred to as the "*P25 Method*". The proposed method initially requires the determination of the critical story, as the other rapid screening methods also do. If an obvious single critical story does not exist, the calculations are done for each candidate that is suspected to be the critical story. The method is primarily based on 7 particular evaluation scores P_1, P_2,…, P_7, representing *7* different failure modes and their interactions. The basic score P_1 is obtained as per Equation 1.9:

$$P_1 = P_0\left(\prod_{i=1}^{14} f_i\right) \tag{1.9}$$

The P_0 is the structural system score and essentially involves the total height of the building, the resultant effective areas and moments of inertias of the columns, shear walls, and infill walls regarding the critical story. The f_i values are coefficients that reflect structural irregularities, considering the torsional irregularity, slab discontinuity, vertical discontinuity, distribution of mass, corrosion, heavy facade elements, mezzanine floors, unequal levels of floor, concrete quality, strong column-weak beam criterion, spacing of transverse reinforcement (ties), soil type, foundation type, and depth of the foundation. The short column score P_2 score is a function of the ratio of the clear height of the column to the story height. The soft and weak story score P_3 not only considers the relative areas and moment of inertias of the vertical elements (columns, shear walls, and infill walls) of the critical story and the story above, but also considers the relative height of the critical story and the story above. The frame discontinuity score P_4 stands for the heavy overhangs and corresponding frame discontinuities. The pounding score P_5 is elaborated with conditions depending on whether the pounding is eccentric or concentric, the floor levels of the two adjacent buildings, the relative heights and stiffness of the adjacent buildings and the location of the building in the case of buildings lined up in a row. P_6 and P_7 are soil failure scores intending to identify the risks arising from liquefaction and various soil movement types. The minimum score, P_{min} , is the smallest evaluation score among the P_i values, and the final score is determined as follows:

$$P = \alpha \beta P_{\min} \tag{1.10}$$

Here, α is a correction factor expressed as a function of the structure importance factor, effective ground acceleration, live load reduction factor and topographic effects. The correction factor β represents the possibility of interactions among various parameters and is calculated by considering the weighted interaction among seven scores from P_1 to P_7. The minimum of these seven scores is considered as P_{min} and the corresponding weight is assumed as w=4. The weighting factors for other scores are shown in Table 1.6. The weighted score P_w, is then calculated as

$$\mathrm{P}_w = \sum(w_i P_i) / \sum w_i \tag{1.11}$$

The interaction correction factor, β, represents the degree of interaction and the possibility of triggering an interactive failure and is recommended, based on the value of the weighted score, P_w.

Table 1.6. Weighting factors for P1-P7 (Bal *et al.*, 2007).

Weighting Factor	P_1	P_2	P_3	P_4	P_5	P_6	P_7	P_{min}
w	4	1	3	2	1	3	2	4

If the particular interest is to identify the buildings posing collapse risk, due to some uncertainties in the parameters, the authors recommend to utilize a safety bandwidth (for example between 25-35) rather than a single certain value.

Seismic evaluation guidelines generally also cover rapid screening methods. FEMA 154 proposed a method to identify buildings that are potentially seismically hazardous, comprising parameters that can be determined by a "*sidewalk survey*". An initial basic score is assigned to the building under consideration in terms of the seismicity level (high, moderate, or low) and the type of the lateral load resisting system (structural system type). 15 types of lateral load resisting systems are defined in the method. The basic score is, then, either increased or decreased (penalized) by score modifiers based on the following parameters: number of stories, vertical irregularity, plan irregularity, pre-code, post-benchmark, and soil type. Examples of vertical irregularity include buildings with setbacks, hillside buildings, soft story, and short column. Examples of plan irregularity are re-entrant corners as in E, L, T, U, or + shaped buildings, and torsional irregularity. The term *"Pre-Code"* herein is applicable for buildings located in moderate and high seismicity regions. The pre-code score modifier applies to buildings which were designed and constructed prior to the initial adoption and enforcement of seismic codes applicable for that building type (e.g. concrete moment-resisting frame, C1). The post-benchmark score modifier is applicable if the building being screened was designed and constructed after significantly improved seismic codes applicable for that building type. The year in which these improvements were adopted is termed the "benchmark" year. The score modifier for soil type is applicable for soil types C, D, and E. A representative part of the data collection form which includes the basic score and the score modifiers is provided in Figure 1.3.

OCCUPANCY			SOIL	TYPE						FALLING HAZARDS			
~~Assembly~~ Commercial Emer. Services	Govt Historic Industrial	Office Residential School	Number of Persons ~~0 – 10~~ 11 – 100 101-1000 1000+	A Hard Rock	B Avg. Rock	C Dense Soil	D Stiff Soil	E Soft Soil	F Poor Soil	☐ Unreinforced Chimneys	☐ Parapets	☐ Cladding	☒ Other: Cornices

BASIC SCORE, MODIFIERS, AND FINAL SCORE, *S*

BUILDING TYPE	W1	W2	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RC SW)	S5 (URM INF)	C1 (MRF)	C2 (SW)	C3 (URM INF)	PC1 (TU)	PC2	RM1 (FD)	RM2 (RD)	URM
Basic Score	**4.4**	**3.8**	**2.8**	**3.0**	**3.2**	**2.8**	**2.0**	**2.5**	**2.8**	**1.6**	**2.6**	**2.4**	**2.8**	**2.8**	**1.8**
Mid Rise (4 to 7 stories)	N/A	N/A	+0.2	+0.4	N/A	+0.4	+0.4	+0.4	+0.4	+0.2	N/A	+0.2	+0.4	+0.4	0.0
High Rise (> 7 stories)	N/A	N/A	+0.6	+0.8	N/A	+0.8	+0.8	+0.6	+0.8	+0.3	N/A	+0.4	N/A	+0.6	N/A
Vertical Irregularity	-2.5	-2.0	-1.0	-1.5	N/A	-1.0	-1.0	-1.5	-1.0	-1.0	N/A	-1.0	-1.0	-1.0	-1.0
Plan irregularity	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
Pre-Code	0.0	-1.0	-1.0	-0.8	-0.6	-0.8	-0.2	-1.2	-1.0	-0.2	-0.8	-0.8	-1.0	-0.8	-0.2
Post-Benchmark	+2.4	+2.4	+1.4	+1.4	N/A	+1.6	N/A	+1.4	+2.4	N/A	+2.4	N/A	+2.8	+2.6	N/A
Soil Type C	0.0	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4
Soil Type D	0.0	-0.8	-0.6	-0.6	-0.6	-0.6	-0.4	-0.6	-0.6	-0.4	-0.6	-0.6	-0.6	-0.6	-0.6
Soil Type E	0.0	-0.8	-1.2	-1.2	-1.0	-1.2	-0.8	-1.2	-0.8	-0.8	-0.4	-1.2	-0.4	-0.6	-0.8
FINAL SCORE, *S*										0.5					

COMMENTS

Detailed Evaluation Required **YES** **NO**

* = Estimated, subjective, or unreliable data
DNK = Do Not Know
BR = Braced frame
FD = Flexible diaphragm
LM = Light metal
MRF = Moment-resisting frame
RC = Reinforced concrete
RD = Rigid diaphragm
SW = Shear wall
TU = Tilt up
URM INF = Unreinforced masonry infill

Figure 1.3. Representative example of the basic score and the score modifiers.

2. SIMPLE DESIGN METHODOLOGY

2.1. Overview

Ersoy (2013) proposed a simple approach for the preliminary design of reinforced concrete buildings to be built in seismic regions. Investigation of the buildings exposed to the recent earthquakes have shown that, structural damage in residential or office type reinforced concrete buildings with 2-8 stories is considerably more common than the other types. Among those buildings, the ones experiencing the pancake collapse mechanism can be considered as the main reason of casualties. The author intends to prevent such defects posing a serious jeopardy for human lives by deriving simple expressions relying on the essential rules of structural engineering, such as limiting the interstory drifts, ensuring member and systems ductilities, and avoiding shear failures. The method does not permit structural systems without shear walls. However, structural systems composed solely of frames are also evaluated within the scope of the study in the following chapters.

2.2. Assumptions and Limitations

The assumptions and limitations within the scope of the proposed method are:

- The concrete compressive strength is 20 MPa.
- For the longitudinal and transverse reinforcement, structural steel class is S420.
- The sum of the self-weight of the building and the live load is 10 kN/m^2. The factored loads are 15 kN/m^2, and 10 kN/m^2 for the cases of pure gravity loads and seismic loads, respectively. The live load reduction is not applied so as to be on the safe side.
- For the transverse reinforcement in the column ends:
    i. For column sections with $h \leq 400$ mm, at least Ø8/100 mm ties and Ø8/100 mm cross-ties in each direction are used.
    ii. For column sections with $h > 400$ mm, at least Ø10/100 mm ties and Ø10/100 mm cross-ties in each direction are used.
- For the transverse reinforcement along the intermediate height of the column :
    i. For column sections with $h \leq 400$ mm, at least Ø8/200 mm ties,

ii. For column sections with $h > 400$ mm, at least Ø10/200 mm ties are used, where h is the larger of the column dimensions along the two orthogonal axes of the column section.

- Enough length for the lap splicing is provided.
- Minimum cross sectional dimensions for beams:

$$b_w \geq 250\,\text{mm}$$

$$h \geq 300\,\text{mm},\ h \geq 3t$$

where, b is the width and h is depth of the beam, and t is the slab thickness.

- For the beam ends, the maximum tie spacing is $d/4$ along a distance of $2h$ from the column face. Along the intermediate span of the beam, maximum tie spacing is $d/2$, where d is the effective beam depth.
- Minimum thickness for structural walls are 200 mm.
- For each of the total horizontal and the total vertical web reinforcement of the structural wall is no less than 0.0025 times the gross web area, and the tie spacing is no more than 250 mm.
- When the wall cross section is considered, boundary regions (confinement zones) are provided for each end. Throughout the boundary regions, the minimum longitudinal reinforcement is 4Ø16 and $\rho_t \geq 0.001 t l_w$. For the confinement, a minimum of Ø8/150 transverse reinforcement is provided.

  i. ρ_t = the ratio of the longitudinal reinforcement of the boundary region

  ii. t = wall thickness

  iii. l_w = length of the wall cross section

- The expressions are derived for the seismic zone-I, and seismic zone-II (the terms used for high seismicity regions in Turkish Seismic Code), but the method can be expanded to other seismic zones provided that required modifications are established.

2.3. Determining the Member Dimensions

2.3.1. Columns

The axial stresses in the columns are intended to be kept under a certain level as defined in Equation 2.1:

$$N_{d1} \geq 0.5 f_{ck} A_{ci} \tag{2.1}$$

where N_{d1} is the axial load acting on the column, f_{ck} is the characteristic concrete compressive strength, and A_{ci} is the gross cross sectional area of the column. It is assumed that each column is subjected to axial loads proportional to their tributary areas and the estimated design loads as mentioned earlier.

$$N_{d2} \geq P_d \sum A_{oi} \tag{2.2}$$

where N_{d2} is the axial force obtained by the estimated loads, P_d, is the design load defined in the previous section (15 kN/m^2), and $\sum A_{oi}$ is the total column tributary area covering all stories above the column. Equating N_{d1} to N_{d2} and assuming f_{ck} =20000 kN/m^2, the minimum allowable column cross section can be determined:

$$A_{ci} \geq 0.0015 \sum A_{oi} \tag{2.3}$$

Shear capacity of the columns must also be considered in determining the column sizes. It is assumed that sufficient transverse reinforcement is provided for the intermediate height of the column referred as in Section 2.2. Although in the proposed method, buildings without shear walls are not permitted, according to the author the columns must be able to resist 30% of the total base shear corresponding to earthquake forces:

$$\begin{aligned} V_r &= V_c + V_w \\ V_{cr} &= 0.65 f_{ctd} A_{ci} \\ V_c &= 0.8 V_{cr} \\ V_w &= (A_{sw} / s) f_{ywd} d \end{aligned} \tag{2.4}$$

where

V_r is the shear capacity of the column

V_{cr} is the cracking strength of the column subjected to shear

V_c is the contribution of the concrete to the shear strength of the column

f_{ctd} is the design concrete tensile strength

f_{ywd} is the design yield strength of the ties

A_{sw} is the total cross sectional area of the tie legs for the direction under consideration

s is the tie spacing

h is the the larger side of the column cross section

b is the the smaller side of the column cross section

d is the effective height of the column cross section (compatible for the direction of the shear)

Adopting f_{ctd} = 1100 kN/m^2 and f_{ywd} =365 000 kN/m^2, the equations below are obtained for the shear capacity of the column.

$$\begin{aligned} V_{cr} &= 715 A_{ci} \\ V_c &= 572 A_{ci} \\ V_w &= 365\ 000 (A_{sw} / s) d \\ V_r &= 572 A_{ci} + 365\ 000 (A_{sw} / s) d \end{aligned} \tag{2.5}$$

For various column sections, shear strength values are computed regarding to the assumptions in Section 2.2. Besides, in order to obtain the minimum shear capacity, d is considered as the smaller dimension of the column cross section (b - 40 mm). Note that according to the Turkish Seismic Code (2007), the smaller column side shall not be less than 300 mm. It is observed that, as long as the shear reinforcement is in compliance with the assumptions in Section 2.2 for a column member, the ratio of the shear capacity to the cracking strength is greater than 1 for all cases and is no less than 1.35. The below relationship is derived if $V_r / V_{cr} = 1.35$ is adopted.

$$V_r = 1.35 x 715 A_{ci} = 965 \sum A_{ci} \tag{2.6}$$

where $\sum A_{ci}$ is the total cross sectional area of the columns located in the i^{th} story. The total base shear prescribed in the Turkish Seismic Code (2007) is:

$$\begin{aligned} \sum V_d &= WA(\mathrm{T}_1) / R_a(T_1) \\ A(T_1) &= A_o IS(T_1) \\ W &= w \sum A_{pi} \end{aligned} \tag{2.7}$$

where

$\sum V_d$ is the total base shear

W is the seismic weight of the building, and w is the load per unit area

$\sum A_{pi}$ is the the total floor area of the building (all the stories are considered)

A_0 is the effective ground acceleration factor

I is the structure importance factor

$S(T)$ is the spectrum coefficient

$R_a(T)$ is the behavior factor of the structural system (related with the ductility)

Assigning w=10 kN/m^2, A_0=0.4, $S(T)$=2.5, and $R_a(T)$=4, the total base shear acting through the building is computed as:

$$\sum V_d = 2.5 \sum A_{pi} \tag{2.8}$$

As mentioned earlier the structural walls are expected to exist within the scope of the proposed method and assumed to take 100% of the total base shear. Despite that, the columns are assumed to receive 30% of the total base shear and hence the total shear capacity of the base columns must resist 30% of the total base shear. In order to be on the safe side, this ratio is increased up to 50%. For a column with a cross sectional area A_{ci}, $\sum A_{pi}$ is replaced by the total column tributary area $\sum A_{oi}$ and Equation 2.6 is equated to the 50% of the Equation 2.8, the required minimum column cross sectional area is obtained:

$$A_{ci} \geq 0.0013 \sum A_{oi} \tag{2.9}$$

Equation 2.9 is very similar to Equation 2.3, which is derived from the axial stress restriction as per Turkish Seismic Code (2007). The greater of these expressions is adopted for the minimum column cross sectional area. Another restriction about the column cross sectional area is taken as:

$$A_{ci} \geq 0.09 \text{ m}^2 \tag{2.10}$$

Another, and the last, restriction for the columns are the ratio of the column edges:

$$h / b \leq 2 \tag{2.11}$$

2.3.2. Structural Walls

The structural walls are assumed to receive the entire total base shear. The shear strength of the structural walls is a well accepted indicative of ductility of the shear walls. Based on this fact, the shear strength was selected as the key factor in determining the wall sizes. The expressions are derived in terms of the limit state approach, and hence material characteristic strength values are adopted. The wall thickness is assumed to be 0.2 m and the effective wall cross sectional area $A_{wi} = t \text{ x } d$. The governing equations are:

$$\begin{aligned} V_r &= V_{cr} + V_w \\ V_{cr} &= 0.65\, f_{ctk} A_{wi} \\ V_w &= \rho_h f_{ywk} A_{wi} \end{aligned} \tag{2.12}$$

where ρ_h is the ratio of the transverse (horizontal) reinforcement, taken as 0.0025. The other terms are assigned as, $f_{ctk} = 1600$ kN/m^2, $f_{ywk} = 420\ 000$ kN/m^2, such that:

$$V_r = 1040A_{wi} + 1050A_{wi} = 2090A_{wi} \tag{2.13}$$

For the strong directions of the shear walls located in the story under consideration, A_{wi} is replaced by $\sum A_{wi}$ in Equation 2.13, and the total shear strength is obtained as follows:

$$V_r = 2090\sum A_{wi} \tag{2.14}$$

The total base shear computed in Equation 2.8 is equated to the total shear capacity of the story (Equation 2.14), the minimum wall area per direction required to resist the seismic forces for a specific story can be obtained in terms of the percentage of the total floor area of the building:

$$\sum A_{wi} \geq 0.0012\sum A_{pi} \tag{2.15}$$

The structural walls should be placed so that torsional irregularity is prevented. In case the above equation does not provide sufficient wall area for lateral stiffness when low-rise buildings are of concern, an additional constraint is introduced in terms of the base floor area as shown in Equation 2.16:

$$\sum A_{wi} \geq 0.004A_{pt} \tag{2.16}$$

Both Equation 2.15 and Equation 2.16 shall be satisfied in each principal direction. Although it is believed that the above conditions ensure adequate strength and stiffness, for some “unexpected” or “special” cases the sum of the column and wall cross sectional areas must be assigned a lower limit as follows:

$$\left(\sum A_{ci} + \sum A_{wi}\right) \geq 0.0020\sum A_{pi} \tag{2.17}$$

3. MODIFIED SIMPLE DESIGN METHOD

3.1. Overview

This chapter presents a description of the modifications implemented to the original method summarized in Chapter 2. Within the scope of this thesis, the extent of the method was aimed to broaden to structures those without structural walls. Besides, various parameters including the axial stress limit, $R_a(T_1)$, concrete class, and some other demand parameters were modified. Thereafter, the modified method was evaluated via nonlinear static procedure (pushover analysis) as per ASCE 41-13 to gain insight into the applicability and boundaries of the method. The structures are classified into two categories: frame systems and dual systems. Frame systems consist solely of columns and beams, whereas dual systems are composed of frames and structural walls.

3.1. Assumptions and Limitations

- The concrete compressive strength is 25 MPa.
- For the longitudinal and transverse reinforcement, structural steel class is S420.
- Self-weight of the building and live load per unit area, referred to as g and q, respectively, are expected to be approximately equal to the values used in the analyses, with g =7 kN/m^2 and q =3 kN/m^2 are assigned. The loads are not factored when computing the required column area for limiting axial stresses.
- For the transverse reinforcement in the column ends, the minimum transverse reinforcement ratio prescribed in Turkish Seismic Code (2007) is adopted as shown in Equation 3.1:

$$A_{sh} \geq 0.3sb_k\left[\left(A_c / A_{ck}\right)-1\right]\left(f_{ck} / f_{ywk}\right) \rightarrow A_{sh} \geq 0.075sb_k\left(f_{ck} / f_{ywk}\right) \quad (3.1)$$

Here,

b_k = dimension of the confined core cross section of the column considered for each direction (measured from centerline of the tie to the centerline of the tie).

A_{sh} = the sum of the cross sectional area of the ties and cross ties running in the direction perpendicular to b_k.

A_c = gross cross sectional area of the column section.

A_{ck} = area of the confined core of the column.

With $h > 400$ mm, at least Ø10/100 mm ties and Ø10/100 mm cross-ties in each direction are used.

- For the column sections, $V_r = 1.5V_{cr}$ is adopted. In order to satisfy this condition, the required minimum total number of ties and cross ties running in the corresponding direction is given in Table 3.1 in terms of specified tie spacings and gross section dimensions. The minimum amount of transverse reinforcement given in Table 3.1 is assumed to exist both in the column ends and in along the intermediate height of the column.

Table 3.1. Required total number of ties and crossties.

b_{max} (mm)	Ø8/100	Ø8//150	Ø8/200	Ø10/100	Ø10/150	Ø10/200
300	2	2	2	2	2	2
400	2	2	3	2	2	2
500	2	3	3	2	2	2
600	2	3	4	2	2	3
700	3	4	5	2	2	3
800	3	4	5	2	3	4
900	3	5	6	2	3	4
1000	4	5	6	2	3	4
1200	4	6	8	3	4	5

- For column sections $h / b \leq 2$, where h is the larger of the column dimensions along the two orthogonal axes of the column section. . The theoretical premises behind this condition are: to satisfy the strong column principle in both directions, to prevent higher shear demands in strong directions of the deep columns, and to ensure that the moment inflection point forms in reasonable heights, i.e. the characteristic of the moment diagram is different from those of the shear walls:

- Only solid slab systems were evaluated in the scope of this thesis, and buildings with flexible diaphragms were kept out of scope.
- Sufficient length for lap splicing is provided.
- The longitudinal reinforcement ratio is taken as $\rho_l = 0.01$ for the columns.
- The moment inflection point for a column member is assumed to be located at a distance of *2H/3* from the base of the column, where *H* is the story height. This assumption is generally valid for columns at the base.
- Minimum cross sectional dimensions for beams:

$$b_w \geq 250\,\text{mm}$$

$$h \geq 300\,\text{mm},\ h \geq 3t$$

where, *b* is the width of the beam, *h* is depth of the beam, and *t* is the slab thickness.

- For the beam ends, the maximum tie spacing is $d/4$ along a distance of 2*h* from the column face, where *d* is the effective beam depth. Along the intermediate span of the beam, maximum tie spacing is $d/2$. The required transverse reinforcement for the beam spans will be discussed in the next chapters.
- Minimum thickness for structural walls is 250 mm.
- Each of the total horizontal and the total vertical web reinforcement of the structural wall is no less than 0.0025 times the gross web area, and the tie spacing is no more than 250 mm. The shear check for the wall web will be done later.
- Boundary regions (confinement zones) are provided for each end of the wall sections. Throughout the boundary regions, the minimum longitudinal reinforcement is 4Ø16 and $\rho_t \geq 0.001tl_w$. For the confinement, a minimum of Ø8/150 transverse reinforcement is provided. In these expressions,

  ρ_t = longitudinal reinforcement ratio for the boundary region

  t = wall thickness

  l_w = length of the wall cross section

- The expressions are derived for Seismic Zone I and Seismic Zone II (the terms used for high seismicity regions in Turkish Seismic Code), but the method can be expanded to other seismic zones provided that required modifications are established.

- The maximum number of stories is determined as 8 during this study, and the maximum story height is 3.5 m.

3.2. Determining the Member Dimensions

3.2.1. Frame Systems

This section covers structural systems composed solely of frames. The base columns are assumed to resist the entire (100%) base shear.

The axial stresses in the columns are intended to be kept under a certain level as prescribed be Equation 3.2:

$$N_{d1} \geq 0.3\, f_{ck} A_{ci} \tag{3.2}$$

In this equation, N_{d1} is the axial load acting on the column, f_{ck} is the characteristic concrete compressive strength, and A_{ci} is the gross cross sectional area of the column. It is assumed that each column is subjected to axial loads proportional to their tributary areas and the estimated design loads as mentioned earlier:

$$N_{d2} \geq (g+q)\sum A_{oi} \tag{3.3}$$

where N_{d2} is the axial force obtained by the estimated loads. For the axial stress constraint, live load reduction has not been implemented in order to compensate for the axial loads corresponding to seismic effects. $\sum A_{oi}$ is the total column tributary area, covering all stories above the column. Equating N_{d1} to N_{d2} and taking f_{ck} =25 000 kN/m^2, the minimum required column cross section can be determined:

$$A_{ci} \geq 0.00014(g+q)\sum A_{oi} \tag{3.4}$$

Shear capacity of the columns must also be considered in determining the column sizes. It is assumed that sufficient transverse reinforcement is provided for the intermediate height of the column:

$$V_r = 1.5 V_{cr} \text{ and } V_{cr} = 0.65 f_{ctd} A_{ci} \rightarrow V_r = 1170 A_{ci} \tag{3.5}$$

In these expressions,

V_r = shear capacity of the column

V_{cr} = cracking strength of the column subjected to shear

V_c = contribution of the concrete to the shear strength of the column

f_{ctd} = design concrete tensile strength (1200 kN/m^2)

f_{ywd} = design yield strength of the ties (365 000 kN/m^2)

Similar to Chapter 2, $\sum A_{ci}$ is the total cross sectional area of the columns located on the i^{th} story. The total base shear prescribed in the Turkish Seismic Code (2007) is:

$$\begin{aligned} \sum V_d &= WA(\mathrm{T}_1) / R_a(T_1) \\ A(T_1) &= A_o I S(T_1) \\ W &= (g + nq) \sum A_{pi} \end{aligned} \tag{3.6}$$

where,

$\sum V_d$ = total base shear

W = seismic weight of the building

$\sum A_{pi}$ = the total floor area of the building (all the stories are considered)

A_0 = effective ground acceleration factor (related with the seismicity of the region)

I = structure importance factor (taken as unity for dwellings and office buildings)

$S(T)$ = spectrum coefficient

$R_a(T)$ = behavior factor of the structural system (related with the ductility)

n = live load reduction factor.

Assigning A_0=0.4, $S(T)$=2.5, $R_a(T)$=4 for frame systems and n=0.3 for residential and office buildings, the total base shear acting through the building is computed as:

$$\sum V_d = 0.25(g+0.3q)\sum A_{pi} \tag{3.7}$$

For a column with a cross sectional area A_{ci}, $\sum A_{pi}$ is replaced by the total column tributary area $\sum A_{oi}$ and Equation 3.4.c is equated to the Equation 3.6 to obtain the required minimum column cross sectional area:

$$A_{ci} \geq 0.00022(g+0.3q)\sum A_{oi} \tag{3.8}$$

The area for an individual column is obtained as the greater of the results of Equation 3.3 and Equation 3.7, and adopted for the minimum column cross sectional area. Another restriction for the column cross sectional area is:

$$A_{ci} \geq 0.09 \text{ m}^2 \tag{3.9}$$

The last restriction for the columns is the ratio of the column edges:

$$h/b \leq 2 \tag{3.10}$$

The drift control in frame structures is crucial, especially for those with number of stories above a certain value. To this end, all of the base columns are considered as a single column and the seismic mass of the building is assumed to be lumped at the first floor level. This single degree of freedom system analogy is utilized for drift-restraining expressions. As stated in the Section 3.2, the moment inflection points for the base columns are assumed to form at a distance of 2H/3 from the base. This assumption yields a lateral stiffness value of $\frac{6EI}{H^3}$ as seen in the Figure 3.1.

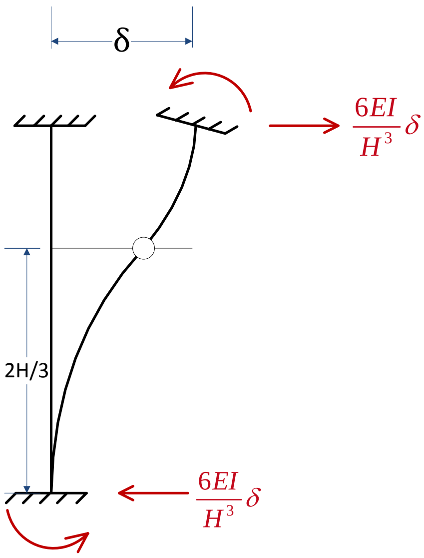

Figure 3.1. Lateral stiffness of a column with 2H/3 inflection point assumption.

The total base shear acting on the base columns is identical to the one obtained from Equation 3.6, except $R_a(T)=1$ is used for drift calculations, leading to Equation 3.10:

$$\sum V_d = (g + 0.3q)\sum A_{pi} \tag{3.11}$$

As seen from the Figure 3.1, the lateral displacement (drift) of the base is:

$$\delta = \sum V_d \frac{H^3}{6EI} \tag{3.12}$$

where E is Young's Modulus of the concrete and taken as 25 000 000 kN/m^2, and I is the second moment of area (moment of inertia) for an uncracked column section for the direction under consideration. The relative interstory drifts are aimed to be limited as is Equation 3.12:

$$\frac{\delta}{H} \le 0.015 \tag{3.13}$$

δ in Equation 3.12 is replaced by the expression in Equation 3.11, and $\sum V_d$ in Equation 3.11 is replaced by the term in Equation 3.10 to get:

$$(g+0.3q)\sum A_{pi}\frac{H^2}{6EI}\leq 0.015 \;\;\rightarrow\;\; \frac{I}{H^2}\geq 4.44\,\mathrm{x}\,10^{-7}(g+0.3q)\sum A_{pi} \tag{3.14}$$

Replacing $\frac{I}{H^2}$ with $\sum\left(\frac{I}{H^2}\right)$ leads to

$$\sum\left(\frac{I}{H^2}\right)\geq 4.44\,\mathrm{x}\,10^{-7}(g+0.3q)\sum A_{pi} \tag{3.15}$$

where $\sum\left(\frac{I}{H^2}\right)$ is the sum of the $\frac{I}{H^2}$ values of individual columns in the base. The theoretical premise behind this assumption is the concentration of seismic demand in the base story. Note that Equation 3.14 must be satisfied for both directions.

3.2.2. Dual Systems (Frame + Structural Walls)

The column sections in dual systems are determined in terms of the following restrictions.

The level of allowed axial stresses in the columns is slightly increased due to the existence of the structural walls:

$$N_{d1}\geq 0.35\,\mathrm{f}_{ck}\mathrm{A}_{ci} \tag{3.16}$$

and Equation 3.4 is modified to:

$$A_{ci}\geq 0.00012\left(g+q\right)\sum A_{oi} \tag{3.17}$$

In dual systems it is assumed that 30% of the total base shear is resisted by columns, and Equation 3.8 is modified to (by setting $R_a\left(T\right)$=3, and taking 30% of the V_d):

$$A_{ci}\geq 0.0001(g+0.3q)\sum A_{oi} \tag{3.18}$$

Equation 3.9 and Equation 3.10 are still valid for the columns of the dual systems.

The individual and the total dimensions of the structural walls are determined in terms of the following restrictions.

The conditions derived in Section 2.3.2 is applied in a similar way, except that $R_a\left(T\right)$=3, and instead of P_d, $\left(g+0.3q\right)$ is used for determining the seismic weight of the structure. Equation 2.15 then evolves to:

$$\sum A_{wi} \geq 0.0002\left(g+0.3q\right)\sum A_{pi} \tag{3.19}$$

The structural walls should be placed so that torsional irregularity is prevented. In case the above equation does not provide sufficient wall area for lateral stiffness when low-rise buildings are of concern, an additional constraint is introduced in terms of the base floor area as shown in Equation 3.20:

$$\sum A_{wi} \geq 0.0007\left(g+0.3q\right)A_{pt} \tag{3.20}$$

Both Equation 3.19 and Equation 3.20 must be satisfied in each principal direction. Although it is believed that the above conditions ensure adequate strength and stiffness, for some “unexpected” or “special” cases the sum of the column and wall cross sectional areas must be assigned a lower limit as follows:

$$\left(\sum A_{ci}+\sum A_{wi}\right) \geq 0.0003\left(g+0.3q\right)\sum A_{pi} \tag{3.21}$$

4. SEISMIC PERFORMANCE ASSESSMENT

Estimating the ‘true’ performance of a structure under seismic loads has been broadly investigated by researchers during the last few decades. Capturing a realistic behavior of a building depends on several parameters which are missing in most of the times. Thus, more complicated analysis methods do not necessarily yield ‘more accurate’ results due to the lack of knowledge required for such sophisticated analyses; there are various issues with plastic hinge characteristics, cracked section rigidities, plastic biaxial bending in columns, moment-rotation backbones, hysteretic rules, hysteretic damping, dynamic instability, rigidity of beam column joints, dynamic characteristics of structural walls, P-Δ effects, analysis of flexible diaphragms, soil-structure interaction, near field effects, etc that must be addressed. Besides these issues, most of which are epistemic uncertainties, the hazard itself is a significant source of aleatory uncertainty. Inherently, the aleatory uncertainties pushed earthquake engineering to “capacity design”, which is a realistic approach in most aspects. Due to the substantial amount of experimental data and analytical knowhow, identifying the capacity is somewhat more reliable than identifying the demand or the hazard. Today’s seismic hazard models require a long time and a vast database to be validated. However, engineers can not wait for hundreds of years for validation and have to select an appropriate model and method of analysis. In this study, the hazard was utilized from the one prescribed in Turkish Seismic Code as 10% probability of exceedance in 50 years, i.e. 475-year return period.

Among the linear and nonlinear analysis, nonlinear static procedure (pushover analysis) was selected as performance assessment method. Although linear analysis methods usually provide sufficient accuracy and safety for design, they can misrepresent the failure modes or the collapse mechanism which are observed in the inelastic range of response. Nonlinear dynamic analysis seems like the ‘most accurate’ method for a given earthquake, yet the hysteretic hinge properties, damping, and especially the selection and scaling of appropriate acceleration time histories are issues still being argued about. Nonlinear static procedure is a good indicative of the behavior and failure modes, but it is not well suited to represent dynamic properties like degradation, dynamic instability, or higher mode effects. Since the buildings under evaluation within the specified scope of this study are so called simple buildings and do not have significant irregularity problems, the nonlinear static procedure was deemed comprehensive enough to provide insight into the response and performance. It should be kept in mind that this choice is questionable and there can always be a ‘better’ method of analysis depending on the nature of the problem.

4.1. Modeling

One of the key elements in determining seismic performance of a structure is the modeling phase. The buildings were modeled as a 3D composition of discrete line elements, which are the beams and columns. The shear walls were also modeled as line elements. The seismic mass was discretized and distributed to the nodes where beams and columns are connected. The base is assumed as fixed and no soil-structure interaction is considered. The beam-column joints were considered as rigid joints and diaphragm constraints were assigned to each floor level since only buildings with solid slab systems were evaluated. The nonlinear hinges were assigned to the member ends as lumped point hinges with no hinge lengths specified. The force displacement relationships and acceptance criteria were implemented as per stipulated in ASCE 41-13.

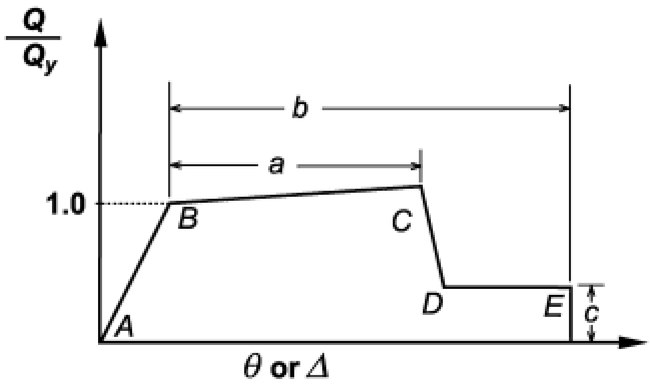

Figure 4.1. Typical load deformation relation from ASCE 41-13.

The strain hardening and softening properties are not implemented to the model and the elastic-perfectly plastic moment-rotation backbone is adopted, but in the post-processing step, the rotations of the members are substantiated to remain within the specified acceptance criteria. In other words, the final plastic rotations are checked to ensure that the plastic rotation value at a hinge was not beyond the rotation limits specified in ASCE 41-13. Another issue is the flexural rigidities of the members with cracked sections. There is an apparent scatter among various studies on the cracked section rigidities. Moreover, during the analyses, it was observed that cracked section rigidities can significantly change the modal responses, and hence corresponding lateral load patterns and the interstory drifts. The cracked section properties adopted from ASCE 41-13 (Table 10.5) are given in Table 4.1.

Table 4.1. Effective stiffness values for RC members as prescribed by ASCE 41-13.

Component	Flexural Rigidity
Beam - nonprestressed	$0.3E_cI_g$
Columns with compression caused by design gravity loads $\geq 0.5A_gf'_c$	$0.7E_cI_g$
Columns with compression caused by design gravity loads $\leq 0.1A_gf'_c$ or in tension	$0.3E_cI_g$
Walls – cracked	$0.5E_cI_g$

Besides, according to the ASCE 41-13, for columns with low axial stresses, deformation caused by bar slip can account for as much as 50% of the total deformations at yield. The effective flexural rigidity values for beams and columns account for the additional flexibility from reinforcement slip within the beam-column joint or foundation before yielding. In ASCE 41-13, component stiffness is generally taken as the effective stiffness based on the secant stiffness to yield-level forces. The cracked section modifiers are constant for the structural walls and the beams, but for columns the level of axial stress determines the effective stiffness values. As the column sizes increase up to a certain level, if the axial load is kept constant, the reduction of stiffness increases. Every single column in the model is assigned a stiffness modifier regarding the level of axial stress.

4.2. Nonlinear Static Procedure (Single-Mode Pushover)

In the nonlinear static procedure (NSP), the target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. The applicability of the NSP depends on two conditions. The first condition is prescribed to ensure the higher mode effects are not significant: it is stated that, a modal response spectrum analysis shall be performed for the structure to produce 90% mass participation. A second response spectrum analysis shall be performed, considering only the first mode participation (here the first mode implies that the governing mode used for the NSP). Higher mode effects shall be deemed significant if the shear in any story resulting from the modal analysis considering the modes required to obtain 90% mass participation exceeds 130% of the corresponding story shear considering only the first mode response. If higher mode effects are found to be significant, NSP is only permitted if a linear dynamic analysis is performed to supplement it. This condition was checked for the base shear forces, not for the individual stories, and it was concluded that the order of the ratio of the forces obtained from the two response spectrum analyses were generally far behind the limits. Therefore, higher mode effects are considered to be insignificant.

The second condition that should be satisfied for the applicability of NSP is related with the strength degradation and dynamic instability. The NSP is applicable as long as $\mu_{strength} < \mu_{\max}$; nonlinear dynamic analysis is required to confirm dynamic stability otherwise. The strength ratio $\mu_{strength}$ is a measure of the extent of the nonlinearity, and $\mu_{\max}$ is a measure of the system degradation.

The effect of second order moments (P-Δ effects) are required to be included in the analysis. Static P-Δ effects are caused by gravity loads acting through the deformed configuration of the system. Dynamic P-Δ effects are caused by a negative post-yield stiffness that increases story drift and the target displacement. In the analysis, P-Δ effects were not taken into consideration, since the software utilized had convergence issues, as stated also in a previous study by Welt (2010). The exclusion of the P-Δ effects, indeed, is acceptable for this study. In the initial stage, it was stated that the simple buildings are expected to remain under a certain story drift level, and P-Δ effects are a far beyond issue for the buildings under evaluation in this study.

In NSP, loads are proportional to the masses at the floor levels (diaphragm levels) and the amplitude of the governing mode shape in the direction under consideration. Then the target displacement is obtained by a modified coefficient method prescribed in ASCE 41-13. The pushover must continue until 150% of the target displacement is reached, because the target displacement for a specified hazard level is a mean value and the scatter is noticeably high. To prevent probable collapses or heavy damage, the building must be forced up to 150% of the target displacement.

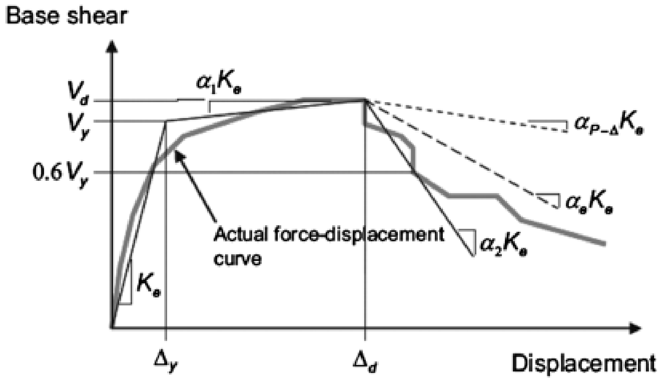

Figure 4.2. Idealized force-displacement curves from ASCE 41-13.

Once a pushover curve, which is base shear versus displacement of the center of mass of the top floor level, is obtained, a graphical procedure is utilized. In the first step, an idealized force-displacement curve (either bi-linear or tri-linear depending on the definition of the nonlinear stress-strain properties of the materials and the P-Δ effects) is constructed. ASCE 41-13 then continues: "The first line segment of the idealized force-displacement curve shall begin at the origin and have a slope equal to the effective lateral stiffness K_e. The effective lateral stiffness, K_e, shall be taken as the secant stiffness calculated at a base shear force equal to 60% of the effective yield strength of the structure. The effective yield strength V_y ,shall not be taken as greater than the maximum base shear force at any point along the force-displacement curve. The second line segment shall represent the positive post-yield slope $(\alpha_1 K_e)$, determined by a point (V_d, Δ_d) and a point at the intersection with the first line segment such that the areas above and below the actual curve are approximately balanced. (V_d, Δ_d) shall be a point on the actual force-displacement curve at the calculated target displacement, or at the displacement corresponding to the maximum base shear, whichever is least. The third line segment shall represent the negative post-yield slope, $(\alpha_2 K_e)$, determined by the point at the end of the positive post-yield slope (V_d, Δ_d) and the point at which the base shear degrades to 60% of the effective yield strength. The effective fundamental period in the direction under consideration shall be based on the idealized force-displacement curve." The effective fundamental period, T_e, is given as:

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \tag{4.1}$$

where T_i is the elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis, K_i is the elastic lateral stiffness of the building in the direction under consideration. The K_i value can be taken as the stiffness value calculated from the first two steps of the pushover analysis. The target displacement is calculated as:

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \tag{4.2}$$

Here, S_a is the spectral acceleration value at the effective fundamental period, g is the gravitational acceleration, C_0 is the modification factor to relate spectral displacement of an equivalent single degree of freedom system to the roof displacement of the multi degree of freedom system building, C_1 is the modification factor to relate the expected maximum inelastic displacements to the displacements calculated for linear elastic response, and C_2 is the modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on the maximum displacement response. The appropriate values of these terms can be found in ASCE 41-13 Section 7.4.3.3.

Although it seems like a complicated iterative procedure, the algorithm required to obtain the target displacement is as follows: From the first two steps of the pushover analysis, obtain K_i. T_i is already computed in modal analysis. By inspection of the pushover curve, make an estimate for a reasonable V_y value. Establish required equations related with the target displacement. Divide the area under the curve into trapezoidal bins and try to iterate V_y until the areas are balanced. This iteration can easily be done by the "goal seek" command in MS Excel™.

The effect of torsion is considered by amplifying the target displacement by the torsional amplification multiplier η, which is calculated as the ratio of the maximum displacement at any point on the diaphragm level to the average displacement at that level, where the displacements are calculated in compliance with the applied forces:

$$\eta = \frac{\delta_{\max}}{\delta_{avg}} \tag{4.3}$$

η is computed for all the floor levels, and the maximum of these values is adopted as the amplifier for the target displacement.

Once the target displacement is determined, plastic rotations in beams, columns, and shear walls at the target displacement are compared with the acceptance criteria tables in ASCE 41-13, Chapter 10. Within the scope of this study, in addition to the plastic rotation checks, several parameters are checked and suggestions for reinforcement are stated. In order to substantiate that the initial assumptions are sufficient for ductility and brittle failure is prevented, wall webs are checked against shear forces with the initial assertion of the total horizontal and the total vertical reinforcement to be 0.0025 of the gross web area. For transverse reinforcements of the beam spans, reinforcement ratios are suggested in terms of the span length.

When it comes to interstory drifts, neither the Turkish Seismic Code, nor the ASCE 41-13 state explicit expressions for nonlinear static procedure. However, drift checks are performed for the nonlinear static procedure. In addition, the beam and column effective rigidities are adjusted as per stipulated by the Turkish Seismic Code and a response spectrum analysis was performed. The design spectrum used is the same one used in the NSP. Thereafter, the interstory drifts are calculated according to the response spectrum analyses and are checked with the interstory drift acceptance criteria for linear dynamic procedure prescribed in Turkish Seismic Code.

5. CASE STUDIES

This chapter represents descriptions of the modelled buildings and corresponding features, and walks through the steps followed during the evaluation process. In terms of their structural characteristics, various 8-story and 4-story existing buildings were selected on the purpose of providing sufficient diversity that can be helpful in determining the extent of the proposed method.

5.1. Building 1.

- Number of stories : 8
- Structural system type : Dual (frame + structural walls)
- Story height : 3.5 m
- Dimesions in plan : 34 m by 17 m
- Plan area : 600 m^2
- $R_a(T)$ for preliminary design of member dimensions : 3
- g = 7 kN/m^2, q=3 kN/m^2
- Live load reduction factor, n = 0.3
- Seismic Zone : I (as specified by TSC 2007)
- Maximum span length : 5.0 m

As a first step, without changing the architectural properties (e.g. location of the columns), the column tributary areas were computed in order to determine the column sections according to the Section 3. The column reinforcement ratio were 0.01 as stated in the previous sections. The beam sections remained the same as the sections of the original buildings. However, during this study, the top and the bottom flexural reinforcement ratio for the beams were kept constant as 0.004 and 0.002, respectively. Structural wall dimensions were also determined according to Section 3. The layout of the building is depicted in Figure 5.1.

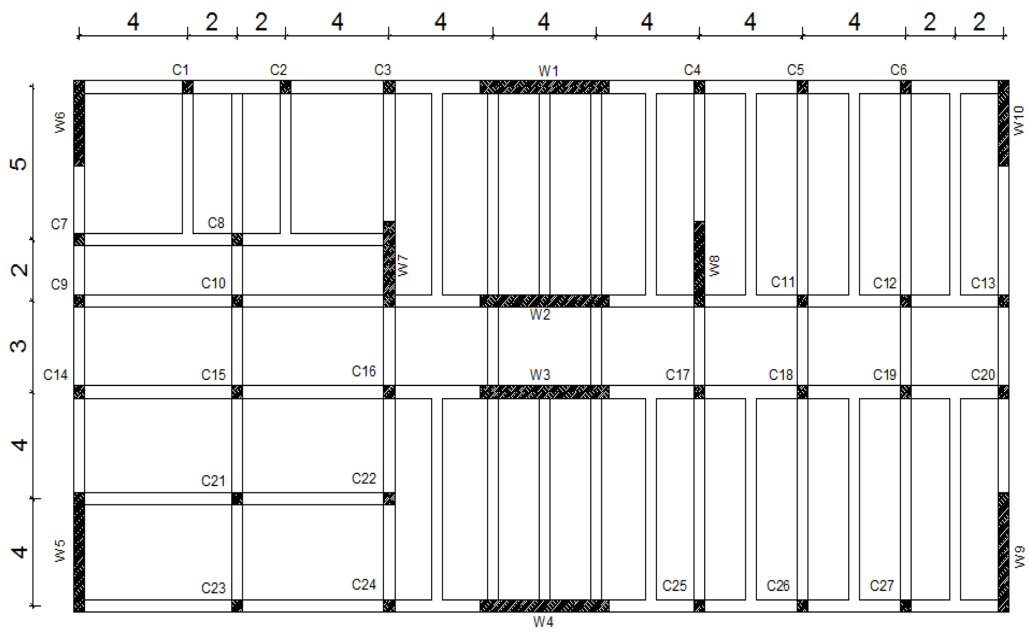

Figure 5.1. Layout of Building 1 (dimensions in m).

The building is modelled in SAP2000™ and analyzed under gravity loads compatible with the seismic weight. Regarding the level of axial stresses, cracked section rigidities of the columns were determined individually as per ASCE 41-13 chapter 10. The average multiplier for the flexural rigidities of the columns was found to be 0.38. This fact is emphasized in order to reveal how crucial the change in stiffness caused by cracked section rigidities can be. For the beams, the cracked section rigidities were decreased to 0.3 times the gross sectional rigidity. For structural walls, the cracked section rigidities are 0.5 times the gross section rigidities. It should be kept in mind that, the distribution of this multipliers throughout the columns can significantly change the modal mass participation ratios, periods and the corresponding seismic response of the buildings especially for the buildings composed solely of frames.

A modal analysis was performed with regard to the final stiffness and mass properties of the specified building. The periods of vibrations and modal mass participation ratios corresponding the first 4 mode is given in Table 5.1. The modal mass participation values are emphasized due to the limitation prescribed in TSC 2007. The TSC 2007 allows pushover analysis only if the mass participations of the relevant fundamental modes are greater than 0.70.

Table 5.1. Periods and modal mass participation ratios for Building 1.

Modal Participating Mass Ratios			
Mode	T (s)	Ux	Uy
1	1.223	0.00005	0.715
2	1.047	0.68823	0.00001
3	0.963	0.01054	0.002
4	0.322	$4x10^{-8}$	0.153

Although the pushover analyses in this study are performed as per ASCE 41-13, the target displacements according to the TSC 2007 are also computed for comparison purposes. Besides, the mass participations are deemed useful when the significance of a specified mode is of concern. Modes with noticeably high mass participations are likely to be dominant in the response when the building is exposed to base excitation. On the other hand, ASCE 41-13 specifies that if the shear for an individual story obtained from response spectrum analysis including sufficient number of modes to ensure 90% mass participation is higher than 1.3 times the story shear obtained from single mode (the mode which will be used in NSP), than the response spectrum analysis, NSP must be supplemented with linear dynamic analysis. For simplicity, the base shear forces were computed as an indicator of the higher mode effects instead of story shears. For Building 1, the ratios of the base shear forces obtained from a response spectrum analyses with sufficient number of modes satisfying the 90% rule to the base shear forces obtained from single mode response spectrum analyses are 1.0775 and 1.085 for mode 1 and mode 2, respectively.

An important issue in most of the buildings is torsion. In this study, the effects of torsion is taken into consideration via multiplying the target displacement by the torsional amplification multiplier η. In ASCE 41-13, the torsional amplification multiplier is described as the ratio of the maximum displacement at any point on a specific floor level (diaphragm) to the average displacement of that diaphragm level:

$$\eta = \frac{\delta_{max}}{\delta_{avg}} \tag{5.1}$$

The η values are calculated under the forces in compliance with those implemented in NSP. In each direction for which NSP is performed, the torsional amplification multiplier η is computed for each story level and the maximum value is adopted as the multiplier on the target displacement. The η values were found to be 1.083 and 1.086 for X and Y directions, respectively. The TSC 2007 limits the extent of NSP with only those buildings having η values smaller than 1.4. Referring to TSC 2007, it can be inferred that the torsion in Building 1 can be considered insignificant.

The next step is the pushover analysis. For column members, interacting P-M2-M3 hinges with elastic-perfectly plastic monotonic rotational backbone curve are adopted. P-M2-M3 hinges considers the combined effects of axial force and biaxial-bending conditions via a 3D interaction surface. For beam members, M3 hinges (flexural hinge) with elastic-perfectly plastic monotonic rotational backbone curves are defined. M3 hinge considers only the bending about strong direction of the member. The hinges are then assigned to both ends of the members. Finally, pushover analysis is performed for negative and positive sense of each direction. An actual pushover curve obtained for mode 1, superposed with an idealized curve, is shown in Figure 5.2.

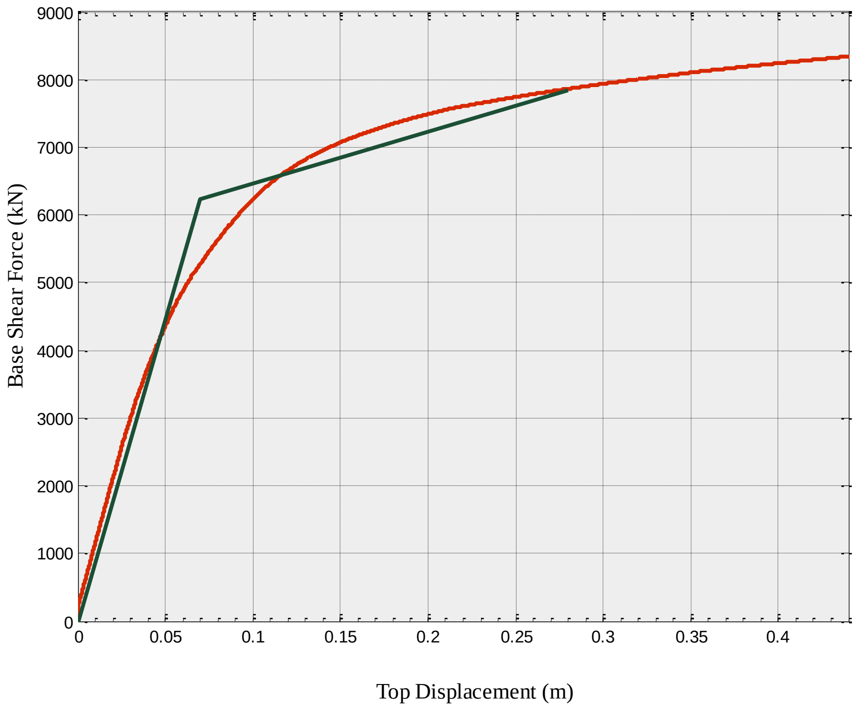

Figure 5.2. Actual and idealized pushover curves for Building 1, mode 1.

Target displacements for negative and positive senses in the two orthogonal directions were obtained. Table 5.2 represents the target displacements for positive directions of mode 1 and mode 2 in obtained as per ASCE 41-13 and TSC 2007.

Table 5.2. Target displacement values for Building 1, computed via different methods.

Target Displacement (m)		
	ASCE 41-13	TSC 2007
MODE-1	0.289	0.294
MODE-2	0.23	0.245

The target displacements then multiplied by the torsional amplification multiplier, η and the according to the ASCE 41-13 tables, the damage states in the members were determined in terms of the plastic rotations as Immediate Occupancy (IO), Life Safety (LS), Collapse Prevention (CP) and in excess on CP, denoted by FAIL. The final damage states for members are given in Table 5.3 and Table 5.4. In these tables, IO represents the Immediate Occupancy

Table 5.3. Final damage states of structural members, Building 1 (mode 1).

∑# of Columns	Elastic	IO	LS	CP	FAIL
216	102	62	52	0	0
%	47.22	28.70	24.07	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
280	61	118	96	5	0
%	21.79	42.14	34.29	2	0
∑# of Walls	Elastic	IO	LS	CP	FAIL
6	0	0	6	0	0
%	0.00	0.0	100.00	0	0

Table 5.4. Final damage states of structural members for Building 2 (mode 2).

∑# of Columns	Elastic	IO	LS	CP	FAIL
216	73	94	49	0	0
%	33.80	43.52	22.69	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
264	28	172	64	0	0
%	10.61	65.15	24.24	0	0
∑# of Walls	Elastic	IO	LS	CP	FAIL
4	0	0	4	0	0
%	0.00	0.0	100.00	0	0

The interstory drift limits for NSP, as mentioned before, are not explicitly addressed by ASCE 41-13 or TSC 2007. However, it is important to check if the story drifts are at least within the acceptable limits. Therefore, in addition to the interstory drifts obtained from the nonlinear static procedure, linear dynamic analysis (response spectrum analysis) is conducted and the results are compared with the interstory drift limits for linear elastic procedures prescribed in the TSC 2007. The response spectrum analyses were conducted in compliance with TSC 2007, and hence the cracked section rigidities are modified as per TSC 2007. The limits for relative interstory drift ratios for linear elastic procedures prescribed in the TSC 2007 are presented in the Table 5.5.

Table 5.5. Limits for the relative interstory drift ratios for linear elastic procedures prescribed in TSC 2007.

Relative Interstory Drift Ratio	Damage State		
	IO	LS	CP
δ_{ij}/H_{ij}	0.01	0.03	0.04

In Table 5.5, δ_{ij} stands for the drift, and H_{ij} stands for the height between the *i*th and the *j*th diaphragms (consecutive). The relative interstory drift ratio values obtained from linear and nonlinear analyses are shown in Figure 5.3 and Figure 5.4. As can be seen from these figures, the relative story drift ratios fall within the life safety limits for the first mode (Y-direction), and the relative story drift ratios fall within the immediate occupancy damage state for the second mode (X-direction).

The structural walls are initially assumed to have both longitudinal and vertical web reinforcement by an amount of 0.0025 times the gross web area. The wall webs are checked against shear and the shear capacity of the walls are determined through Equation 5.2:

$$V_r = A_{ch}\left(0.65\, f_{ctk} + \rho_{sh}\, f_{ywk}\right) \tag{5.2}$$

where A_{ch} is the gross cross sectional area of the wall, ρ_{sh} is the transverse reinforcement ratio of the web. Table 5.6 shows the shear capacity and the shear demand for structural walls.

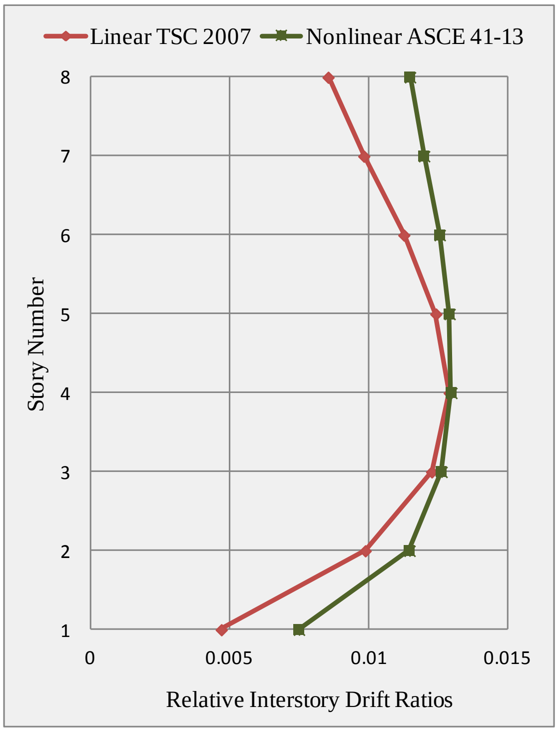

Figure 5.3. Relative interstory drift ratios for Building 1, mode 1.

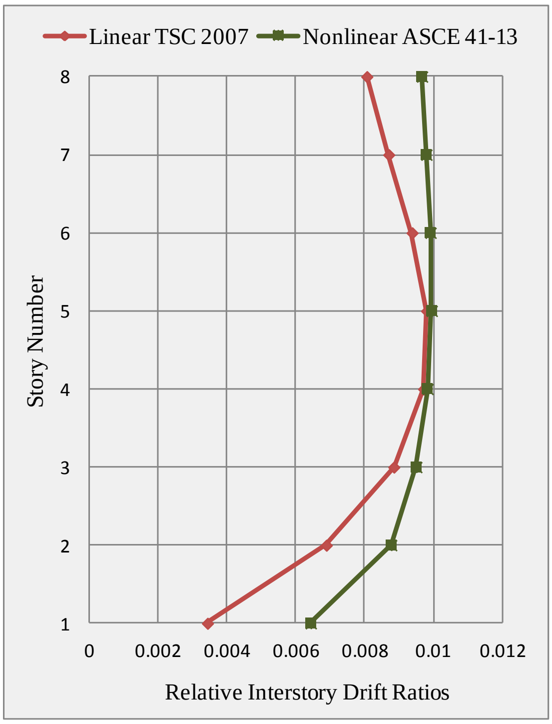

Figure 5.4. Relative interstory drift ratios for Building 1, mode 2.

Table 5.6. Shear capacities and demands for the structural walls of Building 1.

Wall ID	Capacity (kN)	Max. Demand (kN)
W1	4218	1840
W2	4218	755
W3	4218	912
W4	4218	1594
W6	3463.2	493
W7	2486.4	569
W8	2486.4	579
W10	2486.4	560
W5	3463.2	1233
W9	2486.4	1509

The maximum shear demand is the maximum of the negative and positive senses for a specified direction. The shear capacities of the walls at the base are well beyond the shear demands obtained from the nonlinear static procedure.

To sum up, the Building 1 is expected to perform within the life safety performance level.

5.2. Building 2.

- Number of stories : 8
- Structural system type : Dual (frame + structural walls)
- Story height : 3 m
- Dimesions in plan : 4.4 m by 19.5 m
- Plan area : 94.5 m^2
- $R_a(T)$ for preliminary design of member dimensions : 3
- g = 7 kN/m^2, q=3 kN/m^2
- Live load reduction factor, n = 0.3
- Seismic Zone : I (as specified by TSC 2007)
- Maximum span length : 5.0 m

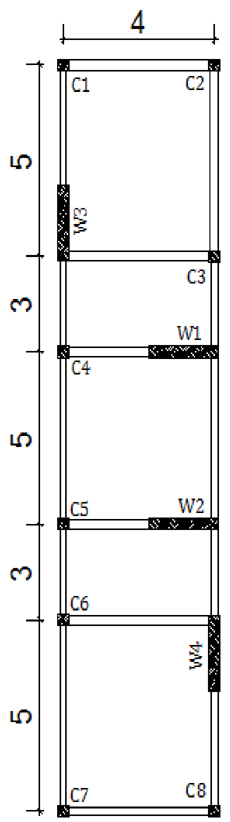

Figure 5.5. Layout of the Building 2 (dimensions in m).

The average multiplier for the cracked section rigidities of the columns is found to be 0.335.

Table 5.7. Periods and modal mass participation ratios for Building 2.

Modal Participating Mass Ratios			
Mode	T (s)	Ux	Uy
1	1.488	$9x10^{-4}$	0.012
2	1.329	0.677	0.041
3	1.151	0.045	0.686
4	0.449	$4x10^{-5}$	0.001

For Building 2, the ratios of the base shear forces obtained from response spectrum analyses with sufficient number of modes satisfying the 90% rule to the base shear forces obtained from single mode response spectrum analyses are 1.12 and 1.061 for mode 2 and mode 3, respectively.

The η values were found to be 1.164 and 1.066 for X and Y directions, respectively.

Table 5.8. Target displacement values for Building 2 computed via different methods.

Target Displacement (m)		
	ASCE 41-13	TSC 2007
MODE-2 (+)	0.279	0.306
MODE-2 (-)	0.337	0.306
MODE-3 (+)	0.249	0.2497
MODE-3 (-)	0.262	0.2497

The target displacements then multiplied by the torsional amplification multiplier, η and according to the ASCE 41-13 specifications, the damage states in the members are determined in terms of the plastic rotations. The final damage states for members are given in Tables 5.9 and 5.10.

Table 5.9. Final damage states of structural members for Building 2 (mode 2).

∑# of Columns	Elastic	IO	LS	CP	FAIL
64	47	9	8	0	0
%	73.44	14.06	12.50	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
48	0	3	45	0	0
%	0.00	6.25	93.75	0	0
∑# of Walls	Elastic	IO	LS	CP	FAIL
2	0	0	2	0	0
%	0.00	0.00	100.00	0	0

Table 5.10. Final damage states of structural members for Building 2 (mode 3).

∑# of Columns	Elastic	IO	LS	CP	FAIL
64	32	19	13	0	0
%	50.00	29.69	20.31	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
80	13	28	39	0	0
%	16.25	35.00	48.75	0	0
∑# of Walls	Elastic	IO	LS	CP	FAIL
2	0	0	2	0	0
%	0.00	0.00	100.00	0	0

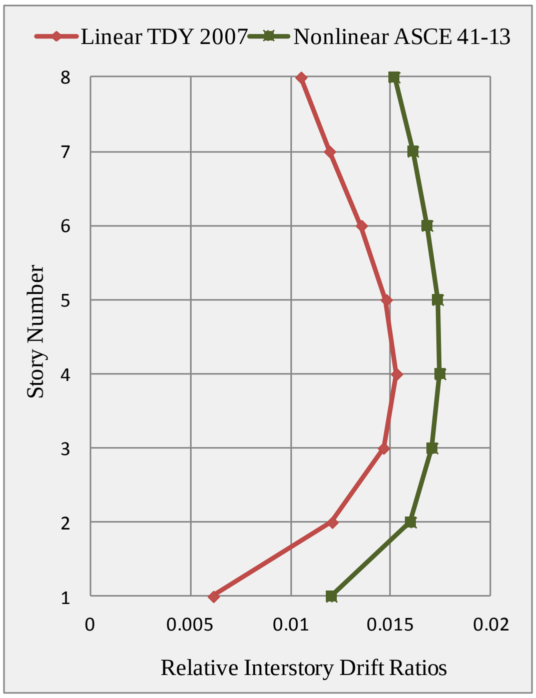

Figure 5.6. Relative interstory drift ratios for Building 2, mode 2.

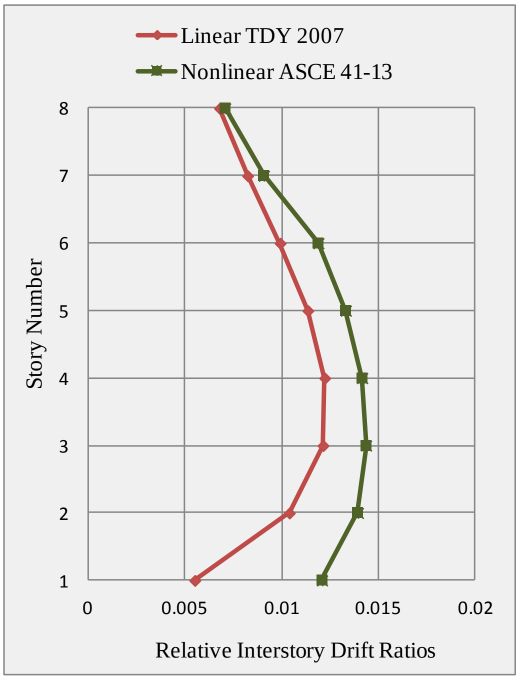

Figure 5.7. Relative interstory drift ratios for Building 2, mode 3.

Table 5.11. Shear capacities and demands for structural walls of Building 2.

Wall ID	Capacity (kN)	Max. Demand (kN)
W1	1332	251
W2	1332	233
W3	1332	479
W4	1332	479

The maximum shear demand is the maximum of the negative and positive senses for a specified direction. The shear capacities of the walls at the base are well beyond the shear demands obtained from the nonlinear static procedure.

To sum up, the Building 2 is expected to perform within the life safety performance level.

5.3. Building 3.

- Number of stories : 8
- Structural system type : Dual (frame + structural walls)
- Story height : 3 m
- Dimesions in plan : 22.2 m by 14.9 m
- Plan area : 331 m^2
- $R_a(T)$ for preliminary design of member dimensions : 3
- g = 7 kN/m^2, q=3 kN/m^2
- Live load reduction factor, n = 0.3
- Seismic Zone : I (as specified by TSC 2007)
- Maximum span length : 7.30 m

The average multiplier for the cracked section rigidities of the columns is found to be 0.348.

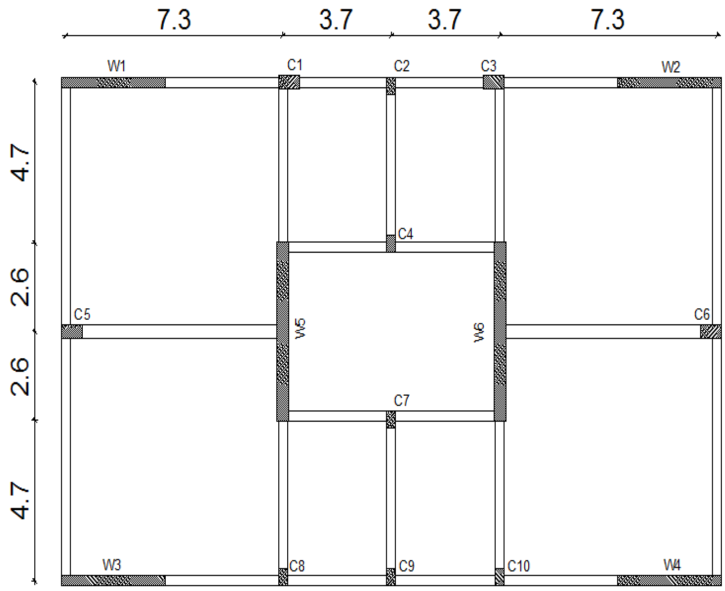

Figure 5.8. Layout of Building 3 (dimensions in m).

Table 5.12. Periods and modal mass participation ratios for Building 3.

Modal Participating Mass Ratios			
Mode	T (s)	Ux	Uy
1	0.94	0.71	0
2	0.92	0	0
3	0.80	0	0.69
4	0.24	0.16	0

For Building 3, the ratios of the base shear forces obtained from response spectrum analyses with sufficient number of modes satisfying the 90% rule to the base shear forces obtained from single mode response spectrum analyses are 1.055 and 1.0585 for mode 1 and mode 3, respectively.

The η values are found to be 1.0 and 1.0 for X and Y directions, respectively.

Table 5.13. Target displacement values for Building 3, computed via different methods.

Target Displacement (m)		
	ASCE 41-13	TSC 2007
MODE-1 (+)	0.2146	0.2170
MODE-1 (-)	0.2146	0.2170
MODE-3 (-)	0.179	0.1815
MODE-3 (-)	0.179	0.1815

The target displacements are multiplied by the torsional amplification multiplier, η and according to the ASCE 41-13 specifications, the damage states in the members are determined in terms of the plastic rotations. The final damage states for members are given in Tables 5.14 and 5.15 below.

Table 5.14. Final damage states of structural members for Building 3 (mode 1).

∑# of Columns	Elastic	IO	LS	CP	FAIL
80	72	0	8	0	0
%	90.00	0.00	10.00	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
112	8	60	44	0	0
%	7.14	53.57	39.29	0	0
∑# of Walls	Elastic	IO	LS	CP	FAIL
4	0	4	0	0	0
%	0.00	100.00	0.00	0	0

Table 5.15. Final damage states of structural members for Building 3 (mode 3).

∑# of Columns	Elastic	IO	LS	CP	FAIL
80	80	0	0	0	0
%	100.00	0.00	0.00	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
80	0	52	28	0	0
%	0.00	65.00	35.00	0	0
∑# of Walls	Elastic	IO	LS	CP	FAIL
2	0	2	0	0	0
%	0.00	100.00	0.00	0	0

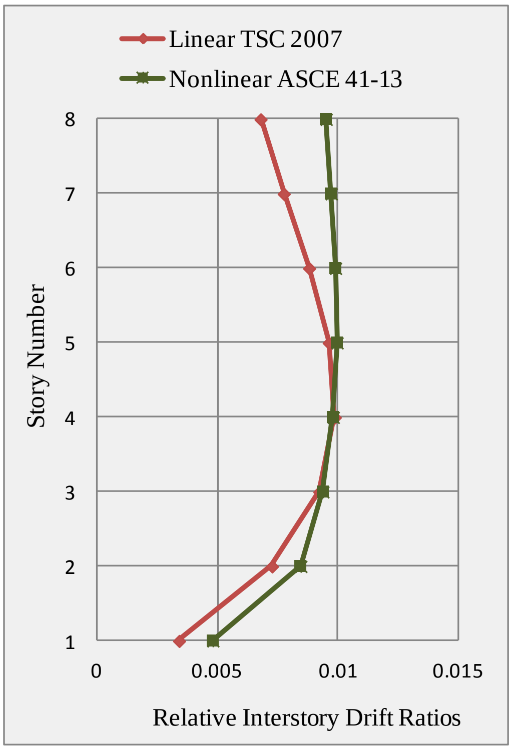

Figure 5.9. Relative interstory drift ratios for Building 3, mode 1.

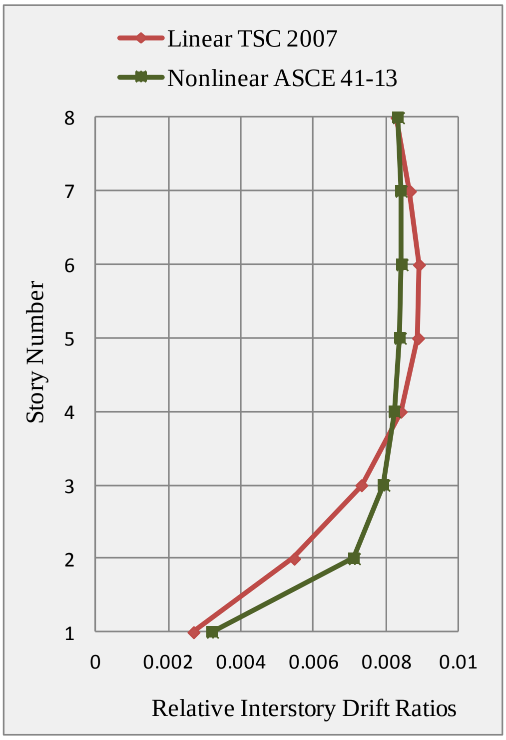

Figure 5.10. Relative interstory drift ratios for Building 3, mode 3.

Table 5.16. Shear capacities and demands for the structural walls of Building 3.

Wall ID	Capacity (kN)	Max. Demand (kN)
W1	2331	754
W2	2331	733
W3	2331	733
W4	2331	754
W5	4662	1786
W6	4662	1786

The maximum shear demand is the maximum of the negative and positive senses for a specified direction. The shear capacities of the walls at the base are well beyond the shear demands obtained from the nonlinear static procedure.

To sum up, the Building 3 is expected to perform within the life safety, even perhaps immediate occupancy performance level.

5.4. Building 4.

- Number of stories : 8
- Structural system type : Dual (frame + structural walls)
- Story height : 3.5 m
- Dimesions in plan : 14.75 m by 17.75 m
- Plan area : 195 m^2 for the ground story, 270 m^2 for upper stories
- $R_a(T)$ for preliminary design of member dimensions : 3
- g = 7 kN/m^2, q=3 kN/m^2
- Live load reduction factor, n = 0.3
- Seismic Zone : I (as specified by TSC 2007)
- Maximum span length : 3.90 m

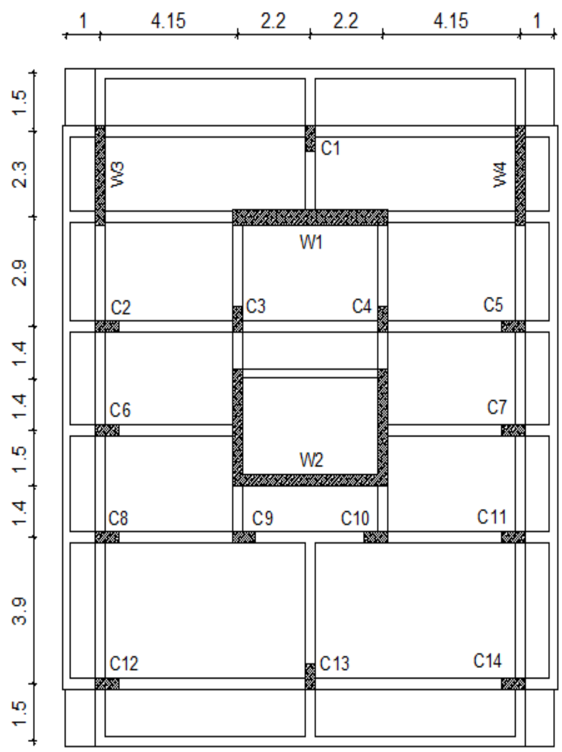

Figure 5.11. Layout of Building 4 (dimensions in m).

The average multiplier for the cracked section rigidities of the columns is found to be 0.353.

Table 5.17. Periods and modal mass participation ratios for Building 4.

Modal Participating Mass Ratios			
Mode	T (s)	Ux	Uy
1	1.299	0	0.737
2	1.298	0	0.0001
3	1.087	0.714	0
4	0.343	0	0.152

For Building 4, the ratios of the base shear forces obtained from response spectrum analyses with sufficient number of modes satisfying the 90% rule to the base shear forces obtained from single mode response spectrum analyses are 1.077 and 1.068 for mode 1 and mode 3, respectively.

The η values are found to be 1.19 and 1.014 for X and Y directions, respectively.

Table 5.18. Target displacement values for Building 4, computed via different methods.

Target Displacement (m)		
	ASCE 41-13	TSC 2007
MODE-1 (+)	0.3213	0.316
MODE-1 (-)	0.27	0.316
MODE-3 (-)	0.239	0.26
MODE-3 (-)	0.239	0.26

The target displacements are multiplied by the torsional amplification multiplier, η and according to the ASCE 41-13 specifications, the damage states in the members are determined in terms of the plastic rotations. The final damage states for members are given in Tables 5.19 and 5.20 below.

Table 5.19. Final damage states of structural members for Building 4 (mode 1).

∑# of Columns	Elastic	IO	LS	CP	FAIL
112	33	21	58	0	0
%	29.46	18.75	51.79	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
128	4	64	56	4	0
%	3.13	50.00	43.75	3	0
∑# of Walls	Elastic	IO	LS	CP	FAIL
3	0	0	3	0	0
%	0.00	0.00	100.00	0	0

Table 5.20. Final damage states of structural members for Building 4 (mode 3).

∑# of Columns	Elastic	IO	LS	CP	FAIL
112	54	38	20	0	0
%	48.21	33.93	17.86	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
121	29	69	23	0	0
%	23.97	57.02	19.01	0	0
∑# of Walls	Elastic	IO	LS	CP	FAIL
2	0	0	2	0	0
%	0.00	0.00	100.00	0	0

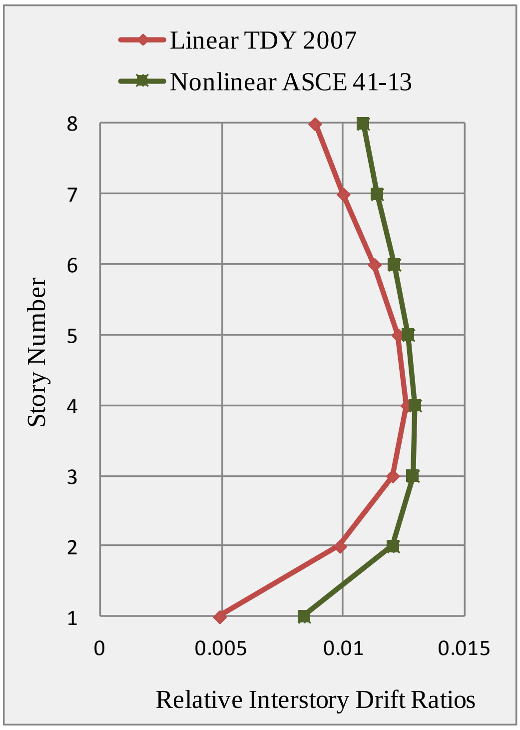

Figure 5.12. Relative interstory drift ratios for Building 4, mode 1.

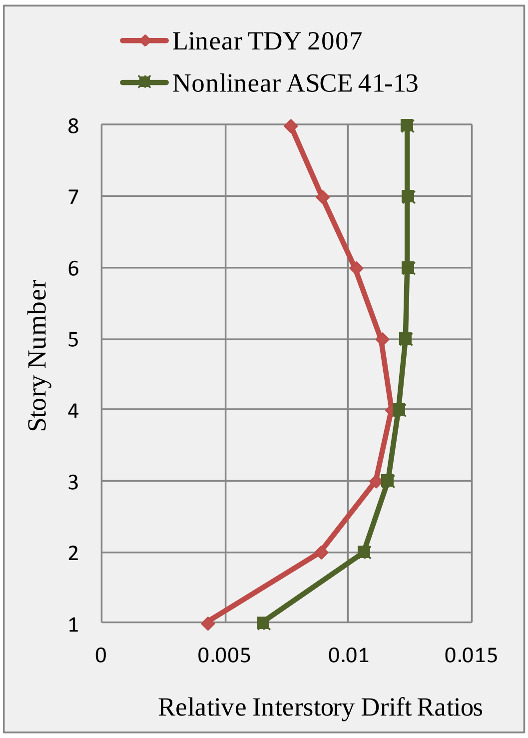

Figure 5.13. Relative interstory drift ratios for Building 4, mode 3.

Table 5.21. Shear capacities and demands for the structural walls of Building 4.

Wall ID	Capacity (kN)	Max. Demand (kN)
W3	1765	436
W4	1765	436
W1	4173.6	1186
W2 (U-Wall) (Y)	3796.2	1190
W2 (U-Wall) (X)	3130.2	1555

The maximum shear demand is the maximum of the negative and positive senses for a specified direction. The shear capacities of the walls at the base are well beyond the shear demands obtained from the nonlinear static procedure.

To sum up, the Building 4 is expected to perform within the life safety performance level.

5.5. Building 5.

- Number of stories : 8
- Structural system type : Dual (frame + structural walls)
- Story height : 3 m
- Dimesions in plan : 19.60 m by 15.50 m
- Plan area : 302 m^2
- $R_a(T)$ for preliminary design of member dimensions : 3
- g = 7 kN/m^2, q=3 kN/m^2
- Live load reduction factor, n = 0.3
- Seismic Zone : I (as specified by TSC 2007)
- Maximum span length : 7 m

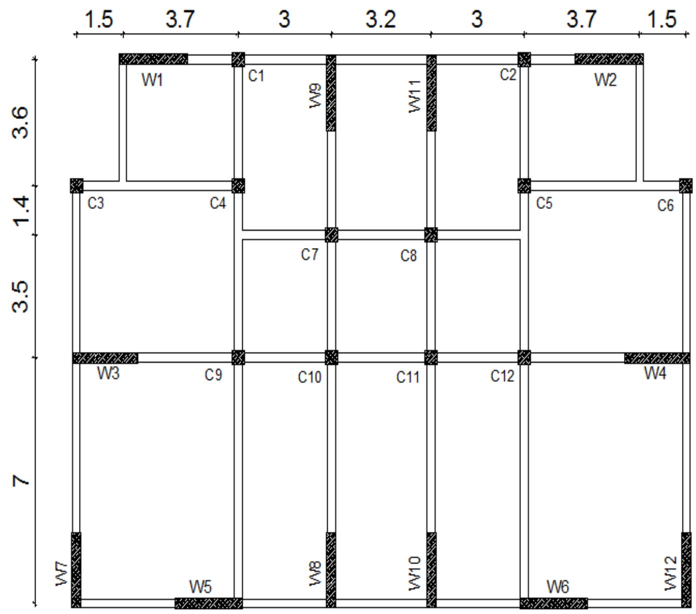

Figure 5.14. Layout of Building 5 (dimensions in m).

The average multiplier for the cracked section rigidities of the columns is found to be 0.344.

Table 5.22. Periods and modal mass participation ratios for Building 5.

Modal Participating Mass Ratios			
Mode	T (s)	Ux	Uy
1	1.283	0.00003	0.723
2	1.235	0.047	0.0004
3	1.126	0.685	0
4	0.342	0	0.144

For Building 5, the ratios of the base shear forces obtained from response spectrum analyses with sufficient number of modes satisfying the 90% rule to the base shear forces obtained from single mode response spectrum analyses are 1.075 and 1.053 for mode 1 and mode 3, respectively.

The η values are found to be 1.273 and 1.031 for X and Y directions, respectively.

Table 5.23. Target displacement values for Building 5, computed via different methods.

Target Displacement (m)		
	ASCE 41-13	TSC 2007
MODE-1 (+)	0.297	0.309
MODE-1 (-)	0.294	0.309
MODE-3 (+)	0.26	0.247
MODE-3 (-)	0.263	0.247

The target displacements are multiplied by the torsional amplification multiplier, η and according to the ASCE 41-13 specifications, the damage states in the members are determined in terms of the plastic rotations. The final damage states for members are given in Tables 5.24 and 5.25 below.

Table 5.24. Final damage states of structural members for Building 5 (mode 1).

∑# of Columns	Elastic	IO	LS	CP	FAIL
96	56	20	20	0	0
%	58.33	20.83	20.83	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
144	19	23	102	0	0
%	13.19	15.97	70.83	0	0
∑# of Walls	Elastic	IO	LS	CP	FAIL
6	0	0	6	0	0
%	0.00	0.00	100.00	0	0

Table 5.25. Final damage states of structural members for Building 5 (mode 3).

∑# of Columns	Elastic	IO	LS	CP	FAIL
96	59	14	23	0	0
%	61.46	14.58	23.96	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
160	20	18	114	8	0
%	12.50	11.25	71.25	5	0
∑# of Walls	Elastic	IO	LS	CP	FAIL
6	0	0	6	0	0
%	0.00	0.00	100.00	0	0

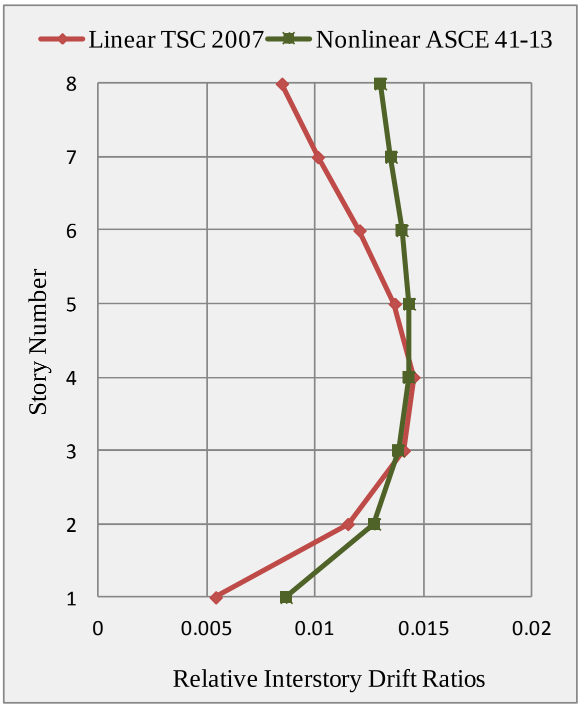

Figure 5.15. Relative interstory drift ratios for Building 5, mode 1.

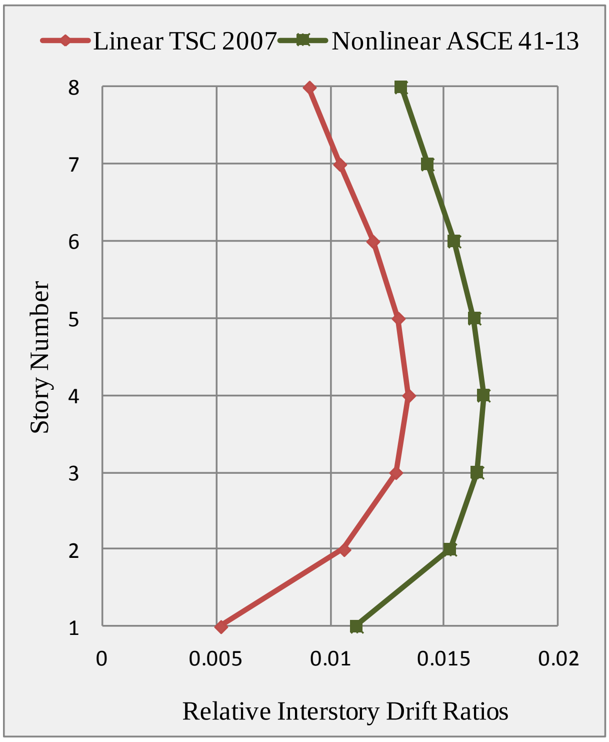

Figure 5.16. Relative interstory drift ratios for Building 5, mode 3.

Table 5.26. Shear capacities and demands for the structural walls of Building 5.

Wall ID	Capacity (kN)	Max. Demand (kN)
W1	1465.2	426
W2	1465.2	426
W3	1398.6	200
W4	1398.6	192
W5	1431.9	215
W6	1431.9	219
W7	1431.9	226
W8	1431.9	216
W9	1431.9	266
W10	1431.9	217
W11	1431.9	278
W12	1431.9	207

The maximum shear demand is the maximum of the negative and positive senses for a specified direction. The shear capacities of the walls at the base are well beyond the shear demands obtained from the nonlinear static procedure.

To sum up, the Building 5 is expected to perform within the life safety performance level.

5.6. Building 6.

- Number of stories : 8
- Structural system type : Dual (frame + structural walls)
- Story height : 3.5 m
- Dimesions in plan : 28 m by 17.50 m
- Plan area : 508 m^2
- $R_a(T)$ for preliminary design of member dimensions : 3
- g = 7 kN/m^2, q=3 kN/m^2
- Live load reduction factor, n = 0.3
- Seismic Zone : I (as specified by TSC 2007)
- Maximum span length : 7 m

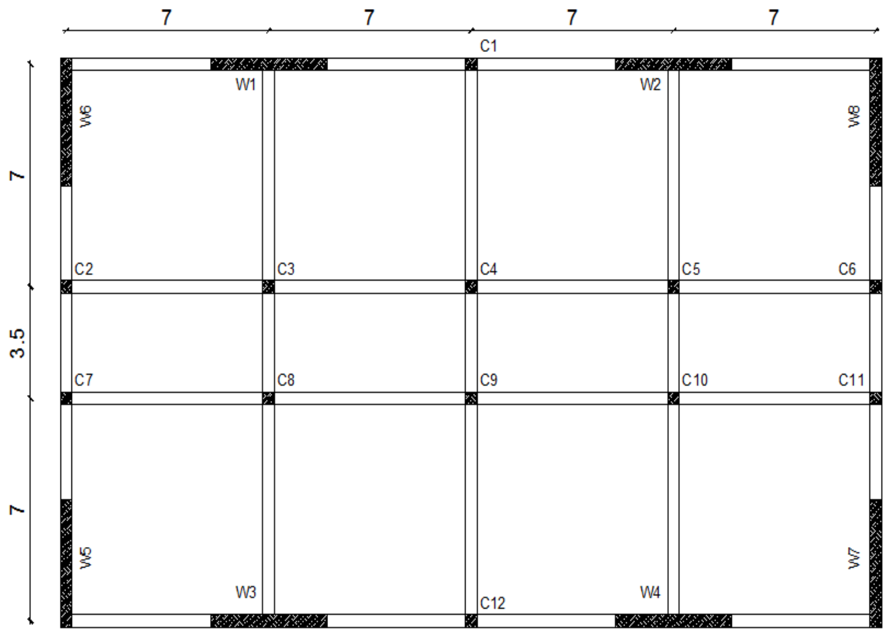

Figure 5.17. Layout of Building 6 (dimensions in m).

The average multiplier for the cracked section rigidities of the columns is found to be 0.3608.

Table 5.27. Periods and modal mass participation ratios for Building 6.

Modal Participating Mass Ratios			
Mode	T (s)	Ux	Uy
1	1.124	0.703	0
2	1.117	0	0.702
3	0.827	0	0
4	0.272	0	0.166

For Building 6, the ratios of the base shear forces obtained from response spectrum analyses with sufficient number of modes satisfying the 90% rule to the base shear forces obtained from single mode response spectrum analyses are 1.0795 and 1.078 for mode 1 and mode 2, respectively.

The η values are found to be 1.0 and 1.0 for X and Y directions, respectively.

Table 5.28. Target displacement values for Building 6, computed via different methods.

Target Displacement (m)		
	ASCE 41-13	TSC 2007
MODE-1 (+)	0.247	0.27
MODE-1 (-)	0.247	0.27
MODE-2 (-)	0.247	0.27
MODE-2 (-)	0.247	0.27

The target displacements are multiplied by the torsional amplification multiplier, η and according to the ASCE 41-13 specifications, the damage states in the members are determined in terms of the plastic rotations. The final damage states for members are given in Tables 5.29 and 5.30 below.

Table 5.29. Final damage states of structural members for Building 6 (mode 1).

∑# of Columns	Elastic	IO	LS	CP	FAIL
96	90	4	2	0	0
%	93.75	4.17	2.08	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
128	0	40	88	0	0
%	0.00	31.25	68.75	0	0
∑# of Walls	Elastic	IO	LS	CP	FAIL
4	0	4	0	0	0
%	0.00	100.00	0.00	0	0

Table 5.30. Final damage states of structural members for Building 6 (mode 2).

∑# of Columns	Elastic	IO	LS	CP	FAIL
96	76	12	8	0	0
%	79.17	12.50	8.33	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
120	2	45	73	0	0
%	1.67	37.50	60.83	0	0
∑# of Walls	Elastic	IO	LS	CP	FAIL
4	0	4	0	0	0
%	0.00	100.00	0.00	0	0

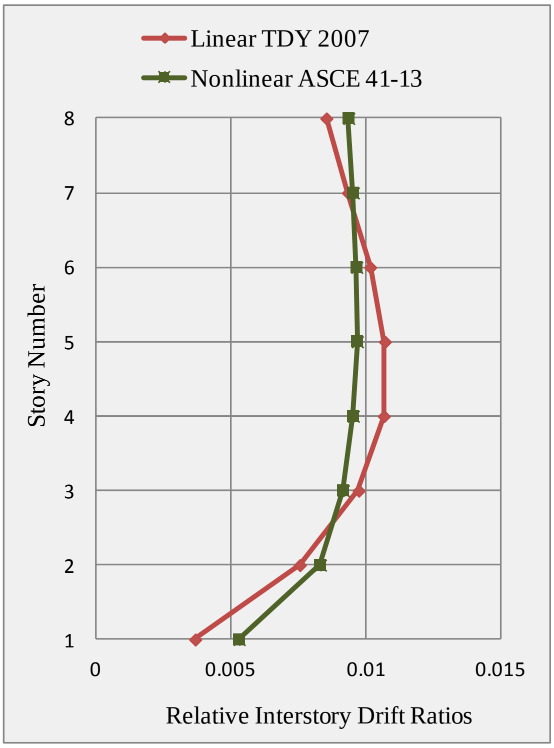

Figure 5.18. Relative interstory drift ratios for Building 6, mode 1.

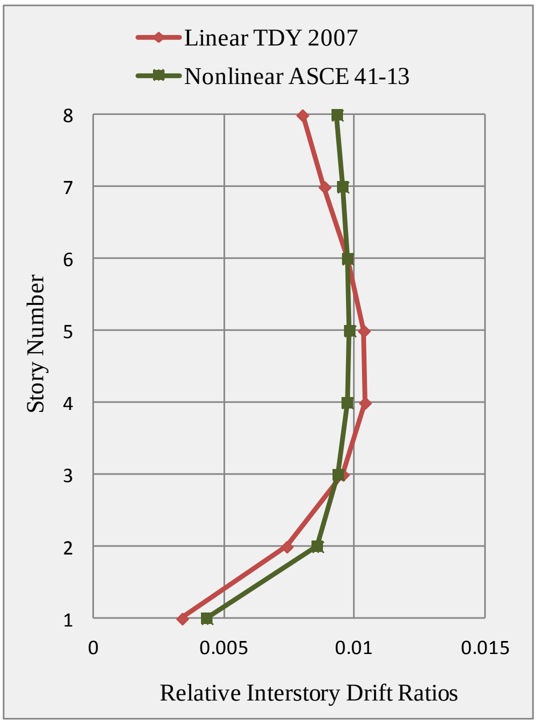

Figure 5.19. Relative interstory drift ratios for Building 6, mode 2.

Table 5.31. Shear capacities and demands for the structural walls of Building 6.

Wall ID	Capacity (kN)	Max. Demand (kN)
W1	3569.76	732
W2	3569.76	732
W3	3569.76	732
W4	3569.76	732
W5	3569.76	963
W6	3569.76	963
W7	3569.76	963
W8	3569.76	963

The maximum shear demand is the maximum of the negative and positive senses for a specified direction. The shear capacities of the walls at the base are well beyond the shear demands obtained from the nonlinear static procedure.

To sum up, the Building 6 is expected to perform within the immediate occupancy performance level when the columns and the structural walls are considered. In the worst case, the expected performance level is life safety.

5.7. Building 7.

- Number of stories : 8
- Structural system type : Frame (without structural walls)
- Story height : 3.5 m
- Dimesions in plan : 29.20 m by 10.50 m
- Plan area : 291 m^2
- $R_a(T)$ for preliminary design of member dimensions : 4
- g = 7 kN/m^2, q=3 kN/m^2
- Live load reduction factor, n = 0.3
- Seismic Zone : I (as specified by TSC 2007)
- Maximum span length : 7.50 m

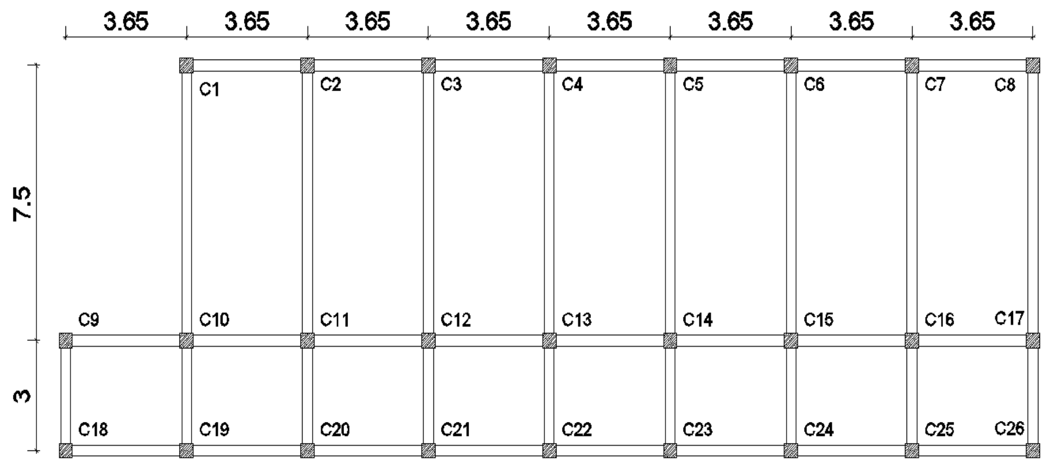

Figure 5.20. Layout of Building 7 (dimensions in m).

The average multiplier for the cracked section rigidities of the columns is found to be 0.3438.

Table 5.32. Periods and modal mass participation ratios for Building 7.

Modal Participating Mass Ratios			
Mode	T (s)	Ux	Uy
1	1.759	$6x10^{-6}$	0.807
2	1.605	0.027	0.0005
3	1.356	0.807	0.00005
4	0.575	$2x10^{-7}$	0.117

For Building 7, the ratios of the base shear forces obtained from response spectrum analyses with sufficient number of modes satisfying the 90% rule to the base shear forces obtained from single mode response spectrum analyses are 1.058 and 1.028 for mode 1 and mode 3, respectively.

The η values are found to be 1.129 and 1.045 for X and Y directions, respectively.

Table 5.33. Target displacement values for Building 7, computed via different methods.

Target Displacement (m)		
	ASCE 41-13	TSC 2007
MODE-1 (+)	0.42	0.43
MODE-1 (-)	0.476	0.43
MODE-3 (+)	0.32	0.30
MODE-3 (-)	0.32	0.30

The target displacements are multiplied by the torsional amplification multiplier, η and according to the ASCE 41-13 specifications, the damage states in the members are determined in terms of the plastic rotations. The final damage states for members are given in Tables 5.34 and 5.35 below.

Table 5.34. Final damage states of structural members for Building 7 (mode 1).

∑# of Columns	Elastic	IO	LS	CP	FAIL
208	126	29	46	0	7
%	60.58	13.94	22.12	0	3
∑# of Beams	Elastic	IO	LS	CP	FAIL
136	21	33	28	54	0
%	15.44	24.26	20.59	40	0

Table 5.35. Final damage states of structural members for Building 7 (mode 3).

∑# of Columns	Elastic	IO	LS	CP	FAIL
208	141	15	52	0	0
%	67.79	7.21	25.00	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
184	60	39	59	26	0
%	32.61	21.20	32.07	14	0

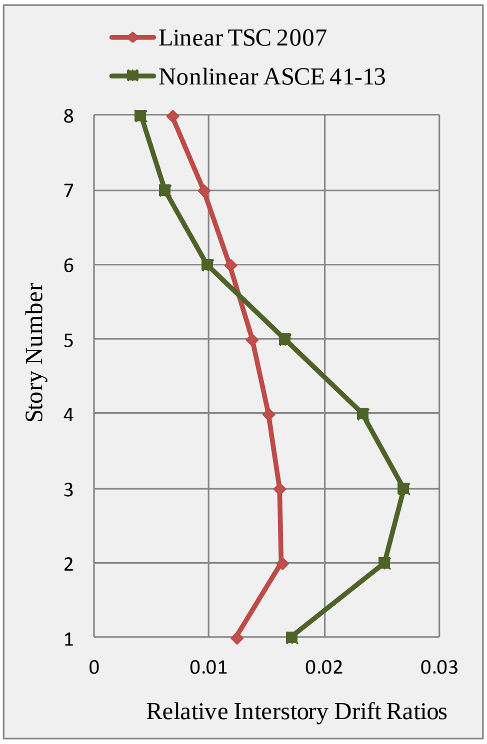

Figure 5.21. Relative interstory drift ratios for Building 7, mode 1.

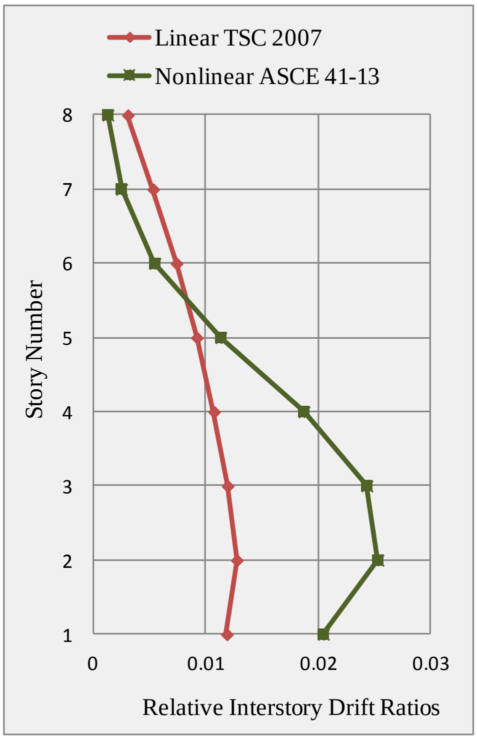

Figure 5.22. Relative interstory drift ratios for Building 7, mode 3.

The buildings with structural walls perform well in terms of the story drifts. The story drifts of these structures are slightly more than the immediate occupancy limit prescribed in TSC 2007. However, the situation in the buildings those which composed solely of frames is more critical. The frame–only buildings should be checked very carefully in terms of story drifts.

To sum up, the Building 7 is expected to perform within the collapse prevention performance level and perhaps even deform into a collapse state due the failure of 7 successive column sections.

5.8. Building 8.

- Number of stories : 8
- Structural system type : Frame (without structural walls)
- Story height : 3.5 m
- Dimensions in plan : 42.65 m by 27.30 m
- Plan area : 1050 m^2
- $R_a(T)$ for preliminary design of member dimensions : 4
- g = 7 kN/m^2, q=3 kN/m^2
- Live load reduction factor, n = 0.3
- Seismic Zone : I (as specified by TSC 2007)
- Maximum span length : 8.90 m

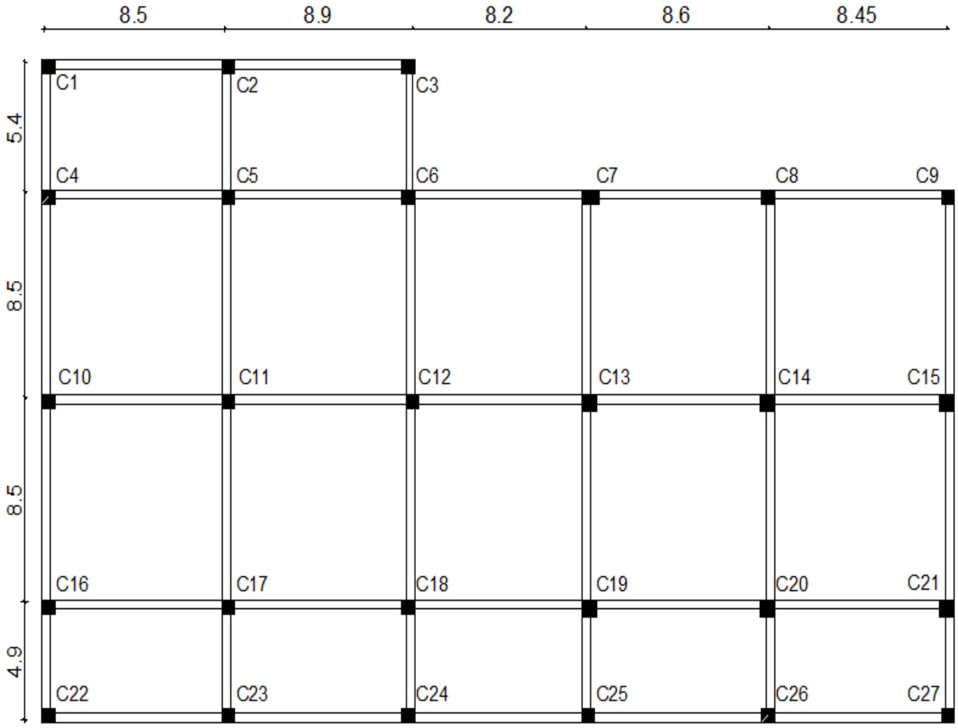

Figure 5.23. Layout of Building 8 (dimensions in m).

The average multiplier for the cracked section rigidities of the columns is found to be 0.3438.

Table 5.36. Periods and modal mass participation ratios for Building 8.

Modal Participating Mass Ratios			
Mode	T (s)	Ux	Uy
1	2.065	0.784	0.00005
2	1.91	0.0007	0.697
3	1.808	0.003	0.089
4	0.638	0.106	0.00001

For Building 8, the ratios of the base shear forces obtained from response spectrum analyses with sufficient number of modes satisfying the 90% rule to the base shear forces obtained from single mode response spectrum analyses are 1.074 and 1.175 for mode 1 and mode 3, respectively.

The η values are found to be 1.07 and 1.58 for X and Y directions, respectively.

Table 5.37. Target displacement values for Building 8, computed via different methods.

Target Displacement (m)		
	ASCE 41-13	TSC 2007
MODE-1 (+)	0.72	0.51
MODE-1 (-)	0.736	0.51
MODE-3 (+)	0.647	0.417
MODE-3 (-)	0.655	0.417

The target displacements are multiplied by the torsional amplification multiplier, η and according to the ASCE 41-13 specifications, the damage states in the members are determined in terms of the plastic rotations. The final damage states for members are given in Tables 5.38 and 5.39 below.

Table 5.38. Final damage states of structural members for Building 8 (mode 1).

∑# of Columns	Elastic	IO	LS	CP	FAIL
216	185	4	27	0	0
%	85.65	1.85	12.50	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
176	0	0	18	158	0
%	0.00	0.00	10.23	90	0

Table 5.39. Final damage states of structural members for Building 8 (mode 3).

∑# of Columns	Elastic	IO	LS	CP	FAIL
216	180	4	25	7	0
%	83.33	1.85	11.57	3	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
168	0	0	4	110	54
%	0.00	0.00	2.38	65	32

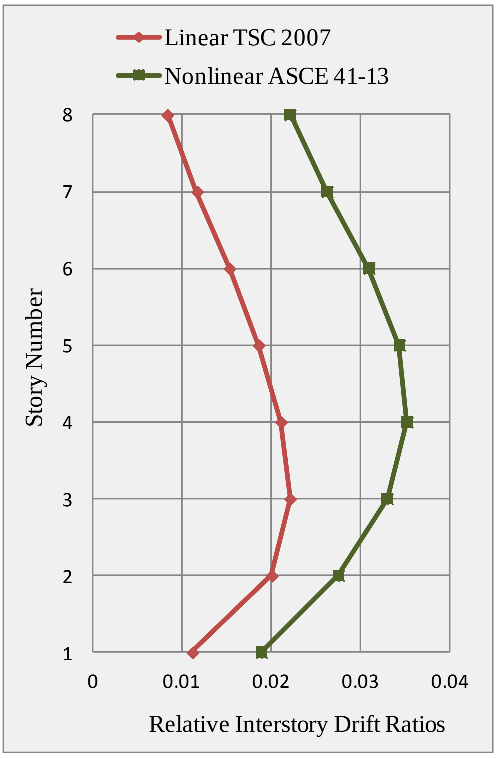

Figure 5.24. Relative interstory drift ratios for Building 8, mode 1.

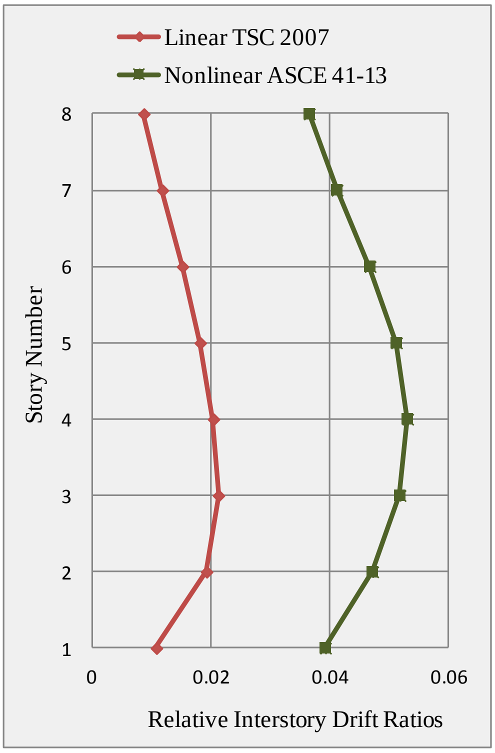

Figure 5.25. Relative interstory drift ratios for Building 8, mode 3.

At first glance, Building 8 draws attention with its high period of vibration. This can make sense when the spans are considered. The building has a relatively big area in plan and hence higher seismic mass. Among the base shear values calculated to check whether the higher mode effects are significant in the buildings under evaluation, the greatest value so far is obtained for Building 8 in the Y direction. Although it does not exceed the limit 1.3 prescribed in ASCE 41-13, it is noticeably high when compared to the other buildings. This may be the reason for the apparent difference between the target displacement values estimated by ASCE 41-13 and TSC 2007, since the equal displacement rule is valid for the first-mode dominant systems and TSC 2007 method is based on the equal displacement rule. Another point that deserves to be mentioned is that most of the beams are within the collapse prevention limits while the columns remain elastic. This is mostly due to the high span lengths. The columns are initially designed according to the corresponding tributary areas, meaning that the strong column weak beam principle is almost guaranteed due to the large tributary areas of the columns.

To sum up, the Building 8 is expected to perform within the collapse prevention performance level when the beams are considered, however the overall performance level is hard to specify due to the performances of the columns, and the building is most likely to remain in the life safety – collapse prevention interface.

5.9. Building 9.

- Number of stories : 8
- Structural system type : Frame (without structural walls)
- Story height : 3.5 m
- Dimensions in plan : 16.80 m by 14.50 m
- Plan area : 200 m^2
- $R_a(T)$ for preliminary design of member dimensions : 4
- g = 7 kN/m^2, q=3 kN/m^2
- Live load reduction factor, n = 0.3
- Seismic Zone : I (as specified by TSC 2007)
- Maximum span length : 5.30 m

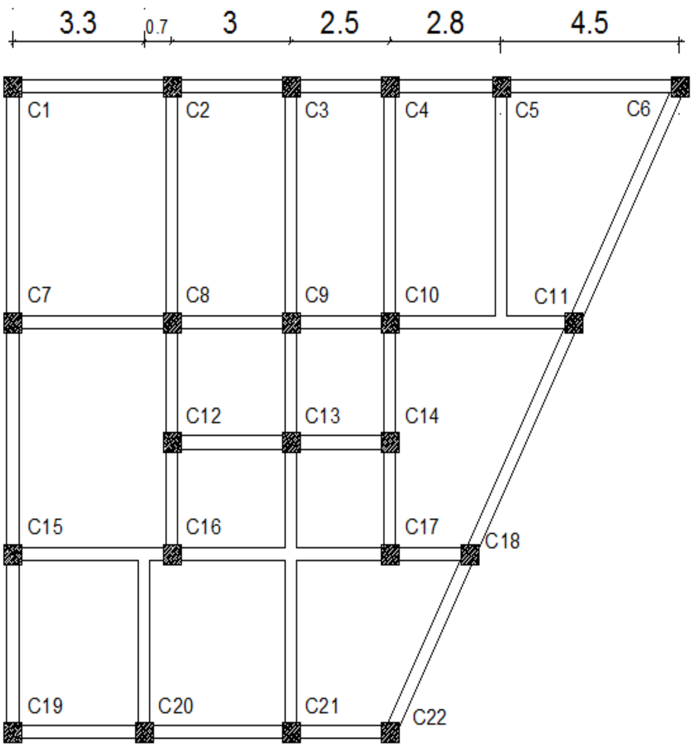

Figure 5.26. Layout of the Building 9 (dimensions in m).

The average multiplier for the cracked section rigidities of the columns is found to be 0.325.

Table 5.40. Periods and modal mass participation ratios for Building 9.

Modal Participating Mass Ratios			
Mode	T (s)	Ux	Uy
1	1.849	0.032	0.773
2	1.693	0.629	0.047
3	1.616	0.162	0.001
4	0.61	0.004	0.097

For Building 9, the ratios of the base shear forces obtained from response spectrum analyses with sufficient number of modes satisfying the 90% rule to the base shear forces obtained from single mode response spectrum analyses are 1.084 and 1.254 for mode 1 and mode 2, respectively.

The η values are found to be 1.58 and 1.222 for X and Y directions, respectively.

Table 5.41. Target displacement values for Building 9, computed via different methods.

Target Displacement (m)		
	ASCE 41-13	TSC 2007
MODE-1 (+)	0.453	0.42
MODE-1 (-)	0.46	0.42
MODE-2 (-)	0.417	0.307
MODE-2 (-)	0.402	0.307

The target displacements are multiplied by the torsional amplification multiplier, η and according to the ASCE 41-13 specifications, the damage states in the members are determined in terms of the plastic rotations. The final damage states for members are given in Tables 5.42 and 5.43 below.

Table 5.42. Final damage states of structural members for Building 9 (mode 1).

∑# of Columns	Elastic	IO	LS	CP	FAIL
176	154	0	22	0	0
%	87.50	0.00	12.50	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
144	32	22	23	67	0
%	22.22	15.28	15.97	47	0

Table 5.43. Final damage states of structural members for Building 9 (mode 2).

∑# of Columns	Elastic	IO	LS	CP	FAIL
176	129	21	26	0	0
%	73.30	11.93	14.77	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
168	39	40	23	50	16
%	23.21	23.81	13.69	30	9.524

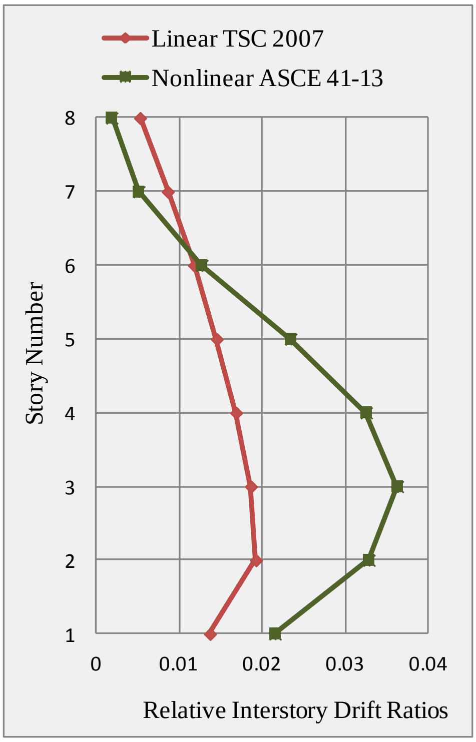

Figure 5.27. Relative interstory drift ratios for Building 9, mode 1.

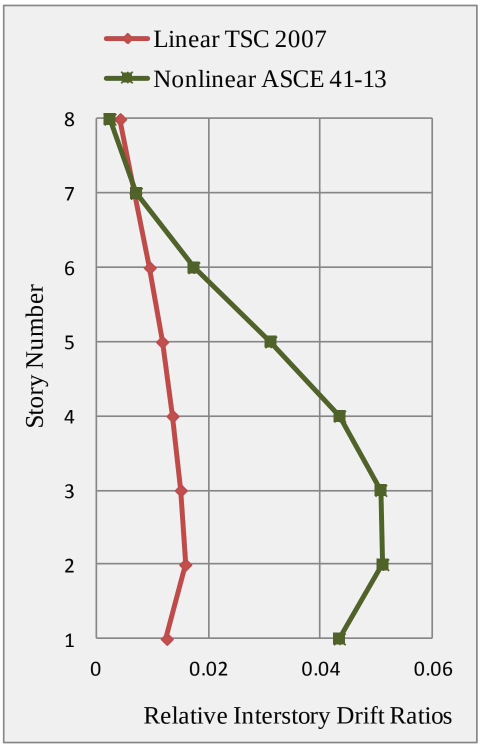

Figure 5.28. Relative interstory drift ratios for Building 9, mode 2.

The statements previously made for the Building 8 are valid for Building 9 as well. An interesting point for Building 9 is that the drifts obtained from the nonlinear analysis for the predominant mode (mode 1) is clearly less from the one for the second mode. This is due to the high torsional amplification of the second mode target displacement.

To sum up, the Building 9 is expected to perform within the collapse prevention performance level when the beams are considered, however the overall performance level is hard to specify due to the performances of the columns, and the building is most likely to remain in the life safety – collapse prevention interface.

5.10. Building 10.

- Number of stories : 8
- Structural system type : Frame (without structural walls)
- Story height : 3.5 m
- Dimensions in plan : 29.20 m by 18.45 m
- Plan area : 534 m^2
- $R_a(T)$ for preliminary design of member dimensions : 4
- g = 7 kN/m^2, q=3 kN/m^2
- Live load reduction factor, n = 0.3
- Seismic Zone : I (as specified by TSC 2007)
- Maximum span length : 7.45 m

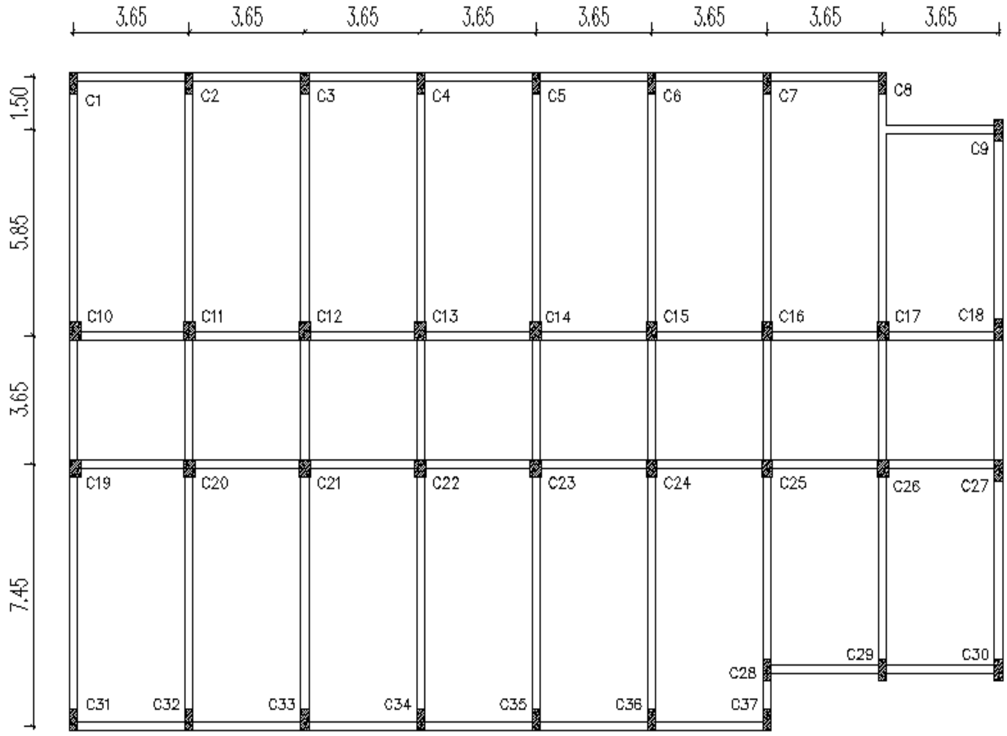

Figure 5.29. Layout of Building 10 (dimensions in m).

The average multiplier for the cracked section rigidities of the columns is found to be 0.347.

Table 5.44. Periods and modal mass participation ratios for Building 10.

Modal Participating Mass Ratios			
Mode	T (s)	Ux	Uy
1	1.872	0.00004	0.781
2	1.674	0.0001	0.033
3	1.541	0.826	0.00008
4	0.614	0.000005	0.101

For Building 10, the ratios of the base shear forces obtained from response spectrum analyses with sufficient number of modes satisfying the 90% rule to the base shear forces obtained from single mode response spectrum analyses are 1.048 and 1.033 for mode 1 and mode 3, respectively.

The η values are found to be 1.012 and 1.32 for X and Y directions, respectively.

Table 5.45. Target displacement values for Building 10, computed via different methods.

Target Displacement (m)		
	ASCE 41-13	TSC 2007
MODE-1 (+)	0.5344	0.4356
MODE-1 (-)	0.539	0.4356
MODE-3 (+)	0.36	0.357
MODE-3 (-)	0.36	0.357

The target displacements are multiplied by the torsional amplification multiplier, η and according to the ASCE 41-13 specifications, the damage states in the members are determined in terms of the plastic rotations. The final damage states for members are given in Tables 5.46 and 5.47 below.

Table 5.46. Final damage states of structural members for Building 10 (mode 1).

∑# of Columns	Elastic	IO	LS	CP	FAIL
296	237	16	42	1	0
%	80.07	5.41	14.19	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
224	15	45	31	115	18
%	6.70	20.09	13.84	51	8

Table 5.47. Final damage states of structural members for Building 10 (mode 3).

∑# of Columns	Elastic	IO	LS	CP	FAIL
296	260	36	0	0	0
%	87.84	12.16	0.00	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
256	66	32	104	54	0
%	25.78	12.50	40.63	21	0

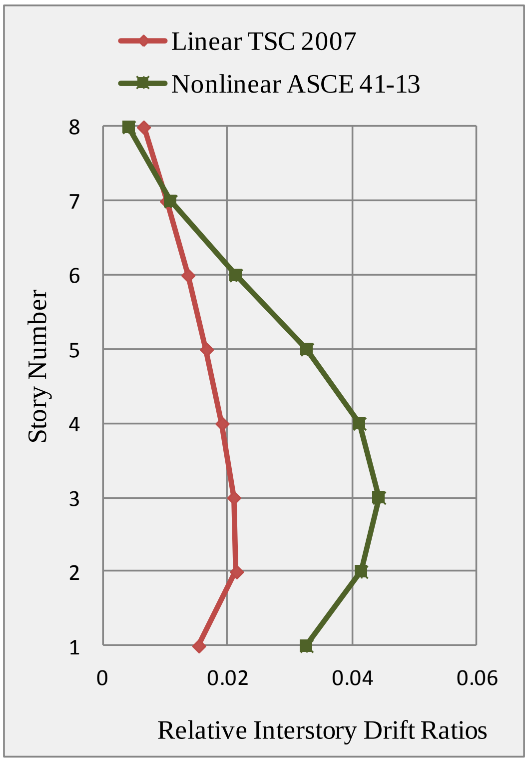

Figure 5.30. Relative interstory drift ratios for Building 10, mode 1.

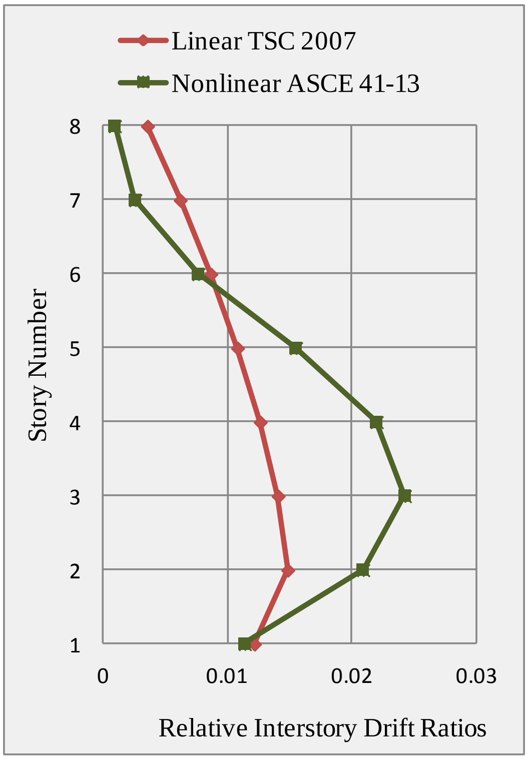

Figure 5.31. Relative interstory drift ratios for Building 10, mode 3.

To sum up, Building 10 is expected to perform within the collapse prevention performance level when the beams are considered, however the overall performance level is hard to specify due to the performances of the columns, and the building is most likely to remain in the life safety – collapse prevention interface.

5.11. Building 11

This structure is identical to Building 8 in plan.

- Number of stories : 4
- Structural system type : Frame (without structural walls)
- Story height : 3.5 m
- Dimensions in plan : 42.65 m by 27.30 m
- Plan area : 1050 m^2
- $R_a(T)$ for preliminary design of member dimensions : 4
- g = 7 kN/m^2, q=3 kN/m^2
- Live load reduction factor, n = 0.3
- Seismic Zone : I (as specified by TSC 2007)
- Maximum span length : 8.90 m

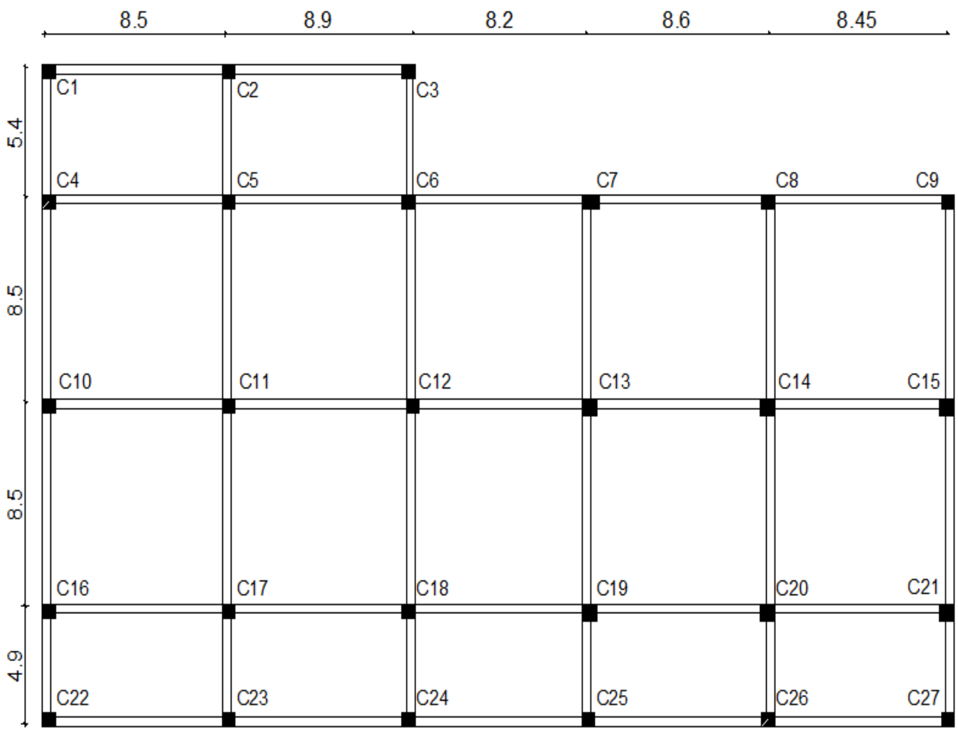

Figure 5.32. Layout of Building 11 (dimensions in m).

The average multiplier for the cracked section rigidities of the columns is found to be 0.351.

Table 5.48. Periods and modal mass participation ratios for Building 11.

Modal Participating Mass Ratios			
Mode	T (s)	Ux	Uy
1	1.198	0.785	0.014
2	1.185	0.019	0.809
3	1.129	0.031	0.013
4	0.368	0.016	0.071

For Building 11, the ratios of the base shear forces obtained from response spectrum analyses with sufficient number of modes satisfying the 90% rule to the base shear forces obtained from single mode response spectrum analyses are 1.079 and 1.025 for mode 1 and mode 2, respectively.

The η values are found to be 1.222 and 1.278 for X and Y directions, respectively.

Table 5.49. Target displacement values for Building 11, computed via different methods.

Target Displacement (m)		
	ASCE 41-13	TSC 2007
MODE-1 (+)	0.314	0.247
MODE-1 (-)	0.319	0.247
MODE-2 (+)	0.304	0.25
MODE-2 (-)	0.308	0.25

The target displacements are multiplied by the torsional amplification multiplier, η and according to the ASCE 41-13 specifications, the damage states in the members are determined in terms of the plastic rotations. The final damage states for members are given in Tables 5.50 and 5.51 below.

Table 5.50. Final damage states of structural members for Building 11 (mode 1).

∑# of Columns	Elastic	IO	LS	CP	FAIL
108	51	13	44	0	0
%	47.22	12.04	40.74	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
88	0	6	19	63	0
%	0.00	6.82	21.59	72	0

Table 5.51. Final damage states of structural members for Building 11 (mode 2).

∑# of Columns	Elastic	IO	LS	CP	FAIL
108	47	14	47	0	0
%	43.52	12.96	43.52	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
84	0	3	11	70	0
%	0.00	3.57	13.10	83	0

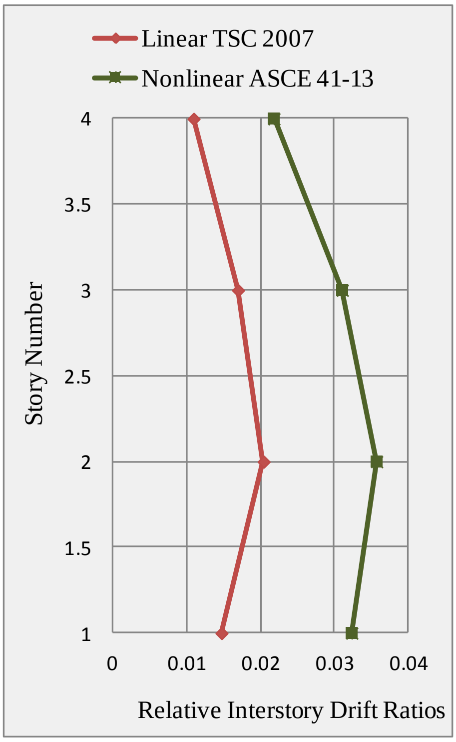

Figure 5.33. Relative interstory drift ratios for Building 11, mode 1.

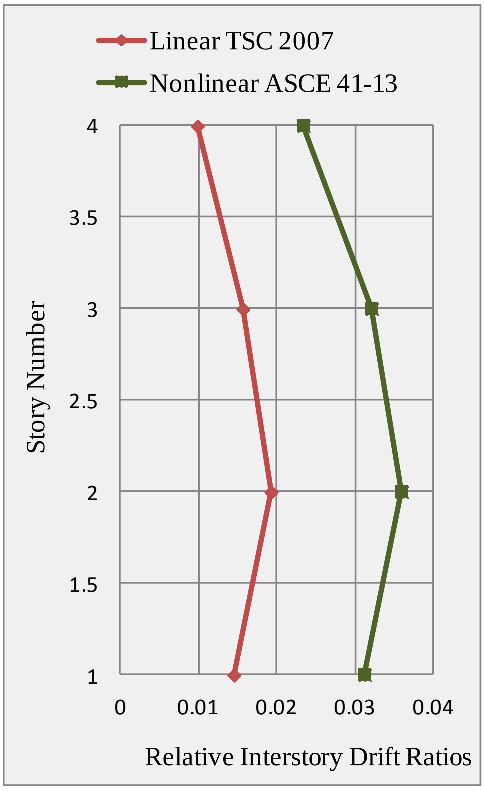

Figure 5.34. Relative interstory drift ratios for Building 11, mode 2.

To sum up, the Building 11 is expected to perform within the collapse prevention performance level when the beams are considered, however the overall performance level is hard to specify due to the performances of the columns, and the building is most likely to remain in the life safety – collapse prevention interface.

5.12. Building 12

This structure is identical to Building 7 in plan.

- Number of stories : 4
- Structural system type : Frame (without structural walls)
- Story height : 3.5 m
- Dimesions in plan : 29.20 m by 10.50 m
- Plan area : 291 m^2
- $R_a(T)$ for preliminary design of member dimensions : 4
- g = 7 kN/m^2, q=3 kN/m^2
- Live load reduction factor, n = 0.3
- Seismic Zone : I (as specified by TSC 2007)
- Maximum span length : 7.50 m

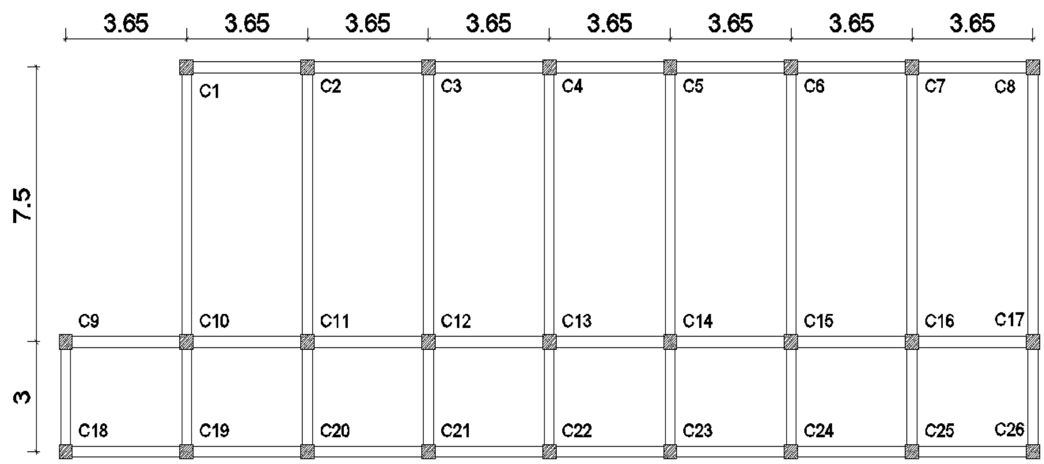

Figure 5.35. Layout of Building 12 (dimensions in m).

The average multiplier for the cracked section rigidities of the columns is found to be 0.316.

Table 5.52. Periods and modal mass participation ratios for Building 12.

Modal Participating Mass Ratios			
Mode	T (s)	Ux	Uy
1	1.013	0.00001	0.879
2	0.917	0.757	0.0008
3	0.9	0.14	0.003
4	0.337	4×10^{-6}	0.09

For building 12, the ratios of the base shear forces obtained from response spectrum analyses with sufficient number of modes satisfying the 90% rule to the base shear forces obtained from single mode response spectrum analyses are 1.015 and 1.193 for mode 1 and mode 2, respectively.

The η values are found to be 1.304 and 1.136 for X and Y directions, respectively.

Table 5.53. Target displacement values for Building 12, computed via different methods.

Target Displacement (m)		
	ASCE 41-13	TSC 2007
MODE-1 (+)	0.196	0.21
MODE-1 (-)	0.196	0.21
MODE-2 (+)	0.194	0.157
MODE-2 (-)	0.195	0.157

The target displacements are multiplied by the torsional amplification multiplier, η and according to the ASCE 41-13 specifications, the damage states in the members are determined in terms of the plastic rotations. The final damage states for members are given in Tables 5.54 and 5.55 below.

Table 5.54. Final damage states of structural members for Building 12 (mode 1).

∑# of Columns	Elastic	IO	LS	CP	FAIL
104	35	20	49	0	0
%	33.65	19.23	47.12	0	0
∑# of Beams	Elastic	IO	LS	CP	FAIL
68	22	12	21	13	0
%	32.35	17.65	30.88	19	0

Table 5.55. Final damage states of structural members for Building 12 (mode 2).

∑# of Columns	Elastic	IO	LS	CP	FAIL
104	37	15	39	10	3
%	35.58	14.42	37.50	9.6	2.885
∑# of Beams	Elastic	IO	LS	CP	FAIL
92	44	26	14	8	0
%	47.83	28.26	15.22	8.7	0

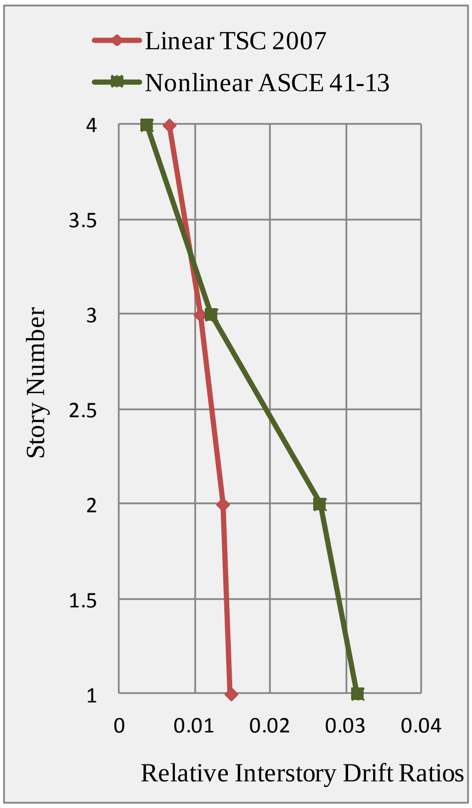

Figure 5.36. Relative interstory drift ratios for Building 12, mode 1.

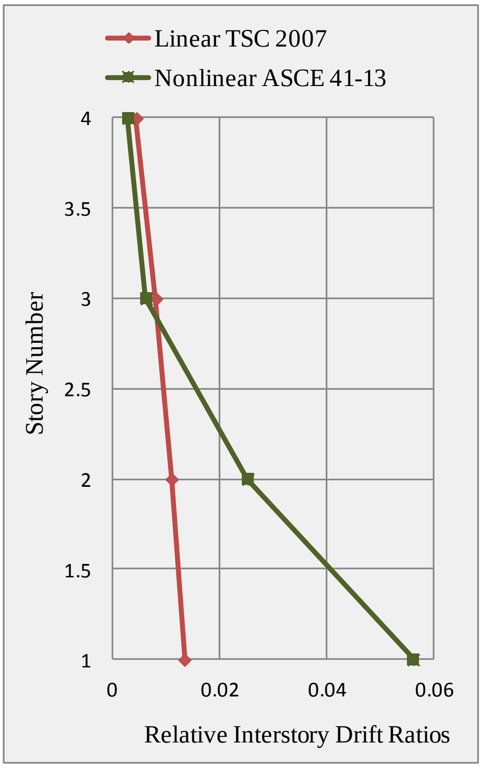

Figure 5.37. Relative interstory drift ratios for Building 12, mode 2.

It can clearly be observed that partial story mechanism forms in the intermediate stories along 2-3 axes (depending on the direction) and excessive story drifts are observed in Building 12. The main reason is that the columns are initially designed according to their total tributary areas along the height. The columns located towards the south have small tributary areas, and in addition the building has 4 stories. This causes the columns to be weak, and consequently a story mechanism occurs. To sum up, Building 12 is expected to perform within or beyond the limits prescribed for the collapse prevention damage state.

5.13. General Summary

In this section, a summary of characteristics and expected performance levels will be given in Table 5.56. Moreover, throughout the Table 5.57 - Table 5.68 the column and the wall sections which were determined according to Section 3, and used in this study will be given.

Table 5.56. Summary of the building characteristics and expected performance levels.

Building ID	Shear Walls	# of Stories	Maximum η	Expected Performance Level
B1	YES	8	1.086	LS
B2	YES	8	1.164	LS
B3	YES	8	1.0	IO-LS
B4	YES	8	1.190	LS
B5	YES	8	1.273	LS
B6	YES	8	1.0	IO-LS
B7	NO	8	1.129	≥ CP
B8	NO	8	1.58	LS-CP
B9	NO	8	1.58	LS-CP
B10	NO	8	1.32	LS-CP
B11	NO	4	1.278	LS-CP
B12	NO	4	1.304	≥ CP

Table 5.57. Column and wall sections of Building 1.

Column ID	b (mm)-X	h (mm)-Y
C1	400	400
C2	350	300
C3	350	300
C4	350	400
C5	350	400
C6	350	400
C7	350	300
C8	450	450
C9	300	300
C10	350	400
C11	400	450
C12	400	450
C13	350	300
C14	350	300
C15	400	450
C16	400	400
C17	400	450
C18	400	450
C19	400	450
C20	350	300
C21	450	450
C22	400	400
C23	350	300
C24	300	300
C25	350	400
C26	350	400
C27	350	400
Wall ID	b (m)-X	h (m)-Y
W1	4.75	0.4
W2	4.75	0.4
W3	4.75	0.4
W4	4.75	0.4
W5	0.4	3.9
W6	0.4	2.8
W7	0.4	2.8
W8	0.4	2.8
W9	0.4	3.9
W10	0.4	2.8

Table 5.58. Column and wall sections of Building 2.

Column ID	b (mm)-X	h (mm)-Y
C1	300	300
C2	300	300
C3	300	300
C4	300	300
C5	300	300
C6	300	300
C7	300	300
C8	300	300
Wall ID	b (m)-X	h (m)-Y
W1	2	0.3
W2	2	0.3
W3	0.3	2
W4	0.3	2

Table 5.59. Column and wall sections of Building 3.

Column ID	b (mm)-X	h (mm)-Y
C1	450	450
C2	450	450
C3	450	450
C4	450	450
C5	500	550
C6	500	550
C7	450	450
C8	450	450
C9	450	450
C10	450	450
Wall ID	b (m)-X	h (m)-Y
W1	3.5	0.3
W2	3.5	0.3
W3	3.5	0.3
W4	3.5	0.3
W5	0.4	5.25
W6	0.4	5.25

Table 5.60. Column and wall sections of Building 4.

Column ID	b (mm)-X	h (mm)-Y
C1	400	400
C2	300	300
C3	300	300
C4	300	300
C5	300	300
C6	300	300
C7	300	300
C8	400	400
C9	300	300
C10	300	300
C11	400	400
C12	350	350
C13	450	450
C14	350	350
Wall ID	b (m)-X	h (m)-Y
W1	4.7	0.4
W2 (UWall) t=0.3m	4.7	2.85
W3	0.3	2.65
W4	0.3	2.65

Table 5.61. Column and wall sections of Building 5.

Column ID	b (mm)-X	h (mm)-Y
C1	300	300
C2	300	300
C3	300	300
C4	450	450
C5	350	350
C6	350	350
C7	450	450
C8	350	350
C9	500	500
C10	400	400
C11	400	400
C12	500	500
Wall ID	b (m)-X	h (m)-Y
W1	2.2	0.3
W2	2.2	0.3
W3	2.1	0.3
W4	2.1	0.3
W5	2.15	0.3
W6	2.15	0.3
W7	0.3	2.15
W8	0.3	2.15
W9	0.3	2.15
W10	0.3	2.15
W11	0.3	2.15
W12	0.3	2.15

Table 5.62. Column and wall sections of Building 6.

Column ID	b (mm)-X	h (mm)-Y
C1	500	500
C2	600	300
C3	600	600
C4	600	600
C5	600	600
C6	600	300
C7	600	300
C8	600	600
C9	600	600
C10	600	600
C11	600	300
C12	500	500
Wall ID	b (m)-X	h (m)-Y
W1	4.02	0.4
W2	4.02	0.4
W3	4.02	0.4
W4	4.02	0.4
W5	0.4	4.02
W6	0.4	4.02
W7	0.4	4.02
W8	0.4	4.02

Table 5.63. Column and wall sections of Building 7.

Column ID	b (mm)-X	h (mm)-Y
C1	350	350
C2	450	450
C3	500	500
C4	500	500
C5	500	500
C6	500	500
C7	500	500
C8	350	350
C9	300	300
C10	500	500
C11	550	550
C12	550	550
C13	550	550
C14	550	550
C15	550	550
C16	550	550
C17	400	400
C18	300	300
C19	300	300
C20	300	300
C21	300	300
C22	300	300
C23	300	300
C24	300	300
C25	300	300
C26	300	300

Table 5.64. Column and wall sections of Building 8.

Column ID	b (mm)-X	h (mm)-Y
C1	500	350
C2	850	400
C3	500	350
C4	650	650
C5	850	1000
C6	800	850
C7	600	850
C8	650	800
C9	350	800
C10	650	800
C11	950	1100
C12	950	1100
C13	950	1100
C14	950	1100
C15	900	600
C16	900	450
C17	900	900
C18	900	900
C19	900	900
C20	900	900
C21	950	450
C22	400	400
C23	450	700
C24	550	550
C25	550	550
C26	550	550
C27	400	400

Table 5.65. Column and wall sections of Building 9.

Column ID	b (mm)-X	h (mm)-Y
C1	400	400
C2	400	400
C3	400	400
C4	400	400
C5	400	400
C6	400	400
C7	400	400
C8	500	500
C9	400	400
C10	500	500
C11	500	500
C12	400	400
C13	400	400
C14	400	400
C15	400	400
C16	500	500
C17	500	500
C18	400	400
C19	400	400
C20	400	400
C21	400	400
C22	400	400

Table 5.66. Column and wall sections of Building 10.

Column ID	b (mm)-X	h (mm)-Y
C1	350	350
C2	450	450
C3	450	450
C4	450	450
C5	450	450
C6	450	450
C7	450	450
C8	450	450
C9	300	300
C10	400	400
C11	550	550
C12	550	550
C13	550	550
C14	550	550
C15	550	550
C16	550	550
C17	550	550
C18	400	400
C19	400	400
C20	550	550
C21	550	550
C22	550	550
C23	550	550
C24	550	550
C25	550	550
C26	550	550
C27	400	400
C28	400	400
C29	400	400
C30	300	300
C31	400	400
C32	450	450
C33	450	450
C34	450	450
C35	450	450
C36	450	450
C37	300	300

Table 5.67. Column and wall sections of Building 11.

Column ID	b (mm)-X	h (mm)-Y
C1	300	300
C2	500	350
C3	300	300
C4	550	400
C5	800	550
C6	600	600
C7	600	450
C8	600	450
C9	300	600
C10	400	700
C11	700	750
C12	700	750
C13	700	750
C14	700	750
C15	350	800
C16	550	400
C17	650	650
C18	650	650
C19	650	650
C20	650	650
C21	550	400
C22	300	300
C23	550	300
C24	550	300
C25	550	300
C26	550	300
C27	300	300

Table 5.68. Column and wall sections of Building 12.

Column ID	b (mm)-X	h (mm)-Y
C1	350	350
C2	400	400
C3	400	400
C4	400	400
C5	400	400
C6	400	400
C7	400	400
C8	400	400
C9	350	350
C10	350	350
C11	400	400
C12	400	400
C13	400	400
C14	400	400
C15	400	400
C16	400	400
C17	400	400
C18	350	350
C19	350	350
C20	350	350
C21	350	350
C22	350	350
C23	350	350
C24	350	350
C25	350	350
C26	400	400

6. SUMMARY AND CONCLUSIONS

6.1. Overview

A simple method for the preliminary design of relatively regular (i.e. simple) low-to-medium rise reinforced concrete buildings is evaluated in terms of seismic performance. The nonlinear static procedure prescribed in ASCE 41-13 is adopted as the evaluation method. The acceptance criteria and the modelling assumptions are also utilized from ASCE 41-13. Within the scope of this study, 10 reinforced real structures, all of them reinforced concrete buildings, are analyzed and assessed. In the selection of these buildings, it is intended to ensure sufficient diversity in terms of the structural characteristics to be able to determine the extent of the proposed simple method.

6.2. Conclusions

The main observations and conclusions are summarized below:

- For the columns of the dual systems, the initial determination of the column sizes are governed by the axial compression restriction, since the columns in the dual system are assumed to receive 30% of the total base shear force.
- For the frame–only buildings, the governing parameter for the column sizes depends on the span length (i.e. column tributary area) and the number of stories. For the buildings with relatively small tributary areas, the governing parameter is the drift limit; accordingly, this is the situation for low-rise buildings. For the other buildings, the axial compression restriction and the requirement derived from the shear capacity/demand relations jointly govern the column sizes.
- Among the buildings considered, the dual systems (frame + structural walls) have performed within the life safety performance level limits in terms of the plastic rotations at the member ends. For buildings with structural walls and which are perfectly symmetrical in plan, the expected performance level of the walls can increase up to immediate occupancy performance level.
- The shear capacities of structural walls designed based on the proposed restrictions have remained well above the demand. Most of the times the shear capacity/shear demand is in the vicinity of 2 to 3. There are two reasons for this over capacity. The

first reason is that, instead of huge and very stiff but few walls, most of the buildings have been designed with more than 4 structural walls (per direction) with relatively small dimensions. Therefore the shear forces are shared among the walls. The second and probably the more important reason is that the structural walls are not heavily reinforced (longitudinal boundary reinforcement ratio is 0.001) and hence the over-strength ratio is relatively small. Therefore the maximum shear demand that a wall is subject to depends on the moment capacity of the walls to satisfy equilibrium (capacity design).

- Story drifts for the dual systems obtained from both linear and nonlinear analyses conform to each other. According to the limits corresponding to the relative interstory drift values obtained from linear elastic analysis prescribed in TSC 2007, the performance level of the buildings with structural walls are slightly less than the immediate occupancy performance level. Moreover, the relative interstory drift ratios obtained from the nonlinear static procedure are in the vicinity of 0.01-0.015, which can be deemed “satisfying”.
- The expected performance of most of the frame buildings are within the life safety limits in terms of the plastic rotations in the column ends. However, when the beams are considered, the expected performance level for most of the beams is within the collapse prevention limits. The expected performances of the remaining beams are beyond the collapse prevention level.
- In 8-story frame buildings, the columns have generally remained elastic while the beams are within or beyond the collapse prevention performance level. The dimensions for the preliminary design of the columns, determined according to the bounds discussed in Section 3, are based on the tributary loads estimated to act on an individual column. Therefore, as the estimated loads on the columns grow, the required area and the corresponding moment capacity increases, which ensures the strong column-weak beam principle in most cases.
- Conversely, if the frame buildings are relatively low or the column tributary areas are small (e.g. Building 12), the estimated load used to dimension an individual column according to Section 3 is low. Consequently, the required column section dimensions either do not satisfy the strong column-weak beam principle or the rotational capacity of that column falls behind the plastic rotation demands. Based

on the observations in this study, the minimum dimension for a column cross section is suggested to be prescribed as 40 cm.

- The strong column-weak beam principle is not likely to be satisfied in the following case: If the two successive span lengths along the same line are noticeably different (which is common in practice), the required beam cross sections for the longer span is generally also used for the shorter span as in the case observed in Building 12. This practice is suitable for the sake of continuity of the beam reinforcement and applicability. However, for the proposed simple method, this fact brings a disadvantage, since the required column sections corresponding to the tributary areas are small while the beam section is noticeably larger. The problem would most likely be mitigated if the minimum dimension for a column cross section were prescribed as 40 cm as suggested in the previous item.
- Equation 3.15 is derived in terms of the gross sectional rigidities since the method is proposed for design. However, as can be seen in the Section 5 the average multiplier on the column cracked section rigidities are about 0.35. Besides, the beam cracked section rigidities are 0.3 as specified in ASCE 41-13. Consequently, the drift results obtained from the nonlinear analysis are noticeably high. It is suggested that Equation 3.15 is modified with respect to the cracked section rigidities so that the lateral stiffnesses of the frame systems are increased.
- The cracked section rigidities significantly change the dynamic characteristics of the buildings. It is observed that, modal mass participation ratios can change on the order of 30 – 40% for some buildings when cracked section rigidities are implemented.
- The beam flexural reinforcement ratio adopted in this study is 0.004 for the top and 0.002 for the bottom. The beam flexural reinforcement ratio or the beam sizes can be increased to improve the beam performances, which generally fall within the collapse prevention performance level. However, increasing the flexural capacity of beams will bring new disadvantages like the violation of the strong column-weak beam principle or increased shear demands in the beams.
- Approximately 3100 beams have been analyzed within this study and checked against shear forces. Table 6.1 presents the observations of beam lengths and the corresponding (required) transverse reinforcement ratios. Based on Table 6.1, it is estimated that 0.002 transverse reinforcement ratio for the beams could be used as

prescribed value. As can be seen from the Table 6.1, even some of the beams with noticeably higher lengths resist the shear forces solely by the contribution of concrete and no reinforcement is required. The main reason behind this is that the long beams have larger cross sections and higher concrete contribution to the shear resistance.

- The detailing of the confinement regions (like beam-column joints, beam ends, column ends, and the wall boundaries) should comply with the design provisions provided by the national or municipal authorities.

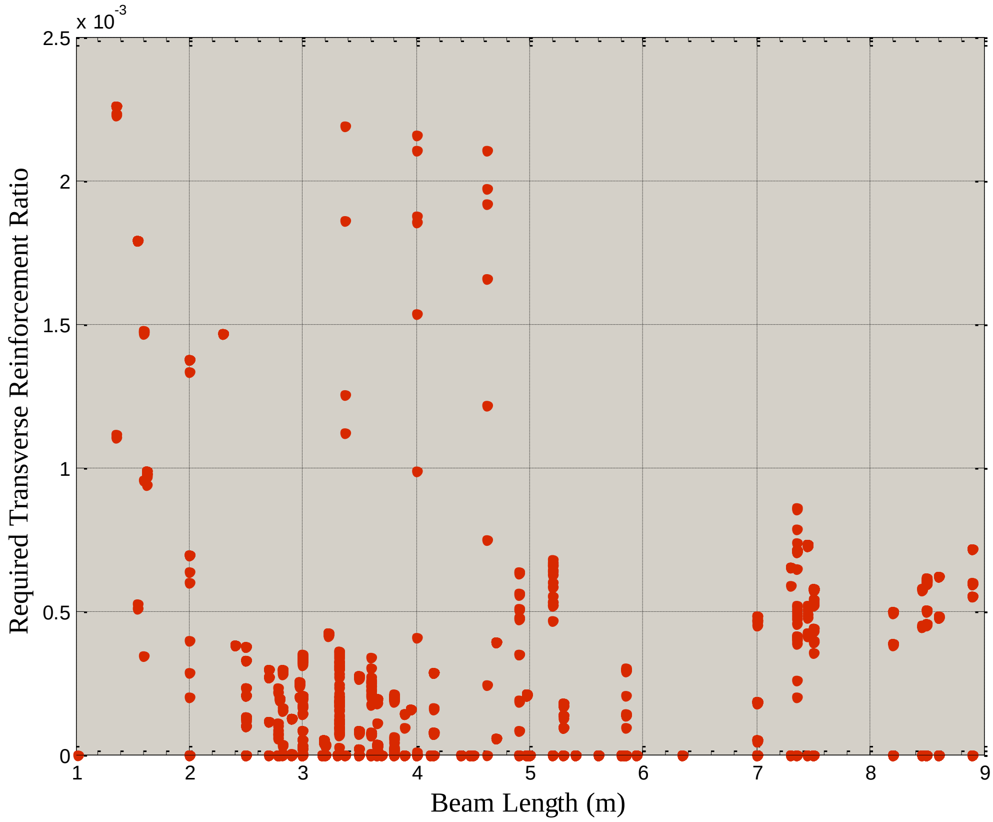

Figure 6.1. Required beam transverse reinforcement ratio.

6.3. Future Recommendations

The recommendations for the proposed simple design method are as stated below.

- The minimum size of a column cross section is should be prescribed to be no less than to be 40 cm.

- The lateral stiffness of the frame systems are should be increased, and Equation 3.15 should be modified in terms of the cracked section rigidities.
- The beam transverse reinforcement ratio should be prescribed as 0.002.
- The flexural reinforcement ratio of the beams or the dimension of the beams should be increased depending on the beam length, provided that the required modifications ensure the strong column-weak beam principle is satisfied.
- For the strong column-weak beam principle, a parametric study involving the frequently used beam sections with various prescribed amounts of flexural reinforcement and relatively small column sections (with 0.01 flexural reinforcement) under various axial loads should be conducted to estimate a minimum column cross section.
- For the low-rise buildings, a minimum tributary area value should be ensured to prevent weak columns and to provide sufficient rotational capacity for the columns.
- A parametric could be conducted to observe the effects of redundancy, which is out of the scope of the study.
- A study including the P-Δ effects could be conducted, especially for the frame buildings, which are significantly vulnerable to interstory drift ratios.
- One bay frames (without structural walls) should not be permitted, since along one side of the building, the columns will be exposed to axial tension under lateral loads.

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