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CODE-BASED EVALUATION OF SEISMIC PERFORMANCE LEVELS OF REINFORCED CONCRETE BUILDINGS WITH LINEAR AND NON-LINEAR APPROACHES

M.Sc. Thesis by
Ahmet Emre TOPRAK, Civil Eng.

Department : **Civil Engineering**

Programme : **Structural Engineering**

FEBRUARY 2008

TECHNISCHE UNIVERSITÄT DRESDEN

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İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

BETONARME BİNALARIN DEPREM PERFORMANS SEVİYELERİNİN DOĞRUSAL VE DOĞRUSAL OLMAYAN YAKLAŞIMLARIN KULLANILARAK BELİRLENMESİ

YÜKSEK LİSANS TEZİ

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PREFACE

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February 2008 Ahmet Emre Toprak

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NOMENCLATURE

$A(T)$	: Spectral acceleration coefficient
A_0	: Effective ground acceleration coefficient
$a_1^{(i)}$	: Modal acceleration of the fundamental period at i-th step
A_C	: Gross section area of column or wall
a_g	: Design ground acceleration on Type A ground
b_w	: Width of beam web
C_{R1}	: Spectral displacement ratio
d	: Effective beam depth
d^*	: Displacement of equivalent single-degree-of-freedom system
d^*_{et}	: Target displacement of the structure with period T^*
$d_1^{(i)}$	: Modal displacement at the fundamental period
$d_1^{(p)}$	: Modal displacement demand at the fundamental period
d_n	: Control node displacement of the multi-degree-of-freedom system
$(EI)_0$	: Bending stiffness for uncracked cross-section
$(EI)_E$	: Effective bending stiffness for cracked cross-section
F^*	: Force of the equivalent single-degree-of-freedom system
F_b	: Base shear force (Eurocode 8)
f_{cm}	: Compression strength of existing concrete
f_{ctm}	: Tensile strength of existing concrete
F_i	: Horizontal seismic force at storey i
h	: Cross-section depth
I	: Building importance factor
L_p	: Length of plastic hinge
L_V	: Moment/shear ratio at the end of section
m^*	: Mass of the equivalent single-degree-of-freedom system
m_i	: Mass of storey i
M_{x1}	: The participated mass at the fundamental period in x direction
n	: Live load participation factor
N_D	: Axial force acting on column or wall, calculated under vertical loads
N_K	: Axial force corresponding to existing material moment capacity
q_i	: Total live load at i-th storey of building
q_u	: Acceleration ratio between structures with elastic and limited stength
R	: Structural behavior factor
r	: Demand / capacity ratio

r_s	: Limit value of demand / capacity ratio
R_{y1}	: Capacity reduction coefficient at the fundamental period
S	: Soil factor
$S(T)$	: Spectrum coefficient
S_{ae1}	: Linear elastic spectral acceleration
$S_d(T)$	: Design response spectrum
S_{de1}	: Linear elastic spectral displacement
S_{di1}	: Non-linear spectral displacement
$S_e(T)$	: Elastic response spectrum
s_i	: Displacement of mass m_i in the fundamental mode shape
T^*	: Period of the idealized equivalent single-degree-of-freedom system
T_1	: Fundamental period
u_{XN1}	: Top displacement value in x direction at i-th step of push-over
V_e	: Design shear force
V_r	: Shear strength of cross-section of column, wall or beam
$V_{X1}{}^{(i)}$	: Base shear at i-th step in x-direction
W	: Total weight of building considering Live Load Participation Factor
w_i	: Weight of the i-th storey considering Live Load Participation Factor
Δ_i	: Normalized displacement of storey i
θ_p	: Plastic rotation demand
θ_y	: Chord rotation at yielding
λ	: Correction factor
ρ	: Tension reinforcement ratio of the beam
ρ'	: Compression reinforcement ratio of the beam
ρ_b	: Balanced reinforcement ratio
ρ_s	: Existing transverse reinforcement ratio
ρ_{sm}	: Required transverse reinforcement ratio
ρ_{sx}	: Ratio of transverse steel parallel to the direction x of loading
ϕ_p	: Plastic curvature demand
ϕ_t	: Total curvature demand
ϕ_y	: Yielding curvature
ω	: Frequency
Γ	: Transformation factor (Eurocode 8)
Γ_{X1}	: Modal participation factor at the fundamental period in x direction
Φ_{XN1}	: Mode shape of N-th storey at the fundamental period in x direction

CODE-BASED EVALUATION OF SEISMIC PERFORMANCE LEVELS OF REINFORCED CONCRETE BUILDINGS WITH LINEAR AND NON-LINEAR APPROACHES

SUMMARY

Determination of seismic performance of existing buildings has become one of the key concepts in structural analysis topics after recent earthquakes (i.e. Northridge Earthquake in 1994, Kobe Earthquake in 1995 and Izmit and Duzce Earthquake in 1999). Considering the need for precise assessment tools to determine seismic performance level, most of earthquake hazardous countries try to include performance based assessment in their seismic codes. Recently Turkish Earthquake Code 2007 (TEC'07), which was put into effect in March 2007, also introduced linear and non-linear assessment procedures to be applied prior to building retrofitting process.

In this thesis study, performance based assessment methods and basic principles given in TEC'07 and Eurocode 8 will be investigated. After the linear elastic approach and non-linear approach will be outlined as given in two codes, the procedures of seismic performance evaluations for existing RC buildings will be applied on a real three dimensional case study building and the results will be compared.

The thesis consists of five chapters. The first chapter presents an introduction and definition of the subject, short review of the previous studies, the scope and the objectives of the study.

The second chapter covers the seismic performance evaluation of existing structures according to TEC'07. The procedure of the equivalent seismic load method in linear static approach and the incremental in non-linear static analysis is explained and investigated.

In the third chapter, the seismic performance assessment procedure is explained briefly as prescribed in Eurocode 8. The lateral force method of analysis and the push-over analysis are overviewed in addition to general assessment principles and rules.

The fourth chapter is devoted to the solution of numerical example as a case study. Seismic performance evaluations according to Eurocode 8 and TEC'07 will be applied on existing building which experienced the seismic action of Ms = 6.3 with a maximum acceleration of 0.28 g during in Adana Ceyhan Earthquake of 1998. The case study building has six storeys with a total of 14.65 m height and it is composed of orthogonal frames, symmetrical in y direction and does not have any significant structural irregularities. The planar dimensions are 16.40 x 7.80 m = 127.90 m^2 with

five spans in x and two spans in y directions. It was reported that retrofitting process is suggested for the residence building because of the moderate damage level. In this chapter, the linear static and non-linear static methods of analysis are applied on the residential building according to TEC'07 and Eurocode 8.

The fifth chapter presents the final results and the discussions of the study. The basic features of the study, the evaluation of the numerical results and possible extensions of the study are presented in this chapter.

The basic conclusions of the numerical evaluations are summarized below.

a. The computations show that the performing methods of analysis with linear and non-linear approaches using either Eurocode 8 or TEC'07 independently produce a very similar performance levels for the critical storey of the structure. The case study building is found to be as in collapse level.

b. The computed base shear value according to Eurocode is much higher than the Turkish Earthquake Code while the selected ground conditions represent the same characteristics. The main reason is that the ordinate of the horizontal elastic response spectrum for Eurocode 8 is increased by the soil factor.

c. According to the displacement-based non-linear assessment described in TEC'07, the strains at plastic cross-sections are to be verified; however, the chord rotations of primary ductile elements must be checked for Eurocode safety verifications.

d. The demand curvatures obtained from linear and non-linear methods of analysis of Eurocode 8 together with TEC'07 are almost similar.

1. INTRODUCTION

1.1 Topic of the Study

Performance based design and assessment in structural engineering is becoming more important in the past several years. The concept which was born in the United States can be defined simply as the design and assessment of a structure regarding one or more performance levels that are foreseen. The latest earthquakes has shown that, even though the structures in the industrial countries were built in an adequately safe fashion, the costs occurring with damage from the quakes as well as the recession of using the buildings for some time has become somewhat difficult to tolerate. In this case, it has become clear that it was necessary to design the structures with respect to different limit states, [1], [2].

Damage conditions which are the primary factor in the determination of structure seismic performance are most realistically expressed as displacement and deformations. For this reason, the use of the analysis tools in the principle of displacement based assessments as well in the decision of the analysis is in at most importance. On the other hand, with the aid of evaluations based on non-linear theory, the behavior of the structural system under the external loads and earthquake effects can be closely monitored, the earthquake performances regarding the displacements and strain can be realistically determined.

Performance based structural design and the decision of the analysis for assessment being a new topic. While linear elastic methods of analysis have been used for long time, on the other hand, non-linear non-elastic analysis procedure has been widespreadly used the last couple of years. The main reason for this is the availability of suitable analysis tools with respect to both non-linear static (pushover) analysis and non-linear dynamic (time history) analysis for methods of analysis.

The most reliable method is non-linear dynamic (time history) analysis of structures under earthquake loads. But this method has its difficulties like getting the suitable surface motion entries, modeling the structure's circumference, and the time consuming calculations. That is why non-linear static (pushover) analysis is mostly preferred. Non-linear static (pushover) analysis shows a simple approach regarding the displacement demands under the dynamic loads and the determination of the structure capacity. The method is applying a foresought lateral load distribution and pushing of the structure up to target displacement.

1.2 Previous Research Work on the Subject

Studies of the methods intending to evaluate structural systems based on non-linear theory have a long history. The analysis methods that have been developed can be divided into two groups regarding their primary hypothesis:

a) the approach which guesses that the non-linear displacements spread in the system continuously

b) the methods that are based on the plastic hinge hypothesis [3]

In parallel to developing these methods, practical and effective computer programs based on the non-linear theory are continuously improving and are widely used, [4], [5].

Structural assessment and design concept with the principle of performance criteria based on the displacement and strain are especially put forward and developed for the realistic safety and rehabilitation of structures in the United States' earthquake regions.

The damage caused by the 1989 Loma Prieta and 1994 Northridge In the state of California – Unites States, made it possible to reconsider not only the current performance criteria regarding the strength of materials but also add more realistic criteria based on displacement and strain.

In this concept, Guidelines and Commentary for Seismic Rehabilitation of Buildings – the ATC 40 [6] Project by the Applied Technology Council (ATC), and NEHRP

Guidelines for the Seismic Rehabilitation of Buildings – FEMA 273 [7] and 356 [8] by the Federal Emergency Management Agency (FEMA) have been developed. Later on, in order to examine the results further on, the ATC 55 and FEMA 440 [9] have been developed. Besides these organizations, different projects like Building Seismic Safety Council (BSSC), American Society of civil Engineers (ASCE) and Earthquake Engineering Research Center of University of California at Berkeley (EERC-UCB) contributed them. With the aid of these projects and papers, the assessment of the performance the existing structures at the quake zones and the redesigning of buildings according to their earthquake performances could be possible.

On the other hand, there also exist some approaches and researches and assessments regarding the performances of structures at the Eurocode 8.3 [10] which is among the standards of the European Union. Eurocode 8 proposes displacement-based approaches for the seismic assessment and retrofit of existing buildings. The seismic effects does not represent a set of lateral loads to be resisted by the structure, as defined in forced-based design or assessment, but a demand of dynamic displacements. Therefore displacements represent a much more realistic for the seismic design or assessment of structures. Eventually, buildings do not collapse due to lateral loads, but due to vertical loads acting under horizontal displacements [11]. Additionally, displacement-based approach fulfills the deficiencies of conventional force-based approach [12].

Recent earthquakes which occurred in our country made it compulsory to assess the safety of structures. Thus, in addition to Turkish Earthquake Code 1998, articles have been added and therefore the Turkish Earthquake Code 2007 [13] has been developed for the assessment and rehabilitation of structures. Researches states that both linear and non-linear static analysis of methods under scope of TEC'07 generally results with same performance levels. However, it is noted that linear analysis method is more relatively more conservative on the basis of component performance damage level [14]. Additionally, both non-linear static (push-over) and dynamic (time history) analyses produce very similar results on component-end damage levels and structure top displacement values for low-rise regular buildings, [15].

Numerical studies comparing FEMA 356 and TEC using non-linear static analysis method shows that both codes results with almost similar damage levels on the basis of structural elements [16].

In addition to code-based linear and non-linear approaches, a preliminary assessment technique is developed by Bal, Tezcan and Gülay [17] to prevent life-loss on existing buildings. The method itself consist of 25 parameters including soil and topographic conditions, earthquake demand, various structural irregularities, material and geometrical properties and location of the buildings. Further researches note that the method results also correlates with code-based linear static and non-linear methods of analysis [18].

Non-linear static method of analysis, which is mainly based on single-mode push-over analysis, has the advantage of establishing elastic response spectrum in estimating the inelastic demand compared to rather time consuming non-linear dynamic (time history) analysis. Therefore, push-over analysis provides an easy and time saving solution. On the other hand, single-mode push-over analysis gives reliable results only when applied to low-rise buildings regular in plan [19] [20] [21]. Before non-linear static approach was introduced to Turkish Earthquake Code, Özer, Pala and others developed incremental load method based on non-linear theory and applied on several 3-D structures to determine their seismic performances [22].

A recent research notes that application of single-mode push-over analysis to high-rise buildings and also to any building irregular in plan-wise leads to incorrect results. Therefore, an improved push-over analysis procedure also contributes the effect of higher modes is required. However, only two procedures up to date, *Modal Push-over Analysis (MPA)* method by Chopra and Goel [23] and *Incremental Response Spectrum (IRSA)* by Aydınoğlu [24] [25] provide the requirements.

For the determination of the performances of buildings, the reliability of the methods mentioned at the code which has been stated above has been widely argued and researched among scientists and academics [26].

1.3 Aim and Scope of the Work

Aim of this study is to investigate the code-based procedure of seismic performance assessments of existing buildings and to determine the seismic performance levels of a case study reinforced concrete building, which represents typical existing building stock in Turkey, using the new Turkish Earthquake Code of 2007 (TEC'07) and Eurocode 8 as well as comparing the consequences of linear static and non-linear static analysis procedures. The investigation is held by using methods of analysis according to Turkish Earthquake Code and Eurocode 8.

The study consists of following steps:

a) Describing analysis procedures for seismic assessment of existing buildings according to TEC 2007.

b) Reviewing the scope of seismic assessment of existing building procedures according to Eurocode 8.

c) Introducing and describing the case study building which experienced Adana-Ceyhan Earthquake of 1998.

d) Evaluating the seismic performance of the existing building according to Eurocode 8 and TEC'07 with linear and non-linear approaches.

e) Reviewing and comparing the results obtained from the analysis.

f) Presenting conclusion remarks regarding to the study.

2. SEISMIC ASSESSMENT OF EXISTING BUILDINGS ACCORDING TO TEC 2007

In Turkey, especially after 1999 Adapazari-Kocaeli and Duzce Earthquakes, practical applications for earthquake risk assessments and retrofitting of insufficient buildings have been significantly increased. However, since there were neither existing regulations nor codes regarding to assessment of existing buildings, these applications were performed under the basis of Turkish Earthquake Code 1998 which was actually aimed for new building design procedures. To prevent upcoming possible inconveniences later on, beginning from 2003, researches and studies to include a new chapter concerning the assessment and retrofitting for existing buildings have been completed.

On following paragraphs, general rules and applications of performance based assessment that is included in Turkish Earthquake Code 2007 (TEC'07) are presented [13].

2.1 Obtaining As-built Information and Knowledge Levels

In order to evaluate the seismic performance of existing buildings, information about structural system geometry, component cross-sections, characteristics of materials and soil conditions can be achieved from available building projects, reports or from in-situ tests and visual inspections. Due to the comprehensiveness of obtained as-built information, knowledge levels and corresponding confidence factors are summarized as follows: Table 2.1.

Table 2.1 : Knowledge Level Confidence Factors

Knowledge Level	Confidence Factor
Limited	0.75
Medium	0.90
Comprehensive	1.00

2.2 Damage Levels of Structural Elements

Building seismic performance evaluation is generally determined with two different criteria. In force-controlled evaluation, capacities of structural elements are compared with linear elastic seismic demands. Verifications are made with consideration of components' ductility and with demand reduction factors under based on each structural component. On the other hand, displacement-controlled evaluation, which constitutes the fundamentals of non-linear analysis methods, the component performance is determined by a nonlinear analysis procedure whereas the deformation demands are checked.

At both approaches, damage limits and levels are defined for structural elements. Before safety verifications, structural elements are first classified as "ductile" or "brittle".

2.2.1 Cross-sectional Damage Limits

For ductile elements, there are three damage levels defined under the basis of their cross-section. These are *Minimum Damage Limit (ML), Safety Limit (SL)*, and *Collapse Limit (CL)*. Minimum damage limit describes the beginning of post-elastic behavior of the cross-section, safety limit describes the limit of non-elastic behavior that can carry demands safely, collapse limit is describes the beginning of collapse state.

2.2.2 Damage Levels

Components that have lower damage than ML are at Slight Damage (SL) level, that have damage between ML and SL are at Moderate Damage (MD) level, that are between SL and CL belong to Heavy Damage (HD) level, and rest of them are considered as at Collapse (CD) level. Figure 2.1

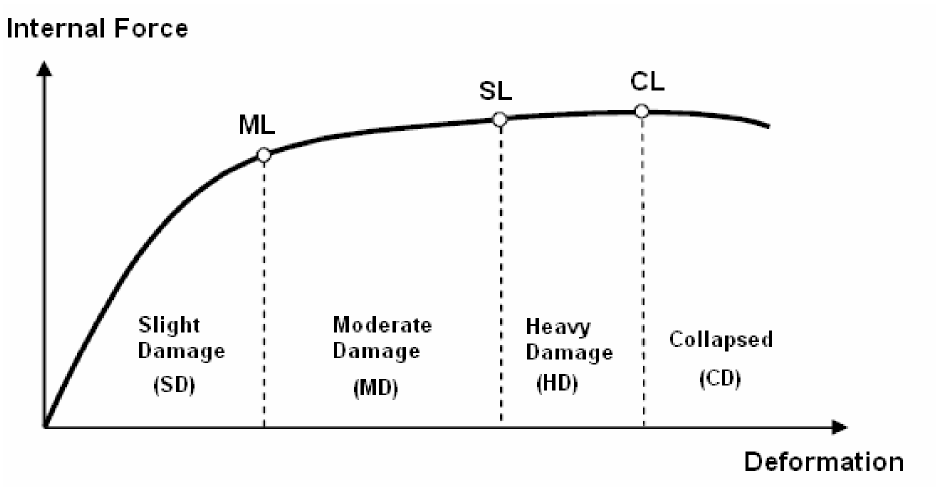

Figure 2.1 : Cross-sectional Damage Levels

2.3 Seismic Performance Levels of Buildings

Seismic performance level of a building is the state of damage ratio limits under a predicted seismic action effects. These limit states are determined due to the measure of structural and non-structural element damage, its influence to risk for life safety, probability of building being operational or not, after the earthquake, and due to the economical loss, [27].

Turkish Earthquake Code 2007 defines the seismic performance as the expected structural damage under considered seismic actions. Seismic performance of a building is determined by obtaining story-based structural element damage ratios under a linear or non-linear analysis.

2.3.1 Immediate Occupancy Performance Level

If the damages occurred at structural elements are all at minimum and those elements keep their initial stiffness and capacity properties, and there are no permanent plastic deformations observed the structural system is defined as at *Immediate Occupancy Performance Level*. Some elements may exceed their yielding capacities and there may be some cracks observed at some non-structural elements, however these damages are at repairable level.

For each main direction that seismic loads affect, at any storey at most 10% of beams can be at moderate damage level; however, the rest of the structural elements should be at slight damage level. With the condition of brittle elements to be retrofitted (reinforced), the buildings at this state are assumed to be at *Immediate Occupancy Performance Level*.

2.3.2 Life Safety Performance Level

Under applied seismic actions, some of the structural elements are damaged; however these elements mostly keep their initial horizontal stiffness and capacity properties. Vertical elements are adequate for axial forces. Non-structural elements may be fairly damaged, yet in-filled walls do not collapse. There may be some plastic deformations, but they are not distinguishable.

For each main direction that seismic loads affect, at any storey at most 30% of beams and some of columns can be at heavy damage level; however, shear contributions of overall columns at heavy damage must be lower than 20%. The rest of the structural elements should be at slight or moderate damage levels. With the condition of brittle elements to be retrofitted, buildings at this state are assumed to be at *Life Safety Performance Level*. For the validity of this performance level, the ratio between the shear force contribution of a column with moderate or higher damage level from both ends and the total shear force of the corresponding storey must be at most 30%. This ratio can be permitted up to 40% at the top storey.

2.3.3 Collapse Prevention Performance Level

Under applied seismic actions, some of the structural elements are damaged. Some of these elements lose their initial horizontal stiffness and capacity properties. Vertical elements are adequate for axial forces, yet some of them reach to their axial load capacities. Non-structural elements are damaged and some of existing in-filled walls may fail. Permanent drifts and deformations occur on the structure itself.

For each main direction that seismic loads affect, at any storey at most 20% of beams can collapse. Rest of the structural elements should be at slight damage, moderate damage, or heavy damage levels. With the condition of brittle elements to be retrofitted, the buildings at this state are assumed to be at *Collapse Prevention Performance Level*. For the validity of this performance level, the ratio between the shear force contribution of a column with moderate or higher damage level from both ends and the total shear force of the corresponding storey must be at most 30%. Functionality of a building at this performance level has risks for life safety and it should be strengthened. Cost-effective analysis is also recommended for such seismic rehabilitation.

2.3.4 State of Collapse

Under applied seismic actions, structure reaches the state of collapse. Some of the vertical structural elements fail. Remaining vertical structural elements still able to carry vertical loads; however, their rigidities and capacities are significantly reduced. Most of the non-structural elements are collapsed. Permanent drifts and deformations significantly occur on the structure itself. Building may either be totally collapsed or is about to collapse under upcoming slight ground motion effects.

Whenever a building fails to achieve collapse prevention performance level, then it is assumed to be in *State of Collapse*. The functionality of a building at this performance level has risks for life safety and it should be strengthened. However, seismic rehabilitation may not be effective in comparison with costs.

2.4 Return Periods of Earthquakes to be Used in Building Assessments

For performance-based designs and assessments, three different return periods of earthquakes are stated. The return periods are generally described by the probability of exceedence of 50 years or by the time interval between two corresponding type of earthquakes.

- Occasional Earthquake: Return period of 72 years, corresponding to a probability of exceeding 50% in 50 years.

- Rare Earthquake: Return period of 475 years, corresponding to a probability of exceeding 10% in 50 years.

- Very Seldom Earthquake: Return period of 2475 years, corresponding to a probability of exceeding 2% in 50 years.

2.5 Minimum Seismic Performance Requirements

Earthquake return periods and required performance levels to corresponding existing building types are given below. Table 2.2.

Table 2.2 : Required Seismic Performance Levels

PURPOSE OF OCCUPANCY	PROBABILITY OF EXCEEDENCE		
	50% in 50 yrs	10% in 50 yrs	2% in 50 yrs
OPERATIONAL AFTER EARTHQUAKE	-	IO	LS
CROWDED FOR LONG-TERM	-	IO	LS
CROWDED FOR SHORT-TERM	IO	LS	-
CONTAINS HAZARDOUS MATERIAL	-	IO	CP
OTHER	-	LS	-

2.6 Methods of Analysis

On the following paragraphs, first the principles and general rules for linear and non-linear analysis methods stated in Turkish Earthquake Code 2007 will be described. Then, the procedures for determination of the seismic performance due to linear analysis methods will be explained. Finally, calculations with non-linear analysis methods will be defined step by step.

2.6.1 General Rules for Linear and Non-linear Analysis Methods

Turkish Earthquake Code 2007 recommends linear and non-linear methods of analysis in order to determine seismic performance of existing buildings. It is not expected that the methods give the same results, since the approaches are theoretically different. The principles and rules valid for both linear and non-linear approaches are given below:

- Within the definition of seismic actions, demands are taken from earthquake with probability of exceedence of 10% in 50 years is used with using elastic (unreduced) response spectrum. For the earthquake with probability of 50% and 2% of exceedences in 50 years, the spectrum function ordinates are to be multiplied by 0.5 and 1.5, respectively. On the other hand, the building importance factor I is not applied, or it is considered as unity.

- The building seismic performance is evaluated under combinations of both vertical and earthquake loads. Storey masses are to be defined properly with constant dead loads with proper participation of live loads.

- Seismic loads are considered as acted on buildings in two main directions, separately.

- On buildings that slabs are defined as rigid diaphragm, two horizontal displacement degrees of freedom and one vertical rotational degree of freedom may be taken into account. Degrees of freedom are defined at each storey center of mass and accidental eccentricity is not applied.

- The uncertainties from as-built information are influenced to assessment by the confidence factors related to knowledge levels.

- The existing short columns in the buildings are considered with their own heights in the mathematical models.

- Interaction curves of reinforced concrete cross-sections under bending moments and axial forces are evaluated with following principles.

    1. Mean values of material properties shall be used.

    2. The ultimate compression strain of concrete and the ultimate tension strain of steel materials may be taken as 0.003 and 0.01, respectively.

    3. Interaction curves can be multi-lined properly to obtain either multi-lined planes or multi-planed surface.

- Unless a more detailed research is performed, the effective elastic stiffness for cracked cross-sections of reinforced concrete elements under flexure with axial force must be used.

    a) At beams: $(EI)_e = 0.4\ (EI)_0$

    b) At columns and shear walls:
    $(EI)_e = 0.40\ (EI)_0$ if $N_D / (A_c\ f_{cm}) \leq 0.10$
    $(EI)_e = 0.80\ (EI)_0$ if $N_D / (A_c\ f_{cm}) \geq 0.40$

Straight line interpolation can be used for intermediate values of N_D. N_D is the axial force, determined by a preliminary analysis under vertical loads compatible with the masses.

2.6.2 Linear Analysis Methods

The suggested methods of linear analysis introduced in Turkish Earthquake Code 2007 are *Equivalent Seismic Load Method* and *Mode Superposition Method*. The main objective of these methods is to compare demands by using unreduced elastic response spectrum with the existing capacity of elements, then to evaluate damage levels on the basis of elements with obtained demand-capacity ratios, and to determine the seismic performance level of the considered overall building.

2.6.2.1 Equivalent Seismic Load Method

The equivalent seismic load method may be applied to buildings whose height is lower than 25 meters and with number of storey not more than 8. Additionally, torsional irregularity factor in plan must be lower than 1.40. In determination of base shear force, unreduced (elastic) response spectrum function must be used and the right hand side of the equation must also be multiplied with λ coefficient. The value of λ can be taken as 1.00 for buildings with 2 storeys or lower excluding basement, and 0.85 for other higher buildings.

2.6.2.2 Mode Superposition Method

In mode superposition method, elastic (unreduced) elastic response is used. For determining internal forces and capacities of elements regarding to a direction of a seismic effects, internal force directions will be taken into account for the fundamental mode relating to the corresponding direction.

2.6.2.3 Determination of Damage Levels of Structural Elements

Denoting by (r), the ratio of the demand obtained from the analysis under the seismic loads, over the capacity of the same ductile element is used in order to determine the damage level of the corresponding element.

Elements are classified as ductile or brittle due to their failure types. To verify elements as ductile, shear force V_e which is related to member end moment capacities must be lower than the shear force resistance V_r determined by using formula as stated in Turkish Standard TS-500 "Requirements for Design and Construction of Reinforced Concrete Structures" [28]. Whenever the shear demand, which is obtained by using vertical and seismic loads as in combinations, is lower than V_e; shear demand value will be used instead of V_e. The elements that are not verified upon these general acceptance rules are defined as "elements under brittle failure".

Demand – capacity ratio (DCR) is obtained by dividing moments from unreduced seismic actions at element end cross-sections to residual moment capacities. Residual moment capacity is the difference between cross-sectional total bending moment capacity and the demand moments under vertical loads. Due to the verifications for horizontal reinforcement configuration acceptance criteria, element ends are classified as "confined" and "unconfined".

The calculated (r) values are to be compared with damage level limit values (r_s) as given in to decide the damage levels of each structural member.

Damage level limits for ductile beams, columns, and shear walls are given on Table 2.3, Table 2.4 and Table 2.5 respectively.

Table 2.3 : Demand-Capacity Ratio and Damage Level Limits (r_s) for Beams

DUCTILE BEAMS			DAMAGE LIMIT		
$(\rho-\rho')/\rho_b$	**CONFINED**	**$V_e/(b_w{*}f_{ctm})$**	**ML**	**SL**	**CL**
≤0.0	YES	≤0.65	3.0	7.0	10.0
≤0.0		≥1.30	2.5	5.0	8.0
≥0.5		≤0.65	3.0	5.0	7.0
≥0.5		≥1.30	2.5	4.0	5.0
≤0.0	NO	≤0.65	2.5	4.0	6.0
≤0.0		≥1.30	2.0	3.0	5.0
≥0.5		≤0.65	2.0	3.0	5.0
≥0.5		≥1.30	1.5	2.5	4.0

Table 2.4 : Demand-Capacity Ratio and Damage Level Limits (r_s) for Columns

DUCTILE COLUMNS			DAMAGE LIMIT		
$N_K/(A_c*f_{cm})$	CONFINED	$Ve/(b_w*d*f_{ctm})$	ML	SL	CL
≤ 0.1	YES	≤ 0.65	3.0	6.0	8.0
≤ 0.1		≥ 1.30	2.5	5.0	6.0
≥ 0.4 and ≤ 0.7		≤ 0.65	2.0	4.0	6.0
≥ 0.4 and ≤ 0.7		≥ 1.30	1.5	2.5	3.5
≤ 0.1	NO	≤ 0.65	2.0	3.5	5.0
≤ 0.1		≥ 1.30	1.5	2.5	3.5
≥ 0.4 and ≤ 0.7		≤ 0.65	1.5	2.0	3.0
≥ 0.4 and ≤ 0.7		≥ 1.30	1.0	1.5	2.0
≥ 0.7	-	-	1.0	1.0	1.0

Table 2.5 : Demand-Capacity Ratio and Damage Level Limits (r_s) for Shear Walls

DUCTILE SHEAR WALLS	DAMAGE LIMIT		
boundires are confined	ML	SL	CL
YES	3.0	6.0	8.0

In calculations using linear elastic methods, relative drift ratios of vertical components under any direction of seismic actions must not exceed the value for the corresponding damage limits given in Table 2.6.

Table 2.6 : Relative Storey Drift Ratio Limits

relative storey drift ratio	DAMEGE LIMIT		
	ML	SL	CL
δ_i / h_i	0.01	0.03	0.04

2.6.3 Non-linear Analysis Methods

The main objective of non-linear analysis methods is to attain plastic deformation demands in ductile members and internal force demands in brittle members under the expected seismic actions. After determining these values, the demands are compared with the existing element capacities in order to decide their damage.

Incremental Equivalent Seismic Load Method, Incremental Mode Superposition Method and *Time History Analysis Method* are introduced as non-linear methods of analysis within the scope of Turkish Earthquake Code 2007. First two methods are based on push-over analysis for verifying seismic performance levels and structural interventions. In the following paragraphs, procedures of push-over analysis with

Incremental Equivalent Seismic Load Method will be introduced and described in detail under the scope of this thesis study.

2.6.3.1 Assessment Procedures for Push-over Analysis

Procedures for seismic performance assessment based on pushover analysis are summarized below;

a) Besides general scope and rules, idealization of non-linear behavior and creation of mathematical model must also be complied.

b) Before push-over analysis, a non-linear static analysis under gravity loads is performed. Results of this analysis are taken as initial conditions for the following push-over analysis.

c) Modal capacity curve, which is coordinated with modal displacement and modal acceleration, must be obtained in order to determine the target displacement under corresponding seismic actions. Then, the plastic deformation and internal force demands at target displacement are calculated.

d) Plastic rotation demands are obtained from plastically deformed ductile cross-sections. From plastic rotations, plastic curvature values are calculated. Later on, total curvature demands are determined. At the end, those curvature demands are converted to strains occurred at concrete and reinforcement bars at the corresponding cross-sections. These strain demands are compared with the limit strain values in order to specify the member end damage levels.

2.6.3.2 Idealization of Non-linear Behavior

Idealization of non-linear behavior is based on *lumped plastic behavior model* since it is more practical than the distributed plastic hinge model and widely used in engineering applications. Deformations are assumed to be constant along the plastic hinge length where internal forces at beams, columns and walls exceed yield capacities. The length of plastic hinge (L_p), can be taken as half of the cross-section depth (h) [13].

$$L_p = 0.5 \cdot h \qquad \textbf{(2.1)}$$

The plastic cross-section, which represents lumped plastic deformation, theoretically must be located in middle of plastic deformation zone. However, replacing them at both ends of beams and columns is also acceptable for practical applications.

Interaction curves of reinforced concrete cross-sections under bending moments and axial forces are evaluated with following principles:

a) Mean values of material properties modified with knowledge confidence factor shall be used.

b) Ultimate compression strain of concrete and ultimate tension strain of steel materials may be taken as 0.003 and 0.01, respectively.

c) Interaction curves can be poly-lined properly to obtain either multi-lined plane or multi-planed surface.

The following idealizations are applicable for internal force – plastic deformation relations:

Strain hardening of steel in internal force - plastic deformation relations can be neglected, (Figure 2.2). In that case, plastic deformation vector is assumed to be approximately perpendicular to yielding surface.

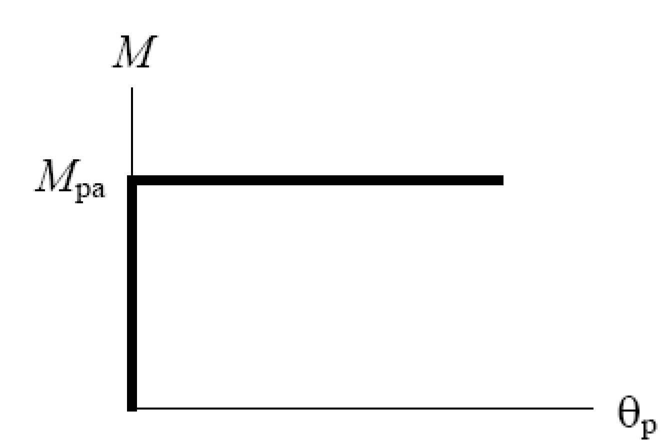

Figure 2.2 : Bending Moment – Plastic Rotation Relation (without hardening)

When strain hardening effect is considered (Figure 2.3), conditions that plastic deformation vector must approve can be defined from the related literature.

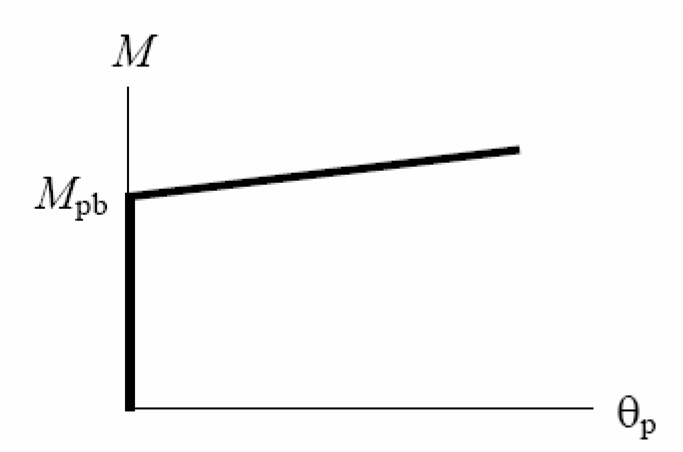

Figure 2.3 : Bending Moment – Plastic Rotation Relation (with hardening)

2.6.3.3 Push-over Analysis with Incremental Equivalent Seismic Load Method

The objective of *Incremental Equivalent Seismic Load Method* is to perform a non-linear analysis with monotonically increasing equivalent seismic loads until the target displacement is reached. The equivalent seismic loads must be compatible with the fundamental mode shape. Following of the vertical load analysis, at each step of the push-over analysis, the maximum top displacement values, plastic deformations and the internal forces are obtained until the target displacement is reached.

The incremental Equivalent Seismic Load Method can be applied to the buildings with 8 storey or less and a torsional irregularity factor in plan lower than 1.40. Additionally, the mass participation ratio corresponding to the fundamental mode in each direction must be at least 70%.

After the performance of the push-over analysis under constant load distribution ratio, a *push-over curve* is obtained. This push-over curve is coordinated with top displacement versus the base shear. The top displacement is the calculated lateral displacement of center of mass at the top floor in the considered earthquake direction. The base shear is the sum of the equivalent seismic loads acting at each step in each considered direction. *Modal Capacity Curve (modal displacement vs* *modal acceleration)*, which is obtained from the push-over curve with coordinate conversions, can be sketched with following procedure:

The modal acceleration of the fundamental period at i-th step $a_1^{(i)}$ can be calculated as follows:

$$a_1^{(i)} = \frac{V_{x1}^{(i)}}{M_{x1}} \tag{2.2}$$

Where $V_{x1}^{(i)}$ is the base shear at i-th step in x direction, and M_{x1} is the participated mass at the fundamental period in x direction.

Modal displacement of the fundamental period at i-th step $d_1^{(i)}$ can be calculated as follows:

$$d_1^{(i)} = \frac{u_{xN1}^{(i)}}{\Phi_{xN1} \cdot \Gamma_{x1}} \tag{2.3}$$

Γ_{x1} is the modal participation factor of the fundamental period in x direction, and Φ_{xN1} represents the modal shape of N-th storey at the fundamental period of x direction. $u_{xN1}^{(i)}$ is the top displacement value in x direction, obtained from i-th step of the push-over analysis.

With *the modal capacity curve* and *the spectral behavior curve* drawn on the same scale together, the maximum *modal displacement demand* is achieved. By definition, the modal displacement demand $d_1^{(p)}$ is equal to *the non-linear spectral displacement* S_{di1}.

Non-linear spectral displacement S_{di1} is dependent on the linear elastic spectral displacement S_{de1} which is related to the first step of push-over analysis.

$$S_{di1} = C_{R1} \cdot S_{de1} \tag{2.4}$$

Linear elastic spectral displacement S_{de1}, is calculated with the help of linear elastic spectral acceleration S_{ae1}.

$$S_{de1} = \frac{S_{ae1}}{\left(\omega_1^{(1)}\right)^2} \tag{2.5}$$

Spectral displacement ratio C_{R1} is determined by fundamental period $T_1^{(1)}$. In case of fundamental period is being equal or longer than the characteristic period T_B, on basis of equal displacement rule, nonlinear spectral displacement S_{di1} is equal to linear elastic spectral displacement S_{de1}. As a result, spectral displacement ratio is taken as:

$$C_{R1} = 1 \tag{2.6}$$

In Figure 2.4 modal capacity curve with coordinates (d_1, a_1) and spectral behavior curve with coordinates of spectral displacement (S_d) and spectral acceleration (S_a) are sketched at common axis.

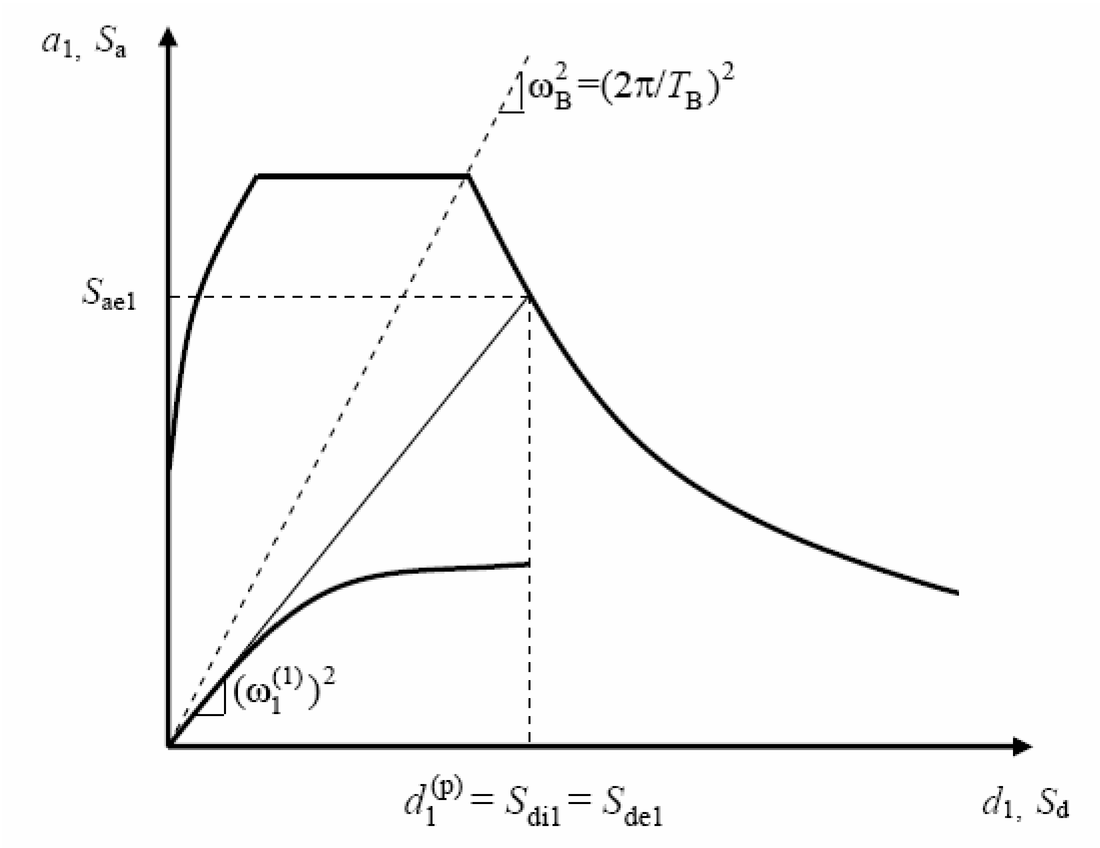

Figure 2.4 : Determination of Modal Displacement Demand $\left(T_1^{(1)} \geq T_B\right)$

When fundamental period $T_1^{(1)}$ is shorter than the characteristic period T_B, then spectral displacement ratio C_{R1} is calculated by iteration. The steps of the iteration procedure are explained below:

Modal capacity curve is converted to a bi-linear diagram as shown in Figure 2.5. The slope of the first line is taken equal to the square of the frequency ($\omega_1^{(1)}$) of the fundamental mode.

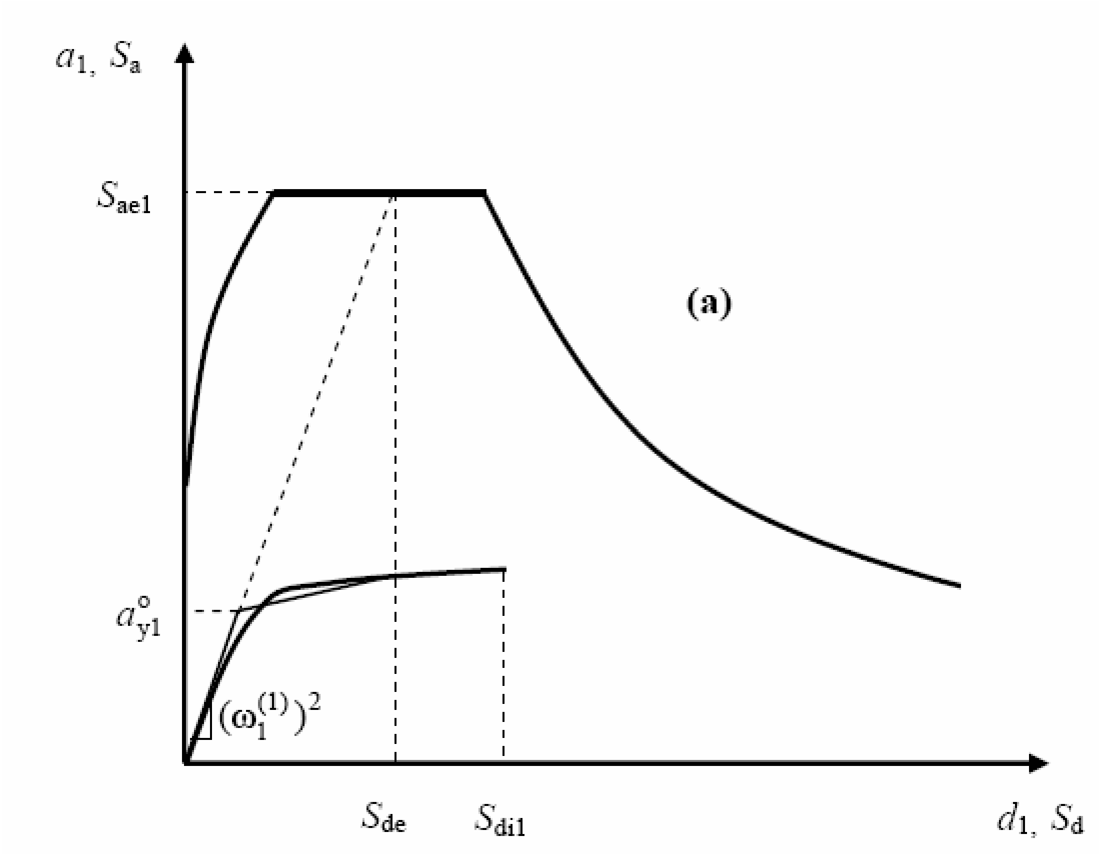

Figure 2.5 : Determination of Modal Displacement Demand $\left(T_1^{(1)} < T_B\right)$

At first step of iteration, with assumption of $C_{R1} = 1$, the coordinates of equivalent yielding are calculated by *Equal Areas Rule*. The calculation of C_{R1} is based on $a_{y1}{}^{0}$ as shown in Figure 2.5.

$$C_{R1} = \frac{1 + \left(R_{y1} - 1\right) T_B / T_1^{(1)}}{R_{y1}} \geq 1 \tag{2.7}$$

R_{y1} is the capacity reduction coefficient at the fundamental mode

$$R_{y1} = \frac{S_{ae1}}{a_{y1}} \tag{2.8}$$

By using C_{R1} from Equation (2.7) and in principle of S_{di1}, the coordinates of equivalent yielding point are determined with the *Equal Areas Rule* as shown in Figure 2.6. Then, a_{y1}, R_{y1} and C_{R1} iterations are repeated until the final values are close enough to each other.

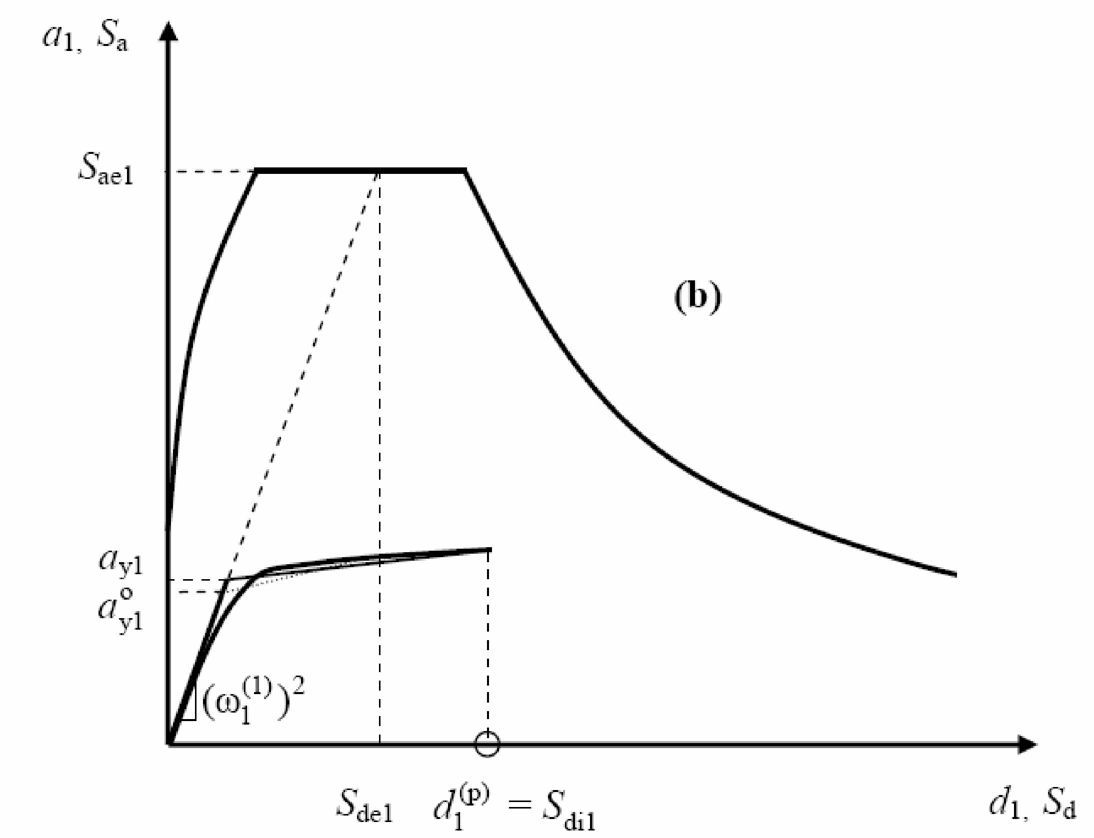

Figure 2.6 : Determination of Modal Displacement Demand $\left(T_1^{(1)} < T_B\right)$

Replacing the modal displacement demand at p-th step yields the target displacement $u^{(p)}{}_{xN1}$.

$$u^{(p)}{}_{xN1} = \Phi_{xN1} \cdot \Gamma_{x1} \cdot d_1{}^{(p)} \quad \textbf{(2.9)}$$

Other demands (displacements, deformations, internal forces) at target displacements can be obtained either from related analysis results or from a new analysis with pushing the system to target displacement value.

2.6.3.4 Determination of Strains at Plastic Cross-sections

Plastic curvature demand is obtained by plastic rotation demands from the outputs of push-over analysis.

Plastic curvature demand is the amount of *plastic rotation demand* at unit plastic hinge length. Plastic rotation values are obtained from the push-over analysis as an output.

$$\phi_p = \frac{\theta_p}{L_p} \qquad \textbf{(2.10)}$$

Total curvature demand ϕ_t is sum of the plastic curvature demand ϕ_p and yielding curvature ϕ_y which can be achieved from a cross-section analysis related to concrete and reinforcement material properties. Then;

$$\phi_t = \phi_y + \phi_p \qquad \textbf{(2.11)}$$

Concrete compression strain and reinforcement tension strain values are calculated with using the total curvature obtained by a cross-section analysis.

Earthquake demands in terms of strains are to be compared with stain capacities in order to determine the damage level of the corresponding member end.

2.6.3.5 Strain Capacities of Plastic Cross-sections for RC Elements

Damage level stain limits (capacities) for plastic deformed elements are introduced on following paragraphs.

Minimum Damage Limit (ML): upper limits of unconfined concrete zone compression strain and reinforcement tension strain values are:

$$(\varepsilon_{cu})_{ML} = 0.0035$$

$$(\varepsilon_s)_{ML} = 0.010$$

Safety Limit (SL): upper limits of confined concrete zone compression strain and reinforcement tension strain values are:

$$\left(\varepsilon_{cu}\right)_{SL} = 0.0035 + 0.01\left(\rho_s / \rho_{sm}\right) \leq 0.0135$$

$$\left(\varepsilon_s\right)_{SL} = 0.040$$

where ρ_s and $\rho_{s\,m}$ are the existing and required transverse reinforcement ratios, respectively.

Collapse Limit (CL): upper limits of confined concrete zone compression strain and reinforcement tension strain values are:

$$\left(\varepsilon_{cu}\right)_{CL} = 0.004 + 0.014\left(\rho_s / \rho_{sm}\right) \leq 0.018$$

$$\left(\varepsilon_s\right)_{CL} = 0.060$$

3. SEISMIC ASSESSMENT OF EXISTING BUILDINGS ACCORDING TO EUROCODE 8

Eurocode 8, Design of Structures for Earthquake Resistance, covers, as its title suggests, earthquake-resistant design and construction of buildings in seismic regions. Main objective is to ensure life safety and building protection in an event of seismic actions. This is important for civil protection's continuity [27].

3.1 As-built Information for Structural Assessment and Knowledge Levels

The comprehension about the as-built situation of the structure, including its geometry, detailing, and material properties and existing of any degradation, is classified as a particular knowledge level. Source of the acquired information also affect the classification. Required information can be collected from available documentation specific to the building, field investigations, and test measurements from laboratories.

From knowledge levels, component capacities are modified using confidence factors. The lower the knowledge level, the more conservative the applied assessment results should be.

The non-linear analysis methods are not applicable when the knowledge level is insufficient, because they require detailed information about the properties of the structure.

For each knowledge level, EC8 recommends respective confidence factors for dividing mean values of material properties.

Table 3.1 : Confidence Factors according to Eurocode 8

Knowledge Level	Confidence Factor
Limited (KL1)	1.35
Normal (KL2)	1.20
Full (KL3)	1.00

3.2 Performance Requirements and Compliance Criteria

Building seismic performance levels are chosen discrete levels of building damage under earthquake excitation. In Eurocode 8 denotes seismic performance levels as *"Limit States"*. These limit states are characterized on following paragraphs, namely Damage Limitation (DL), Significant Damage (SD), and Near Collapse (NC).

3.2.1 Limit State of Damage Limitation

The structure is only slightly damaged with insignificant plastic deformations. Repair of structural components is not required, because their resistance capacity and stiffness are not compromised. Cracks may present on non-structural elements, but they can be economically repaired. The residual deformations are unnecessary.

3.2.2 Limit State of Significant Damage

The structure is significantly damaged and it has undergone resistance reduction. The non-structural elements are damaged, yet the partition walls are not failed. The structure consists of permanent significant drifts and generally it is not economic to repair.

3.2.3 Limit State of Near Collapse

The structure is heavily damaged; on the other hand, vertical elements are still able to carry gravity loads. Most non-structural elements are failed, and remained ones will not survive under next seismic actions, even for slight horizontal loads.

The adapted limit states are achieved by choosing, for each performance levels, a return period for the seismic action. European countries check the return periods ascribed to the various limit states and define it in its National Annex. Recommended return periods to corresponding limit states are defined at below:

a) *Limit State of Near Collapse*: Return period of 2457 years, corresponding to a probability of exceedence of 2% in 50 years.

b) *Limit State of Significant Damage*: Return period of 475 years, corresponding to a probability of exceedence of 10% in 50 years.

c) *Limit State of Damage Limitation*: Return period of 225 years, corresponding to a probability of exceedence of 20% in 50 years.

3.3 Assessment and Methods of Analysis

There are four types of displacement-based analysis procedures described EC8. Depending on the structural characteristics of the building, *lateral force method of analysis* or *modal response spectrum analysis* may be used as linear-elastic methods. As an alternative to a linear method, a non-linear method may also be used, such as *non-linear static (pushover) analysis* or *non-linear time history (dynamic) analysis*.

Static procedures may be used whenever participation of higher modes is negligible. The load patterns, used for static analyses, are not able to represent deformed shape of the structure when higher modes are put into effect. The participation of higher modes depends generally on regularity of mass and stiffness and on the distribution of natural frequencies of the building with respect to seismic fundamental frequencies.

Linear procedures (lateral force method of analysis and modal response spectrum) are applicable when the structure remains almost elastic or when expected plastic deformations are uniformly distributed all over the structure.

For the horizontal components of the seismic action, the elastic response spectrum $S_e(T)$ is defined by the following expressions (Figure 3.1):

$$0 \leq T \leq T_B : S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2.5 - 1) \right] \qquad \textbf{(3.1)}$$

$$T_B \leq T \leq T_C : S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \quad \textbf{(3.2)}$$

$$\textbf{(3.3)} \quad T_C \leq T \leq T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \left[\frac{T_C}{T}\right]$$

$$T_D \leq T \leq 4s : a_g \cdot S \cdot \eta \cdot 2.5 \cdot \left[\frac{T_C \cdot T_D}{T^2}\right] \quad \textbf{(3.4)}$$

Where a_g is the design ground acceleration on type A soil condition profile, while, T_B, T_C and T_D represent characteristic period values describing the shape of the spectrum curve. Soil factor S, also depends on ground type, affects the overall curve ordinate. Damping correction factor η may be taken as 1.00 for 5% viscous damping.

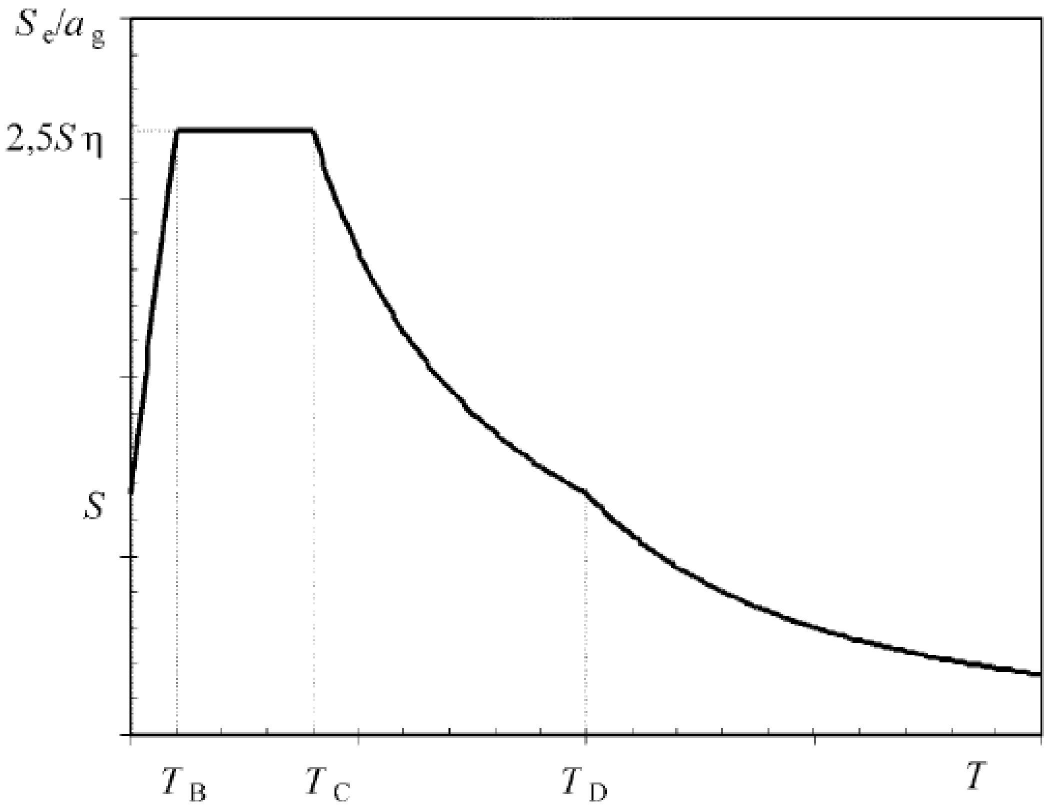

Figure 3.1: Shape of elastic response spectrum

The values of the periods T_B, T_C and T_D and of the soil factor S depend upon the soil type where the building is located. Characteristic periods and soil factor values are categorized for two different elastic response spectra curves. Type 1 elastic response is recommended for the purpose of probabilistic hazard assessment have a surface-wave magnitude greater than 5.5, therefore, it includes a wider peak acceleration zone on its spectrum curve. On the other hand, Type 2 elastic response represents less critical seismic zones (Table 3.2 and Table 3.3).

Table 3.2 : Parameters for Type 1 Elastic Response Spectra

Ground Type	S	T_B (sec)	T_C (sec)	T_D (sec)
A	1.00	0.15	0.40	2.00
B	1.20	0.15	0.50	2.00
C	1.15	0.20	0.60	2.00
D	1.35	0.20	0.80	2.00
E	1.40	0.15	0.50	2.00

Table 3.3 : Parameters for Type 2 Elastic Response Spectra

Ground Type	S	T_B (sec)	T_C (sec)	T_D (sec)
A	1.00	0.05	0.25	1.20
B	1.35	0.05	0.25	1.20
C	1.50	0.10	0.25	1.20
D	1.80	0.10	0.30	1.20
E	1.60	0.05	0.25	1.20

On the following paragraphs, procedures of lateral force method of analysis and non-linear static (push-over) analysis will be introduced in detail under scope of this thesis study.

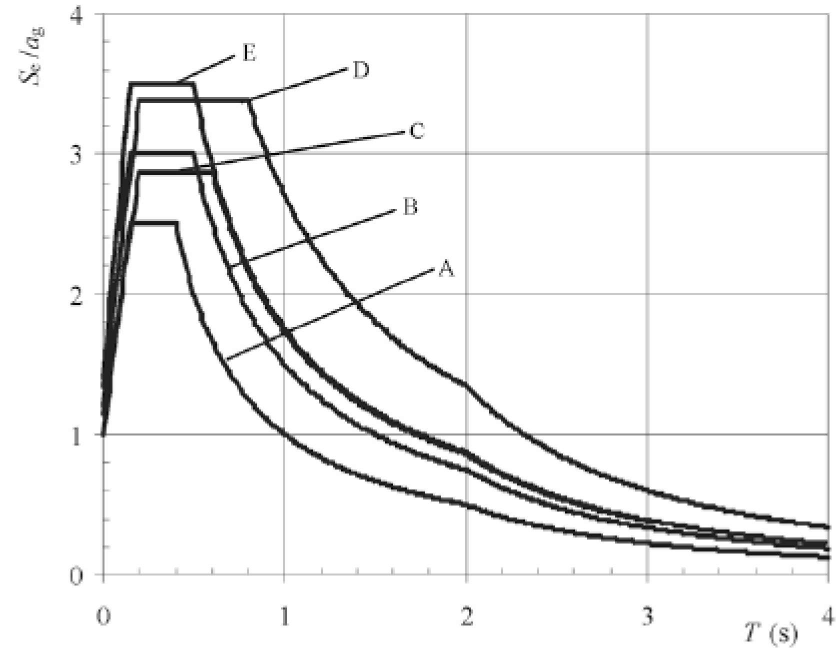

Figure 3.2 : Recommended Type 1 Elastic Response Spectra

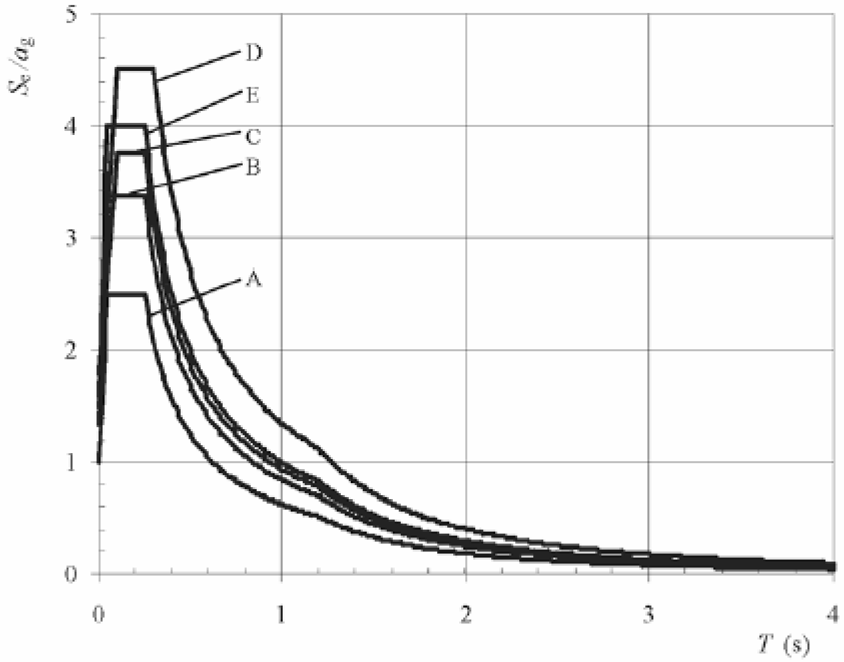

Figure 3.3 : Recommended Type 2 Elastic Response Spectra

3.3.1 Lateral Force Method of Analysis

Usage of linear static procedure instead of dynamic one is allowed when the structure is verified to be regular in elevation and when the fundamental period T is less than 2 seconds and also less than the four times the characteristic period T_C.

$$T_1 \leq \min(4 \cdot T_C; 2\,\text{sec}) \tag{3.5}$$

Another restriction for applying the lateral force method of analysis is that the ratio of the maximum value to minimum value of demand-capacity-ratios (r) for all ductile elements that go beyond elastic limit must be lower than 2.50.

$$r_{\max}/r_{\min} \leq 2.50 \tag{3.6}$$

This rule makes the lateral force method of analysis can only be applied for buildings with fairly uniform distribution of overstrengths of members.

Despite of using design spectrum $S_d(T)$ as for new building design, elastic response spectrum $S_e(T)$ must be used for the existing building assessments.

The seismic base shear force F_b, for the existing building in each horizontal direction for which the existing building seismic performance is analyzed, shall be determined using the following expression:

$$F_b = S_e(T_1) \cdot m \cdot \lambda \tag{3.7}$$

Total mass of the building m is obtained by considering above the foundation or above the top of a rigid basement. Correction factor λ is equal to 0.85 whenever $T_1 \leq 2 \cdot T_C$ and the building has more than two storeys; otherwise it must be taken as 1.00.

The distribution of the horizontal forces F_i along the height of the building depends on the mass and the mode shape contribution of each storey to overall building.

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum (s_j \cdot m_j)} \tag{3.8}$$

where s_i represents the displacement of mass m_i in the fundamental mode shape of a building.

3.3.2 Non-linear Static Analysis

In non-linear static (pushover) analysis procedure, the model directly incorporates the non-linear force and deformation relations of the structural components (material non-linearity) and accounts for P-delta influences (geometric non-linearity). At the beginning, the structure is subjected to gravity loads, and then horizontal forces are statically applied. The process is carried-out under monotonically increasing horizontal loads to investigate the relation of the displacements of the control node. The control node is generally located at the center of mass of the roof. The diagram which base shear is versus control displacement is called *capacity curve* or *push-over curve*.

Chord rotation demands at *target displacement* are checked due to the safety verifications to determine the overall structure performance level. The target displacement shall be defined as the seismic demand derived from elastic response spectrum in terms of the displacement of an equivalent single-degree-of-freedom (SDOF) system. The transformation procedure from multi-degree-of-freedom system (MDOF) to an equivalent single-degree-of-freedom system is explained below.

The mass of an equivalent SDOF system m^* is determined as:

$$m^* = \sum m_i \cdot \Delta_i \tag{3.9}$$

where normalized displacements Δ_i are obtained is such a way that the control node displacement at roof is equal to a value of 1.

The transformation factor Γ, which is required in order to convert *push-over curve* to *modal capacity curve*, can be determined as shown below:

$$\Gamma = \frac{m^*}{\sum m_i \cdot \Delta_i^{\,2}} \tag{3.10}$$

The force F^* and displacement d^* of the equivalent SDOF system are computed as:

$$F^* = \frac{F_b}{\Gamma} \tag{3.11}$$

$$d^* = \frac{d_n}{\Gamma} \tag{3.12}$$

where F_b and d_n are, respectively, the base shear force and the control node displacement of the multi-degree-of-freedom (MDOF) system, respectively.

Actual modal capacity curve can be idealized as elasto-perfectly plastic capacity relation where both modal displacement – modal force relations consist of equal deformation energy up to the formation of the plastic formation.

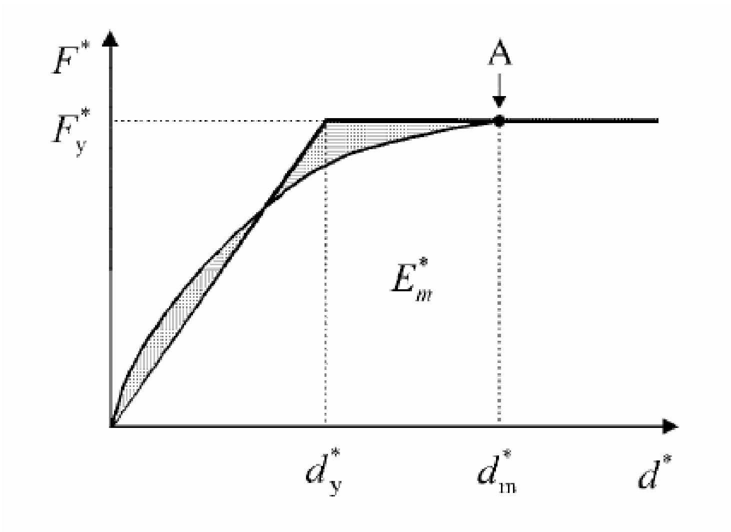

Figure 3.4 : Idealized Elasto-Perfectly Plastic Force - Displacement Relationship

From this assumption, the yield displacement of the idealized SDOF system d_y^* can easily be determined from equation at below.

$$d_y^* = 2 \cdot \left(d^*_m - \frac{E^*_m}{F^*_y} \right) \tag{3.13}$$

where d^*_m and E^*_m are modal displacement and modal deformation energy respectively as shown in Figure 3.4.

The period T^* of the idealized equivalent SDOF system is determined by:

$$T^* = 2 \cdot \pi \cdot \sqrt{\frac{m^* \cdot d^*_y}{F^*_y}} \tag{3.14}$$

The target displacement of the structure with period T^* and unlimited elastic behavior can be calculated from:

$$d^*_{et} = S_e(T^*) \cdot \left[\frac{T^*}{2 \cdot \pi} \right]^2 \tag{3.15}$$

where $S_e(T^*)$ represents the elastic acceleration response spectrum at the period T^*.

In determination of the SDOF system displacement d_t^* for structures in the short-period range and for the structures in the medium and long-period ranges different formulas must be used as expressed below. The corner period T_C distinguishes the short- and medium-period ranges from each other.

In short period range, $d_t^* = d_{et}^*$ when the response is elastic. The condition of response being elastic can be verified from expression given below:

$$F_y^* / m^* \geq S_e(T^*) \tag{3.16}$$

Otherwise, the response will be non-linear and

$$d_t^* = \frac{d_{et}^*}{q_u} \cdot \left(1 + (q_u - 1) \cdot \frac{T_C}{T^*}\right) \geq d_{et}^* \tag{3.17}$$

where q_u is defined as the ratio between the acceleration in the structure with unlimited elastic behavior $S_e(T^*)$ and in the structure with limited strength F_y^* / m^*.

$$q_u = \frac{S_e(T^*) \cdot m^*}{F_y^*} \tag{3.18}$$

In medium- and long- period range, SDOF displacement is taken as $d_t^* = d_{et}^*$.

The actual target displacement for the MDOF system can be retrieved by multiplying single-degree-of-freedom system displacement by the transformation factor which is computed earlier in Eqn.(3.10). The target displacement corresponds to the control node.

$$d_t = \Gamma \cdot d_t^* \tag{3.19}$$

3.4 Safety Verifications

Comparisons of demand obtained from the analysis between the capacity, which is dependent on the geometry and material property of the structural members, must be done for safety verifications. Verifications are necessary to determine whether the investigated structural component is compatible with requested limit state or not.

Structural members are classified as ductile and brittle elements in advance. For linear methods of analysis, ductile elements are verified on the basis of deformations, while brittle elements must be verified on the basis of forces. Brittle elements or mechanisms must be verified with demands calculated by means of equilibrium conditions, on the basis of the action effects delivered to the brittle component by ductile components.

If analysis is linear, shear force demand is determined from equilibrium conditions under end moments consistent with the formations of plastic hinges there or around joint.

For columns:

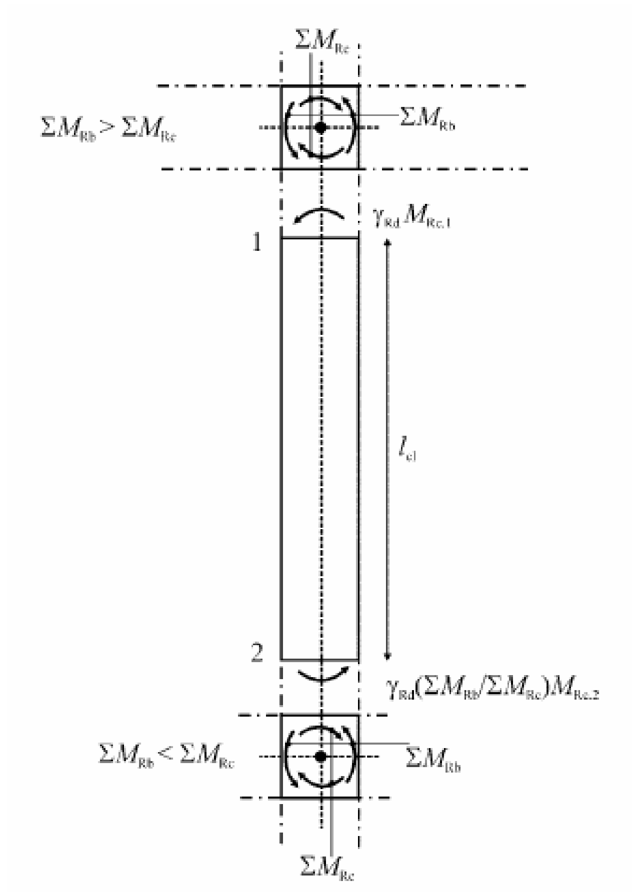

Figure 3.5 : Equilibrium of column end moments

$$M_{i,d} = M_{Rc,i} \cdot \min\left(1, \frac{\sum M_{Rb}}{\sum M_{Rc}}\right) \tag{3.20}$$

For beams:

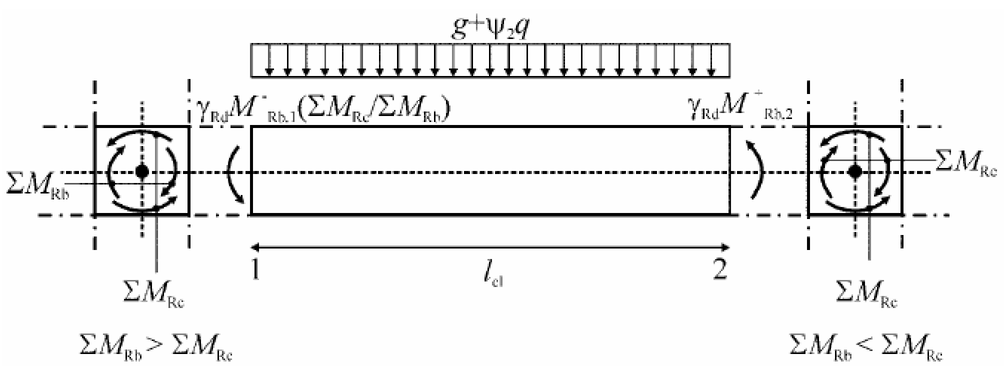

Figure 3.6 : Equilibrium of beam end moments

$$M_{i,d} = M_{Rb,i} \cdot \min\left(1, \frac{\sum M_{Rc}}{\sum M_{Rb}}\right)$$

(3.21)

where M_{Rc} and M_{Rb} are the design value of the column end moment and beam end moment, respectively.

Each action in a ductile component delivered to the brittle component under consideration shall be taken equal to demand value from analysis whenever demand-capacity ratio (DCR) is smaller than 1. Otherwise, component capacities multiplied with knowledge confidence factor must be used.

Chord rotation capacity limits of ductile components for linear analysis and both ductile and brittle components for non-linear static analysis for each structure limit states are described in detail in following paragraphs.

For limit state of near collapse, the value of the total chord rotation capacity (elastic plus inelastic part) at ultimate, θ_u, of concrete members under cycling loading can be calculated from the following expression:

$$\theta_u = \frac{1}{\gamma_{el}} \cdot 0.016 \cdot (0.3)^{\upsilon} \cdot \left[\frac{\max(0.01;\rho')}{\max(0.01;\rho)} \cdot f_c\right]^{0.225} \left(\frac{L_v}{h}\right)^{0.35} \cdot 25^{\left(\alpha \cdot \rho_{sx} \cdot \frac{f_{yw}}{f_c}\right)} \cdot \left(1.25^{100 \cdot \rho_d}\right) \quad \textbf{(3.22)}$$

For the evaluation of the ultimate chord rotation capacity an alternative expression may also be used:

$$\theta_u = \frac{1}{\gamma_{el}} \cdot \left(\theta_y + (\varphi_u - \varphi_y) \cdot L_{pl} \cdot \left(1 - \frac{0.5 \cdot L_{pl}}{L_V}\right)\right)$$

(3.23)

where

h : The depth of the cross-section

L_v : The ratio moment/shear at the end section

υ : Axial demand/capacity ratio of the section

ρ, ρ' : Mechanical reinforcement ratio of tension and compression, respectively, longitudinal reinforcement

ρ_{sx} : The ratio of transverse steel parallel to the direction x of loading

The cord rotation corresponding to significant damage θ_{SD} may be assumed to be ¾ of the ultimate chord rotation capacity.

The capacity for *limit state of damage limitation* used in the verifications is the yielding bending moment under the design value of the axial load.

In case of verifications is carried out in terms of deformations, the chord rotation at yielding θ_y can be evaluated as:

$$\theta_y = \phi_y \frac{L_V + a_V z}{3} + 0.00135\left(1 + 1.5\frac{h}{L_V}\right) + \frac{\varepsilon_y}{d - d'}\frac{d_b f_y}{6\sqrt{f_c}} \qquad \textbf{(3.24)}$$

As for the brittle elements, the verification for damage limitation and significant damage limit states is not required, unless these two limit states are the only ones to be checked for the structures.

For limit state of near collapse, the following expression may be used for the shear strength capacity. The units must be set in mega Newtons (MN) and meters.

$$V_R = \frac{1}{\gamma_{el}}\left[\frac{h - x}{2L_V}\min(N; 0.55 A_c f_c) + \left(1 - 0.55\min\left(5; \mu_\Delta{}^{pl}\right)\right)\right]$$

$$\cdot\left[0.16\max(0.5; 100\rho_{tot})\left(1 - 0.16\min\left(5; \frac{L_V}{h}\right)\right)\sqrt{f_c}A_c + V_w\right] \qquad \textbf{(3.25)}$$

4. CASE STUDY: EVALUATION OF SEISMIC PERFORMANCE LEVEL OF AN EXISTING REINFORCED CONCRETE BUILDING

In this chapter, the procedures of seismic performance evaluations for existing reinforced concrete buildings according to Eurocode 8 and TEC'07 are applied on a real 3D case study building with six storeys.

The case study building is designed as a framed RC structural system. The investigated building does not have any irregularities in elevation or in plan. It had been experienced the Adana-Ceyhan Earthquake occurred in 1998, and it was reported by the authorities that retrofitting process is suggested for the residence building because of the experienced medium damage level [29].

This chapter includes the description of the case-study building and the procedure of seismic performance evaluation with linear static and non-linear static (push-over) analyses according to Turkish Earthquake Code and Eurocode 8. For each methods of analysis, damage levels of structural components and building performance levels are determined. Softwares that were used for aid of the evaluations will also be introduced in this section. The evaluations are summarized as a flowchart shown in Figure 4.1.

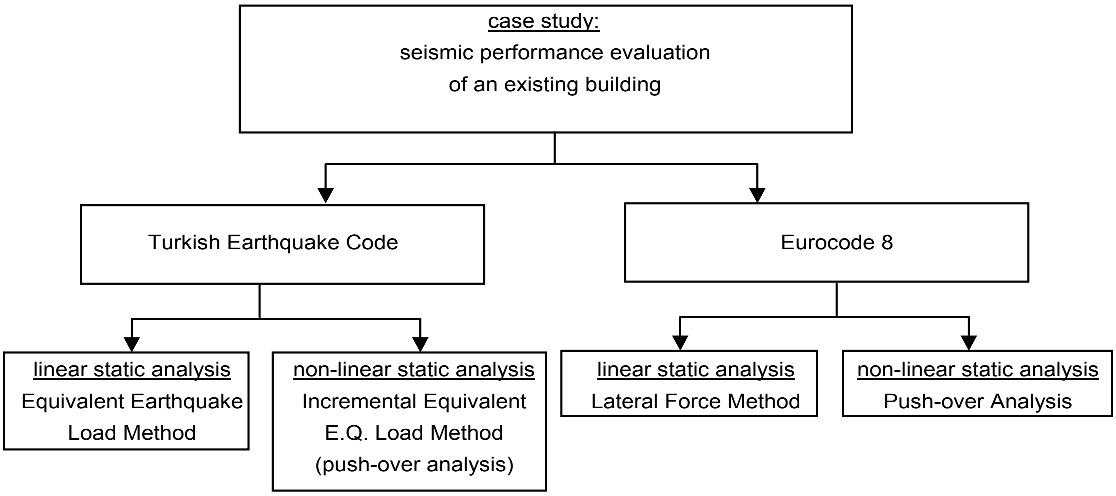

Figure 4.1 : Case Study Flowchart

4.1 Structural Information of the Building

The considered building was exposed to seismic action of Ms = 6.3 with a maximum acceleration of 0.28 g occurred in Adana Ceyhan Earthquake of 1998 and reported as moderately damaged under that seismic action. The case study building has six storeys with a total of 14.65 m height and it is composed of orthogonal frames, symmetrical in y direction and does not have any structural irregularities. The planar dimensions are 16.4 x 7.8 m = 127.9 m^2 with five spans in x and two spans in y directions (Figure 4.2). It was initially designed and constructed according to the 1975 Turkish Seismic Code.

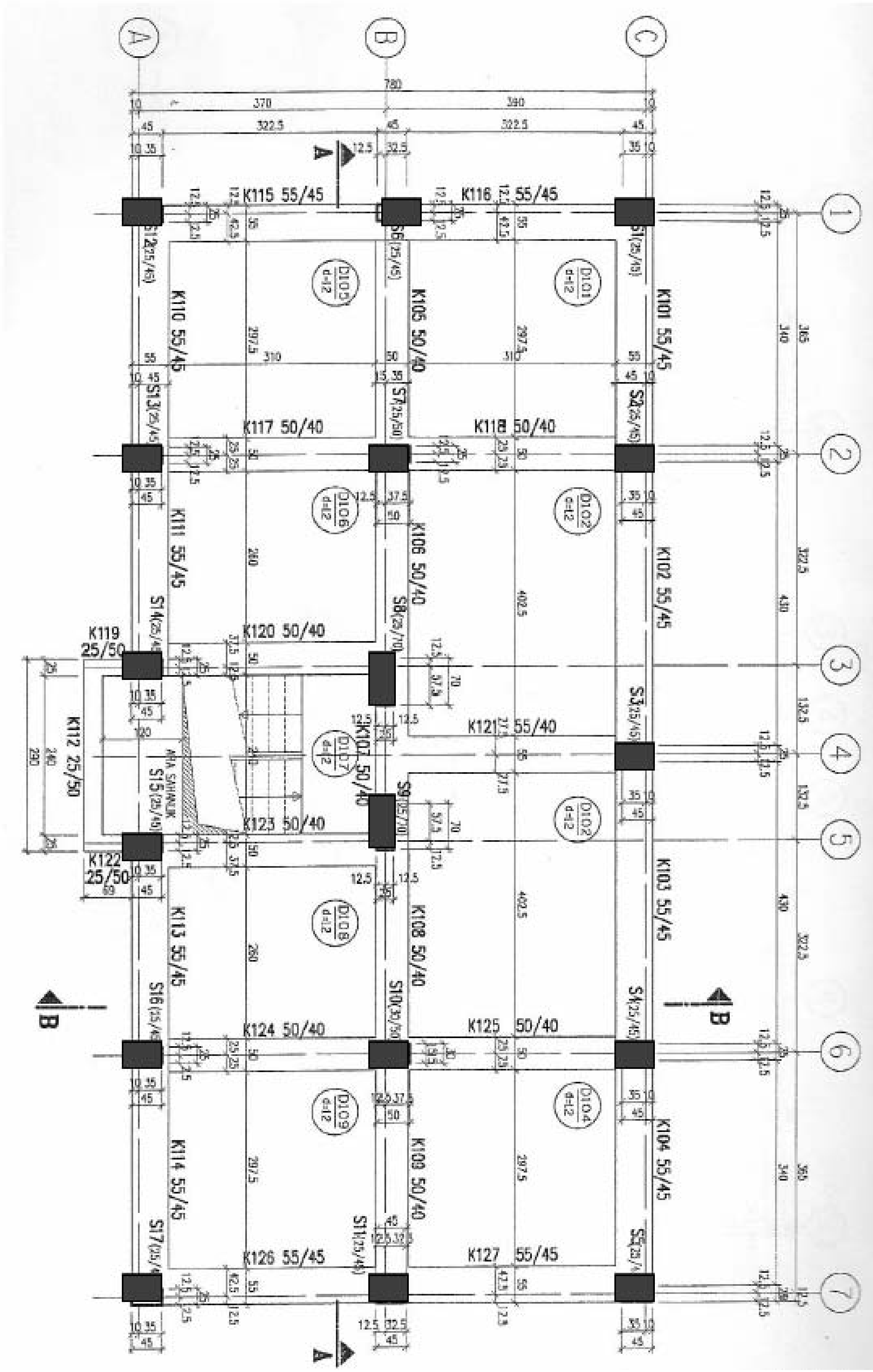

Figure 4.2 : First Floor Plan

Storey heights are 2.15 m for the first storey and 2.50 m for the other storeys. Slabs are having a thickness of 12 cm and they are modeled as rigid diaphragm at each storey level while creating the mathematical model. The columns have section with and depth as 25/45 cm, 25/50 cm, 25/70 cm at first storey for column groups A, B and C, respectively. For upper storeys, columns keep their cross-section dimensions as the same as at the first storey except group-A which is 25/40 cm. Outer dimensions and mathematical model created by using SAP 2000 [4] is shown with Figure 4.3. First floor structure component label map is shown in Figure 4.4.

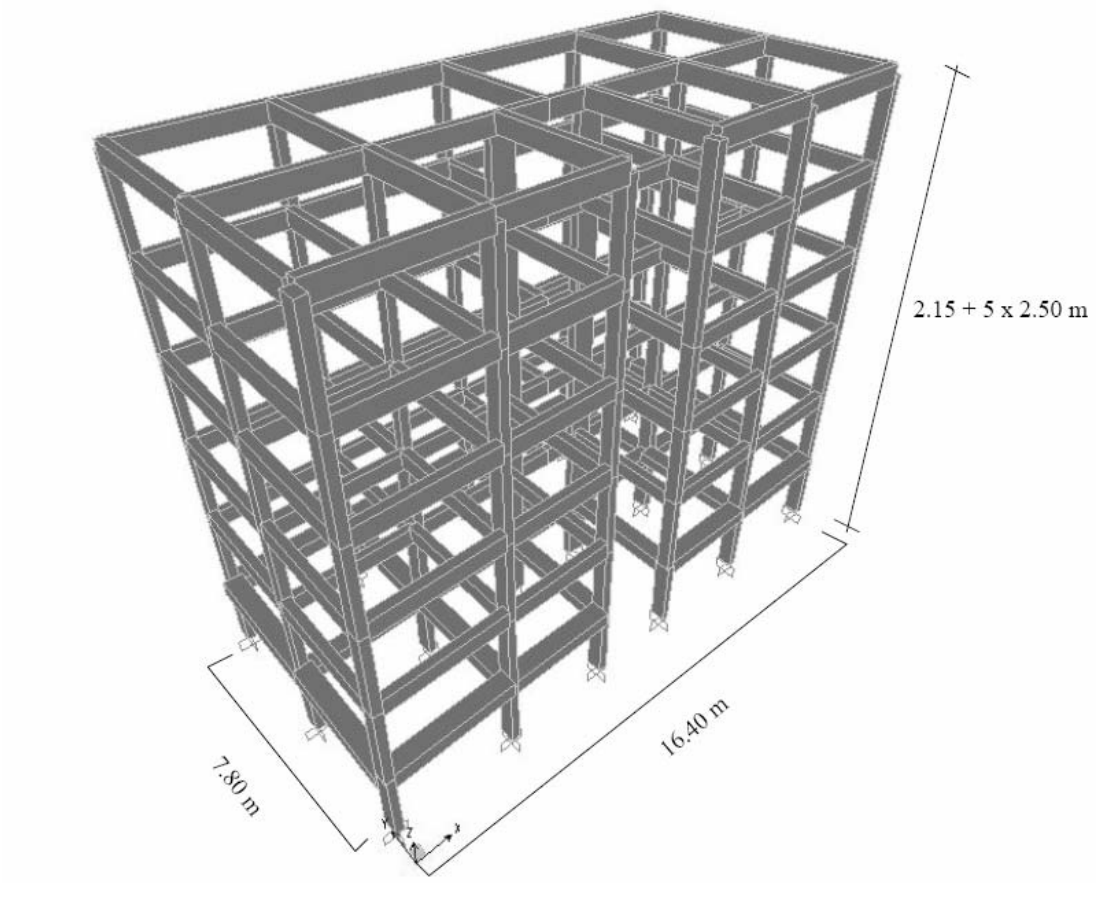

Figure 4.3 : 3D Mathematical Model

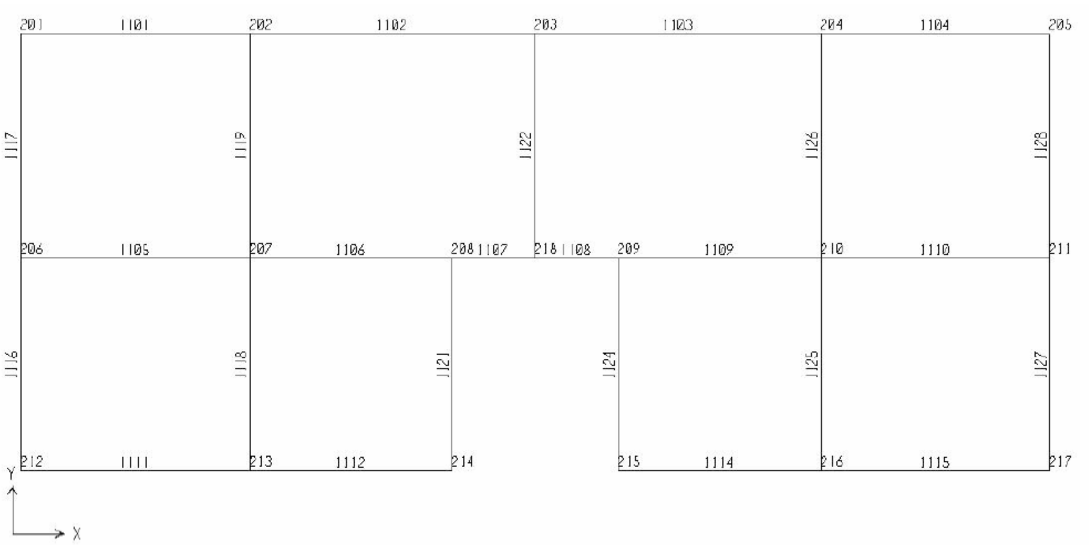

Figure 4.4 : First Floor Structural Component Label Map

4.1.1 Frame Elements Cross-sectional Details

Member cross-section dimensions and corresponding longitudinal reinforcements belong to the first storey, which is also called as critical storey, are given for columns and beams respectively.

Table 4.1 : Dimensions and Longitudinal Reinforcements of First Storey Columns

Column	b [cm]	d [cm]	Reinforcement	
			top + bottom	edges
101	25	45	8Φ20	2Φ14
102	25	45	8Φ20	4Φ16
103	25	45	8Φ20	4Φ16
104	25	45	8Φ20	4Φ16
105	25	45	8Φ20	2Φ14
106	25	45	8Φ20	4Φ14
107	25	50	8Φ20	6Φ20
108	70	25	8Φ20	8Φ14
109	70	25	8Φ20	8Φ14
110	30	50	8Φ20	6Φ20
111	25	45	8Φ20	4Φ14
112	25	45	8Φ20	2Φ14
113	25	45	8Φ20	4Φ16
114	25	45	8Φ20	4Φ14
115	25	45	8Φ20	4Φ14
116	25	45	8Φ20	4Φ16

Table 4.2 : Dimensions and Longitudinal Reinforcements of First Storey Beams

Beam	b_w [cm]	h [cm]	Reinforcement			
			Left		Right	
			Top	Bottom	Top	Bottom
1101	55	45	6Φ14	5Φ14	6Φ14	5Φ14
1102	55	45	6Φ14	5Φ14	6Φ14	5Φ14
1103	55	45	6Φ14	5Φ14	6Φ14	5Φ14
1104	55	45	6Φ14	5Φ14	6Φ14	5Φ14
1105	50	40	5Φ14	4Φ14	5Φ14	4Φ14
1106	50	40	5Φ14	4Φ14	5Φ14	4Φ14
1107	50	40	5Φ14	4Φ14	5Φ14	4Φ14
1108	50	40	5Φ14	4Φ14	5Φ14	4Φ14
1109	50	40	5Φ14	4Φ14	5Φ14	4Φ14
1110	50	40	5Φ14	4Φ14	5Φ14	4Φ14
1111	55	45	6Φ14	5Φ14	6Φ14	5Φ14
1112	55	45	6Φ14	5Φ14	6Φ14	5Φ14
1114	55	45	6Φ14	5Φ14	6Φ14	5Φ14
1115	55	45	6Φ14	5Φ14	6Φ14	5Φ14
1116	55	45	6Φ14	5Φ14	6Φ14	5Φ14
1117	55	45	6Φ14	5Φ14	6Φ14	5Φ14
1118	50	40	5Φ14	4Φ14	5Φ14	4Φ14
1119	50	40	5Φ14	4Φ14	5Φ14	4Φ14
1121	50	40	5Φ14	4Φ14	5Φ14	4Φ14
1122	55	40	5Φ14	4Φ14	5Φ14	4Φ14
1124	50	40	5Φ14	4Φ14	5Φ14	4Φ14
1125	50	40	5Φ14	4Φ14	5Φ14	4Φ14
1126	50	40	5Φ14	4Φ14	5Φ14	4Φ14
1127	55	45	6Φ14	5Φ14	6Φ14	5Φ14
1128	55	45	6Φ14	5Φ14	6Φ14	5Φ14

4.1.2 Material Properties

Information on the mechanical properties of construction materials from extended in-situ testing are summarized at below [29].

Characteristic Compression Capacity of Concrete : 10 MPa

Elasticity Modulus of Concrete : 24250 MPa

Characteristic Yielding Capacity of Reinforcement : 220 MPa

Which are lower values than the ones given in the original project.

4.1.3 Seismic Parameters

General seismic properties, which are dependent of the location and the functional purpose of the building, are summarized below, in order to be used commonly according to TEC'07 and Eurocode 8 assessment procedures.

Description of the statigraphic profile of the ground mentions that it consists of deposits of very stiff clay, at least several tens of meters in thickness, with a gradual increase of mechanical properties with depth.

Seismic Zone	: 2nd Zone
Functionality Purpose	: Residence
Live Load Participation	: 0.30

4.1.4 Gravity Loads

At below, gravity loads from slabs that are used both in linear and non-linear static analysis procedures are given.

Normal Storeys

Dead Load	: 5.25 kN/m^2
Live Load	: 2.00 kN/m^2

Roof

Dead Load	: 6.15 kN/m^2
Live Load	: 2.00 kN/m^2

4.1.5 Assumptions Made in Modeling

Assumptions that are made during the creation of the mathematical model are described at below.

a) Joints are defined as rigid and the behavior of restraints at foundation level is assumed as fixed.

b) Gravity loads from infill walls (4.00 kN/m) are acted to columns as single loads [30].

c) Diaphragm constraints are defined at each storey levels that cause all of its constrained joints to move together as a planar diaphragm that is rigid against membrane deformations. All constrained joints are connected to each other by links that are rigid for out-of-plane bending, but do not affect the in-plane deformations.

d) Plastic hinges that are valid for non-linear methods of analysis are replaced at column and beam end points.

4.2 Assessment According to TEC 2007

After 1999 Adapazari-Kocaeli and Duzce Earthquakes analyses regarding to seismic risk assessments for existing buildings in Turkey have been significantly increased. Up to date, the existing seismic prestandard did not include assessment procedures for existing buildings. The applications were performed under the basis of Turkish Earthquake Code 1998 which was actually aimed only for new building design procedures. Since 2003, researches and studies to include a new chapter concerning the seismic performance evaluation of existing buildings have been completed and have been put into effect under Turkish Earthquake Code 2007.

In this section, calculation steps for the case building according to *Equivalent Seismic Load* (linear static analysis) and *Incremental Equivalent Seismic Load* (push-over analysis) methods of analysis will be explained briefly on the basis of TEC'07.

Before investigating the seismic performance of the case building under different analysis types, on the following paragraphs, common seismic properties, which are valid both for linear static and non-linear static analysis procedures, are determined. Acceptance criteria for using the methods of analysis will also be checked.

Storey weights and building total weight considering the participation of live loads are calculated with the formulas given below:

$$w_i = g_i + n \cdot q_i \qquad \textbf{(4.1)}$$

$$W = \sum_{i=1}^{n} w_i \qquad \textbf{(4.2)}$$

where w_i represents the weight of an individual storey i.

Storey weights regarding to the case study building are given in Table 4.3.

Table 4.3 : Storey Weights

storey	weight [kN]
6	925.50
5	1048.51
4	1048.51
3	1050.72
2	1052.93
1	1258.32
total	**6384.50**

Due to the comprehensiveness of obtained as-built information, knowledge level of the case study building is confirmed as "medium" and the corresponding confidence factor is 0.90 as given in Table 2.1.

The initial elastic stiffness values of beams are multiplied with 0.40 in order to obtain the effective stiffness. The calculation of the effective elastic stiffness for column 101 under flexure with axial force is given below. The results for the rest of vertical elements at critical storey are given in Table 4.4.

$$(EI)_e = 0.40\ (EI)_0 \text{ if } N_D / (A_c\ f_{cm}) \leq 0.10$$
$$(EI)_e = 0.80\ (EI)_0 \text{ if } N_D / (A_c\ f_{cm}) \geq 0.40$$

For column 101:

$N_D = 226.78$ kN

$A_c = 0.1125$ m^2

$f_{cm} = 9000$ kN/ m^2

$N_D / (A_c\ f_{cm}) = 0.223$

$(EI)_e = 0.45\ (EI)_0$ for column 101

Table 4.4 : Effective Elastic Stiffness for Columns under Flexure

COLUMN	EI_E / EI_0
101	0.56
102	0.43
103	0.41
104	0.43
105	0.56
106	0.45
107	0.80
108	0.52
109	0.52
110	0.80
111	0.45
112	0.54
113	0.44
114	0.56
115	0.56
116	0.44
117	0.54

Fundamental periods for along x axis and y axis and rotation about z axis are obtained by SAP2000 modal analysis after effective stiffness values are applied to the structural elements.

Tx	0.861 sec
Ty	0.814 sec
Tϴ	0.701 sec

Initially determined fundamental periods, which are obtained from SAP2000 [4] modal analysis, must be verified with the following relation.

$$T_1 \leq 2\pi \cdot \left(\frac{\sum_{i=1}^{N} m_i \cdot d^2{}_{fi}}{\sum_{i=1}^{N} F_{fi} \cdot d_{fi}} \right) \qquad \textbf{(4.3)}$$

Modal analysis of the software gives 0.861 sec for the fundamental period along x axis. This period time is verified since it is lower than the allowed maximum value calculated as in Table 4.5.

Table 4.5 : Maximum Fundamental Period for x Direction

storey	mass [ton]	d_{fi}	F_{fi}	$m_i\, d_{fi}^2$	$F_{fi}\, d_{fi}$
6	94.34	0.11	580.46	1.10	62.69
5	106.88	0.10	545.39	1.03	53.61
4	106.88	0.08	433.17	0.71	35.30
3	107.11	0.06	321.63	0.40	19.65
2	107.33	0.04	209.61	0.14	7.61
1	128.27	0.01	115.82	0.02	1.38
			T max	**0.8628455 sec**	

Same criteria are also applied for y direction and it is validated that the initially period is shorter than the maximum allowed value 0.825 sec.

Lateral loads acting on each storey are determined on the following lines according to Equivalent Seismic Load Method.

Local Site Class: Z2 (T_A = 0.15 sec, T_B = 0.40 sec)

$A_0 = 0.30$

$I = 1.00$

$W = 6384.50kN$

$$S(T) = 2.5 \cdot \left(\frac{T_B}{T}\right)^{0.8} \quad \textbf{(4.4)}$$

Spectrum coefficients $S(T)$ are 1.35 and 1.42 for x and y directions respectively.

Spectral Acceleration Coefficient

$$A(T) = A_0 \cdot I \cdot S(T) \quad \textbf{(4.5)}$$

Spectral accelerations in terms of g for both directions are given below.

$A(T_x) = 0.30 \cdot 1 \cdot 1.35 = 0.41$

$A(T_y) = 0.30 \cdot 1 \cdot 1.42 = 0.43$

$R_a = 1.00$

$$Vt = \frac{\lambda \cdot W \cdot A(T_1)}{R_a} \quad \textbf{(4.6)}$$

$$Vt_x = \frac{0.85 \cdot 6384.5 \cdot 0.41}{1} = 2206.09 kN$$

$$Vt_y = 2314.37 kN$$

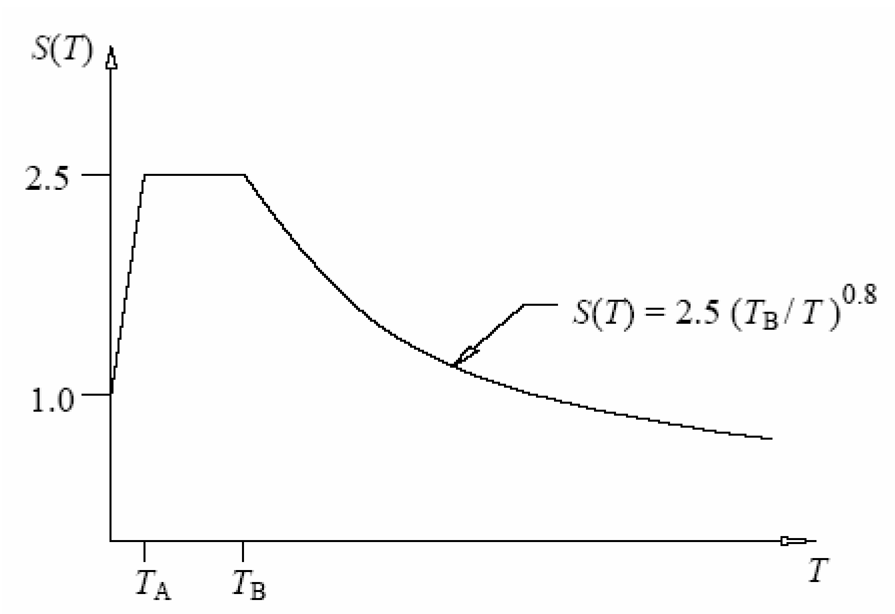

Figure 4.5 : Elastic Response Spectrum

Table 4.6 : Distribution of Horizontal Forces at Storeys

STOREY	m i [ton]	HEIGHT[m]	Hi [m]	Wi*Hi [kNm]	Fix [kN]	Fiy [kN]
6	94.34	2.50	14.65	13558.53	580.46	608.95
5	106.88	2.50	12.15	12739.43	545.39	572.16
4	106.88	2.50	9.65	10118.13	433.17	454.43
3	107.11	2.50	7.15	7512.65	321.63	337.41
2	107.33	2.50	4.65	4896.13	209.61	219.90
1	128.27	2.15	2.15	2705.39	115.82	121.51
	650.81			**51530.27**	**2206.09**	**2314.37**

Torsional Irregularity exists whenever the maximum displacement in a storey in each direction exceeds more than 20% of the average displacement of corresponding direction. Table 4.7 states that there is not any existing torsional irregularity neither with x nor y direction seismic actions.

Table 4.7 : Maximum and Average Displacements at Storey Levels

X DIRECTION						
STOREY	1	2	3	4	5	6
MAXIMUM	0.0119	0.0363	0.0611	0.0821	0.0983	0.1080
AVERAGE	0.0118	0.0360	0.0606	0.0815	0.0977	0.1074
RATIO	**1.0076**	**1.0078**	**1.0076**	**1.0073**	**1.0062**	**1.0059**
Y DIRECTION						
STOREY	1	2	3	4	5	6
MAXIMUM	0.0094	0.0308	0.0529	0.0728	0.0882	0.0975
AVERAGE	0.0093	0.0306	0.0527	0.0726	0.0880	0.0972
RATIO	**1.0132**	**1.0056**	**1.0038**	**1.0031**	**1.0028**	**1.0026**

Since the investigated building has a height lower than 25 meters and also does not have any torsional irregularity, Equivalent Seismic Load Method is usable as linear analysis. Additionally, having more than 70% mass participation for each fundamental period also make the Incremental Equivalent Seismic Load Method applicable as non-linear analysis for case study building.

The horizontal load distribution along stories which were calculated on previous paragraphs will be used directly in linear static analysis and also be used in non-linear static analysis with monotonically increments (push-over analysis).

4.2.1 Linear Analysis with Equivalent Seismic Load Method

In this section, seismic performance assessment of the case study building will be performed with TEC’07 linear static analysis by using Equivalent Seismic Load Method. Assessment procedure is summarized on a flowchart given in Figure 4.6.

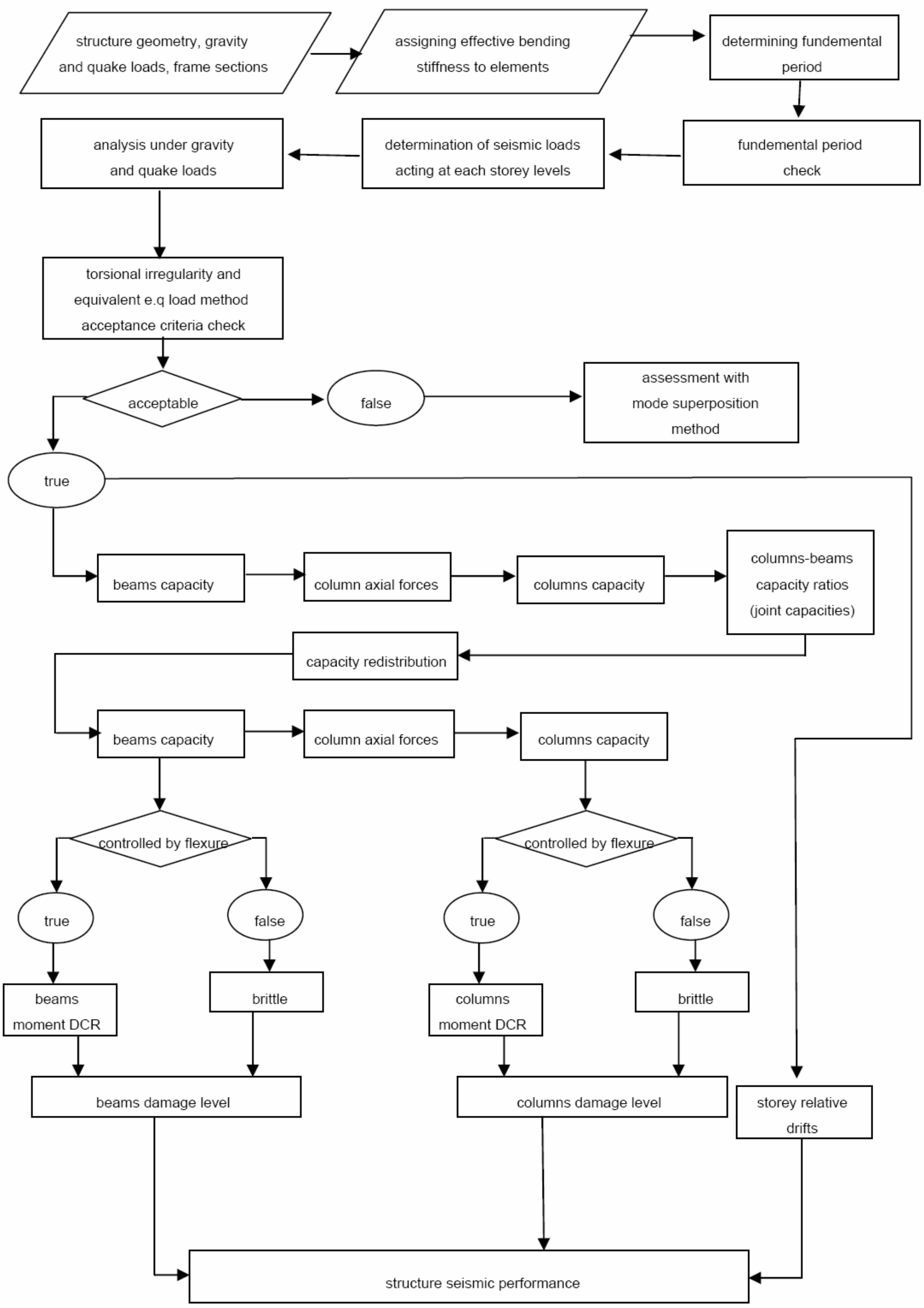

Figure 4.6 : Linear Static Analysis Flowchart

4.2.1.1 Relative Storey Drift Ratio

Relative drift ratios of vertical components under seismic actions for both directions are shown in Table 4.8.

Table 4.8 : Relative Storey Drift Ratios

storey	disp x [m]	rel. disp.x [m]	disp y [m]	rel. disp.x [m]	h [m]	drift ratio x	drift ratio y
1	0.0119	0.0119	0.0094	0.0094	2.15	0.006	0.004
2	0.0363	0.0244	0.0308	0.0214	2.5	0.010	0.009
3	0.0611	0.0248	0.0529	0.0221	2.5	0.010	0.009
4	0.0821	0.021	0.0728	0.0199	2.5	0.008	0.008
5	0.0983	0.0162	0.0882	0.0154	2.5	0.006	0.006
6	0.108	0.0097	0.0975	0.0093	2.5	0.004	0.004
					max	**0.010**	**0.009**

In calculations using linear elastic methods, relative drift ratios of vertical components under any direction of seismic actions must not exceed the value for the corresponding damage limits given in Table 2.6. Maximum drift ratio value for x direction exceeds the minimum damage limit. As a result, even if the components represents *slight damage level* due to the demand-capacity ratio verification, they will be considered as at *moderate damage level* under seismic actions acting along x direction.

4.2.1.2 Moment Capacities at Beam Ends

Determination of end point moment capacity will be explained in detail for beam 1101, and the results for all beams at first storey will be summarized.

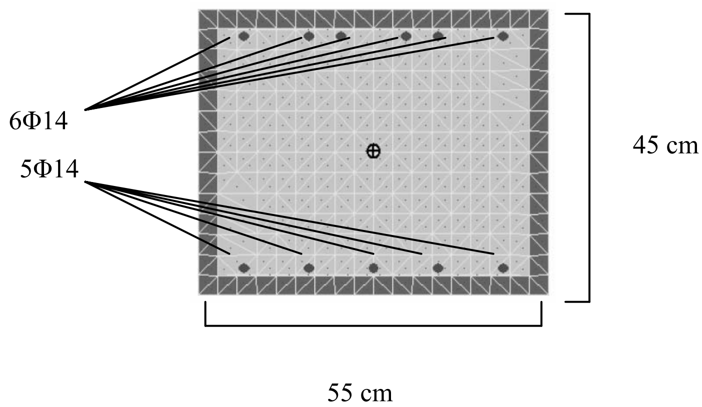

Figure 4.7 : Beam 1101 Left-end Cross-sectional Detail

After defining confined, unconfined concrete and reinforcement steel material properties to XTRACT [5] cross-section analysis software, moment-curvature plots can be obtained as an output (Figure 4.8). Unconfined and confined concrete material properties are defined and introduced to cross-section models in the software. In 1988, Mander proposed reinforced concrete member stress-strain model which takes confinement effect into account, [31]. In Table 4.9 end-point moment capacities of Beam 1101 are summarized.

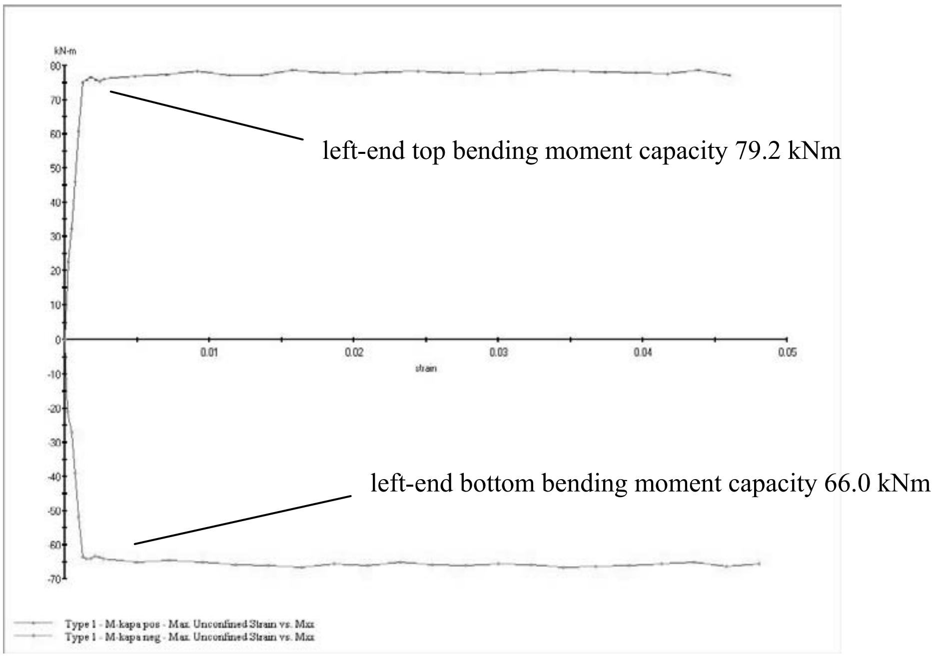

Figure 4.8 : Beam 1101 Moment-Curvature Graph

Table 4.9 : Beam 1101 End-point Moment Capacities

Bottom Moment Cap. [kNm]	
i	j
66.0	66.0

Table 4.10 : First Storey Beam Bending Moment Capacities

Beam	Top Moment Cap. [kNm]		Bottom Moment Cap. [kNm]	
	i	j	i	j
1101	79.21	79.21	66.01	66.01
1102	79.21	79.21	66.01	66.01
1103	79.21	79.21	66.01	66.01
1104	79.21	79.21	66.01	66.01
1105	57.54	57.54	46.03	46.03
1106	57.54	57.54	46.03	46.03
1107	57.54	57.54	46.03	46.03
1108	57.54	57.54	46.03	46.03
1109	57.54	57.54	46.03	46.03
1110	57.54	57.54	46.03	46.03
1111	79.21	79.21	66.01	66.01
1112	79.21	79.21	66.01	66.01
1114	79.21	79.21	66.01	66.01
1115	79.21	79.21	66.01	66.01
1116	79.21	79.21	66.01	66.01
1117	79.21	79.21	66.01	66.01
1118	57.54	57.54	46.03	46.03
1119	57.54	57.54	46.03	46.03
1121	57.54	57.54	46.03	46.03
1122	57.54	57.54	46.03	46.03
1124	57.54	57.54	46.03	46.03
1125	57.54	57.54	46.03	46.03
1126	57.54	57.54	46.03	46.03
1127	79.21	79.21	66.01	66.01
1128	79.21	79.21	66.01	66.01

4.2.1.3 Moment Capacities of Columns

On the following lines, evaluation of residual axial force demand acting on Column 101 under x direction seismic actions will be explained in detail. Maximum axial force acting from seismic action is required for obtaining the residual moment capacity with axial force – bending moment interaction relation. After the example calculation for Column 101 is introduced, results for other columns will be summarized.

Left bottom moment capacity of Beam 1101 = 66.01 kNm

Left-end moment demand from gravity loads = -16.83 kNm

Residual Moment Capacity of Beam left-end = 66.01-(-16.83) = 82.84 kNm

Right top moment capacity of Beam 1101 = 79.21 kNm

Right-end moment demand from gravity loads = 23.07 kNm

Residual Moment Capacity of Beam right-end = 79.21-23.07 = 56.14 kNm

Span length of Beam 1101 = 3.65 m

Residual Axial Force acting on Column 101 from Beam 1101

= - (82.84 + 56.14) / 3.65 = -36.66 kN

Residual Axial Force acting on Column 101 from Beam 1201 = -41.49 kN

Residual Axial Force acting on Column 101 from Beam 1301 = -42.00 kN

Residual Axial Force acting on Column 101 from Beam 1401 = -42.05 kN

Residual Axial Force acting on Column 101 from Beam 1501 = -41.91 kN

Residual Axial Force acting on Column 101 from Beam 1601 = -40.91 kN

Total Residual Axial Force acting from beams

= -36.66 - 41.49 – 42.00 – 42.05 – 41.91 – 40.91 = -244.99 kN

Total Demand Axial force can be obtained by adding gravity load demand to residual axial forces

Total Axial Force Demand acting on Column 101 = -244.99 + 217.84 = 27.15 kN

Total Bending Capacity of Column 101 top-end section is evaluated as 43.1 kNm by using interaction curve as shown in Figure 4.9.

Bending capacity of Column 201 bottom-end section is also calculated with the same procedure above and it is found as 42.9 kNm

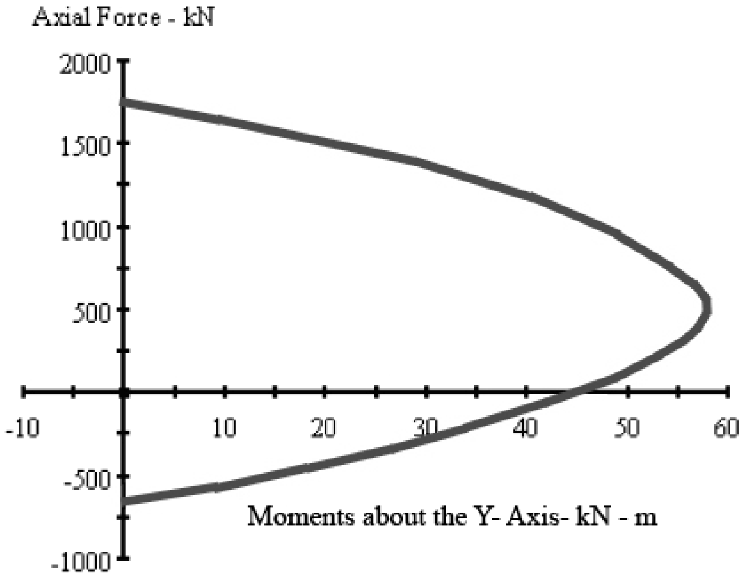

Figure 4.9 : Column 101 Moment – Axial Force Interaction Curve

In order to check whether if the hinges are developing at beam ends or not, moment capacity comparison must be done at column-beam joints.

At joint 201 where Column 101, Column 201 and Beam 1101 intersect:

$(M_{C101} + M_{C201}) / (M_{B1101}) = (43.1 + 42.9) / 66.01 = 1.30$

Same procedures are done for upper joints

At Joint 301 = 1.11

At Joint 401 = 1.04

At joint 501 = 0.98

At joint 601 = 0.96

At joint 701 = 0.47

Since beams along x direction connecting to joint 501, 601 and 701 are stronger than columns, beam capacities relevant to these joints must be multiplied with 0.98, 0.96 and 0.47 respectively. After reducing the moment capacities of beams, moment capacities of columns are recalculated since the axial force transferred to the connecting columns are also changed.

Subtracting moment demand of gravity loads from the total moment capacity gives the residual moment capacity. Residual moment capacity values of corresponding columns are given in Table 4.11.

Table 4.11 : Residual Moment Capacities of Columns

column	NG+ NE [kN]	total moment cap [kNm]	bottom residual [kNm]	top residual [kNm]
101	-21.0	43.0	44.5	39.9
102	488.6	49.0	48.2	47.5
103	436.2	47.0	47.0	47.0
104	357.2	48.0	47.7	48.5
105	422.1	46.0	45.5	50.0
106	161.4	48.0	49.8	44.4
107	636.5	62.0	61.4	63.3
108	-58.7	225.0	223.8	227.6
109	726.6	272.0	272.9	269.9
110	741.4	62.0	62.8	60.3
111	596.4	46.0	45.3	50.4
112	1.9	43.0	44.7	39.6
113	408.7	47.0	46.7	47.7
114	444.0	48.0	47.1	49.8
115	21.9	47.0	47.9	45.2
116	341.0	47.0	47.3	46.4
117	489.9	47.0	45.3	50.4

4.2.1.4 Safety Verification against Shear Failure

In order to classify the structural components as ductile or brittle elements, their safety verification against shear failure must be achieved. The main principle of any structural component being controlled by flexure is its shear capacity is exceeded after its moment capacity. Otherwise the component is controlled by force.

Shear force check for Column 101 will be explained specifically and the results for whole components will be summarized later on.

Shear Resistance of Column 101

$Vr = Vc + Vw = 0.8 \times 0.65 \times fctm \times d \times (1+ \gamma \times N / Ac)$

$= 0.8 \times 0.65 \times 0.9 \times 450 \times 220 \times (1 + 0.07 \times 28.2 \times 1000 / (450 \times 250) = 51480$ N

= 51.48 kN

Vw = A_{sw} x f_{yw} x d / s = 48.63 kN

Vr = 100.11 kN

The maximum shear is determined on the basis of end point bending moment capacities. The stronger column – weaker beam condition must be checked at both column end joints in order to determine exact moment capacities at ends.

$(M_{C101} + M_{C201}) / (M_{B1101}) = (43.1 + 42.9) / 66.01 = 1.30$

VE = Min((Mtop + Mbottom) / column height),(shear demand from seismic actions))

VE = 86 / 2.15 = 40 kN

VE < Vr , thus Column 101 is controlled by flexure.

Table 4.12 : Ductility Condition of Beams

beam	VE [kN]	Vr [kN]	controlled by
1101	36.66	166.54	flexure
1102	27.53	166.54	flexure
1103	27.01	166.54	flexure
1104	37.95	166.54	flexure
1105	26.45	137.10	flexure
1106	31.07	137.10	flexure
1107	96.72	137.10	flexure
1108	59.82	137.10	flexure
1109	32.70	137.10	flexure
1110	30.80	137.10	flexure
1111	37.64	166.54	flexure
1112	41.45	166.54	flexure
1114	42.91	166.54	flexure
1115	35.50	166.54	flexure
1116	38.07	166.54	flexure
1117	38.32	166.54	flexure
1118	26.92	137.10	flexure
1119	27.88	137.10	flexure
1121	27.43	137.10	flexure
1122	27.13	146.72	flexure
1124	27.44	137.10	flexure
1125	26.56	137.10	flexure
1126	28.19	137.10	flexure
1127	38.08	166.54	flexure
1128	38.31	166.54	flexure

Table 4.13 : Ductility Conditions of Columns

column	VE [kN]	V r [kN]	controlled by
101	40.0	100.1	flexure
102	45.6	114.5	flexure
103	43.7	114.0	flexure
104	44.7	111.2	flexure
105	42.8	115.5	flexure
106	44.7	104.8	flexure
107	57.7	126.3	flexure
108	209.3	235.2	flexure
109	253.0	260.5	flexure
110	57.7	153.1	flexure
111	42.8	120.3	flexure
112	40.0	100.1	flexure
113	43.7	111.6	flexure
114	44.7	116.4	flexure
115	43.7	100.1	flexure
116	43.7	112.9	flexure
117	43.7	115.8	flexure

4.2.1.5 Damage Levels of Structural Elements

Demand-capacity ratios are determined by dividing moments acting on ductile elements by the residual moment capacities calculated from previous sections. Later on, these demand-capacity ratios (r) are compared with the values for corresponding damage limits for beams and columns given in Table 2.3 and Table 2.4 respectively. Damage levels of critical storey structural components are specified on Table 4.14, Table 4.15, Table 4.16 and Table 4.17.

Table 4.14 : Column Damage Levels under x-Direction Seismic Actions

column	x direction					
	shear	r	r limit ML	r limit SL	r limit CL	damage
101	3.6%	2.20	2.00	3.50	5.00	moderate
102	3.5%	1.80	1.50	2.00	3.00	moderate
103	3.4%	1.79	2.02	3.56	5.08	slight
104	3.5%	1.82	2.14	3.91	5.55	slight
105	3.6%	2.16	2.04	3.62	5.17	moderate
106	2.8%	1.55	2.43	4.78	6.71	slight
107	6.3%	2.66	1.50	2.00	3.00	heavy
108	19.7%	3.37	1.58	2.66	3.74	heavy
109	19.9%	2.77	1.00	1.50	2.00	collapse
110	9.9%	4.24	1.50	2.00	3.00	collapse
111	2.9%	1.73	1.50	2.00	3.00	moderate
112	3.4%	2.09	2.00	3.50	5.00	moderate
113	3.8%	1.96	2.06	3.68	5.24	slight
114	3.3%	1.98	2.01	3.53	5.04	slight
115	3.3%	1.95	2.00	3.50	5.00	slight
116	3.7%	1.93	2.16	3.98	5.65	slight
117	3.4%	2.07	1.50	2.00	3.00	heavy

Table 4.15 : Column Damage Levels under y-Direction Seismic Actions

column	y direction					
	shear	r	r limit ML	r limit SL	r limit CL	damage
101	6.1%	1.88	1.83	3.16	4.50	moderate
102	4.6%	1.35	1.55	2.48	3.53	slight
103	4.2%	1.23	1.54	2.45	3.49	slight
104	4.5%	1.35	1.58	2.53	3.60	slight
105	5.9%	2.06	1.89	3.28	4.67	moderate
106	7.4%	1.55	1.38	2.15	3.03	moderate
107	10.0%	1.87	1.00	1.50	2.00	heavy
108	3.5%	1.54	2.45	4.86	6.82	slight
109	3.5%	1.51	2.47	4.91	6.88	slight
110	11.4%	2.14	1.07	1.57	2.15	heavy
111	7.2%	1.53	1.37	2.13	3.00	moderate
112	6.1%	1.57	1.21	1.71	2.42	moderate
113	4.7%	1.24	1.19	1.69	2.38	moderate
114	5.2%	1.46	1.37	2.13	3.00	moderate
115	5.2%	1.45	1.19	1.69	2.38	moderate
116	4.6%	1.22	1.20	1.70	2.40	moderate
117	5.9%	1.56	1.42	2.22	3.15	moderate

Table 4.16 : Beam Damage Levels under x-Direction Seismic Actions

beam	x direction				
	r	r limit ML	r limit SL	r limit CL	damage
1101	2.50	2.50	4.00	6.00	moderate
1102	1.10	2.50	4.00	6.00	slight
1103	1.25	2.50	4.00	6.00	slight
1104	2.36	2.50	4.00	6.00	slight
1105	3.09	2.50	4.00	6.00	moderate
1106	6.55	2.50	4.00	6.00	collapse
1107	6.60	2.50	4.00	6.00	collapse
1108	8.74	2.50	4.00	6.00	collapse
1109	4.97	2.50	4.00	6.00	heavy
1110	3.02	2.50	4.00	6.00	moderate
1111	2.32	2.50	4.00	6.00	slight
1112	2.76	2.50	4.00	6.00	moderate
1114	2.85	2.50	4.00	6.00	moderate
1115	2.28	2.50	4.00	6.00	slight
1116	0.05	2.50	4.00	6.00	slight
1117	0.07	2.50	4.00	6.00	slight
1118	0.09	2.50	4.00	6.00	slight
1119	0.02	2.50	4.00	6.00	slight
1121	0.14	2.50	4.00	6.00	slight
1122	0.00	2.50	4.00	6.00	slight
1124	0.14	2.50	4.00	6.00	slight
1125	0.09	2.50	4.00	6.00	slight
1126	0.01	2.50	4.00	6.00	slight
1127	0.05	2.50	4.00	6.00	slight
1128	0.07	2.50	4.00	6.00	slight

Table 4.17 : Beam Damage Levels under y-Direction Seismic Actions

beam	y direction				
	r	r limit ML	r limit SL	r limit CL	damage
1101	0.01	2.50	4.00	6.00	slight
1102	0.02	2.50	4.00	6.00	slight
1103	0.01	2.50	4.00	6.00	slight
1104	0.01	2.50	4.00	6.00	slight
1105	0.01	2.50	4.00	6.00	slight
1106	0.08	2.50	4.00	6.00	slight
1107	0.40	2.50	4.00	6.00	slight
1108	0.73	2.50	4.00	6.00	slight
1109	0.05	2.50	4.00	6.00	slight
1110	0.01	2.50	4.00	6.00	slight
1111	0.00	2.50	4.00	6.00	slight
1112	0.01	2.50	4.00	6.00	slight
1114	0.00	2.50	4.00	6.00	slight
1115	0.01	2.50	4.00	6.00	slight
1116	3.21	2.50	4.00	6.00	moderate
1117	3.43	2.50	4.00	6.00	moderate
1118	5.38	2.50	4.00	6.00	heavy
1119	4.40	2.50	4.00	6.00	heavy
1121	3.60	2.50	4.00	6.00	moderate
1122	3.88	2.50	4.00	6.00	moderate
1124	3.60	2.50	4.00	6.00	moderate
1125	5.50	2.50	4.00	6.00	heavy
1126	4.33	2.50	4.00	6.00	heavy
1127	3.16	2.50	4.00	6.00	moderate
1128	3.38	2.50	4.00	6.00	moderate

Summary of structural element damage levels also considering the storey drift ratios are given visually in Figure 4.10 and Figure 4.11.

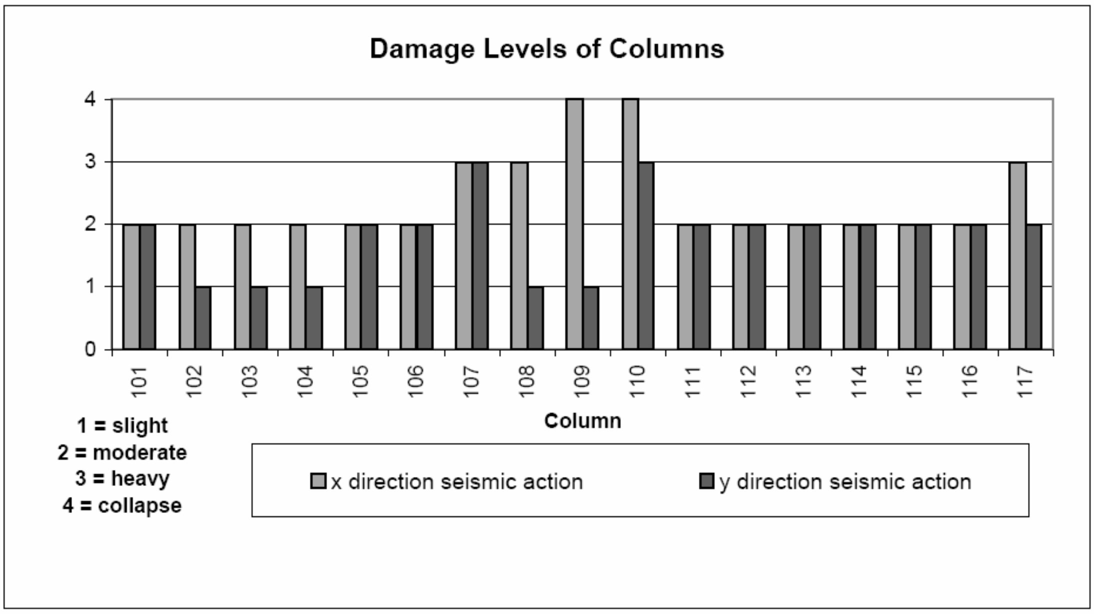

Figure 4.10 : Damage Levels of Columns

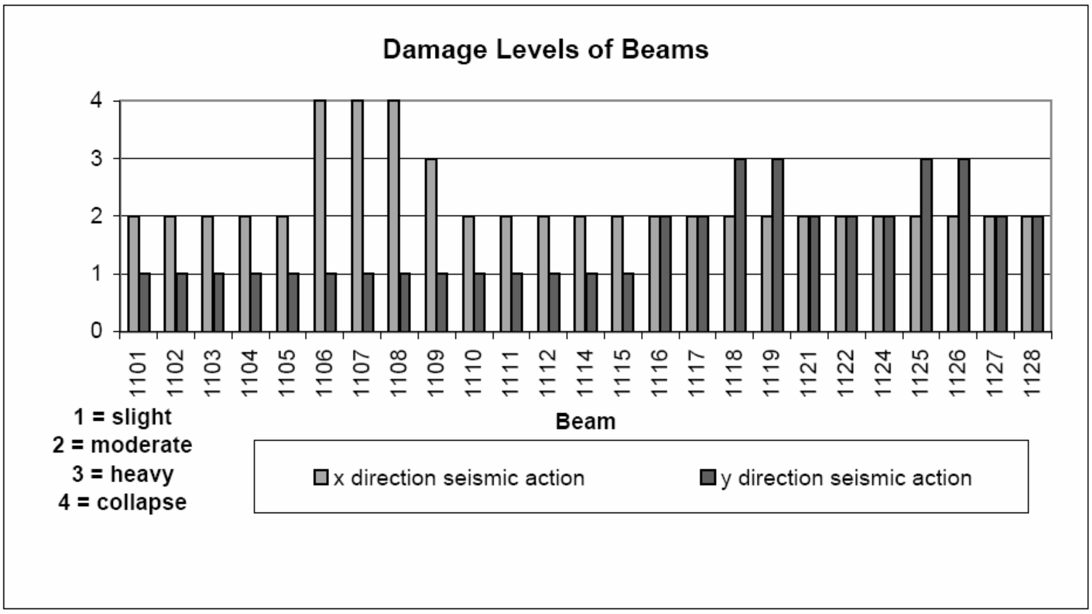

Figure 4.11 : Damage Levels of Beams

4.2.1.6 Seismic Performance of the Structure

From linear static analysis results, building is more vulnerable when seismic actions affect along x direction. Since relative drift ratio of vertical elements does not satisfy the *minimum damage limit* case, all elements at *slight damage level* must be assumed as *moderate damage level*.

While 84% of beams at critical storey are moderately damaged, 12% of overall beams are collapsed. The remaining beams are heavily damaged.

For the vertical structural elements at the critical storey, 11.8% of columns collapse due to the analysis. This situation gives the building seismic performance of *State of Collapse* without questioning.

4.2.2 Non-linear Analysis with Push-over Method

In this section, seismic performance assessment of the case study building will be performed with TEC’07 non-linear static analysis by using Incremental Equivalent Seismic Load Method. Assessment procedure is summarized on a flowchart given in

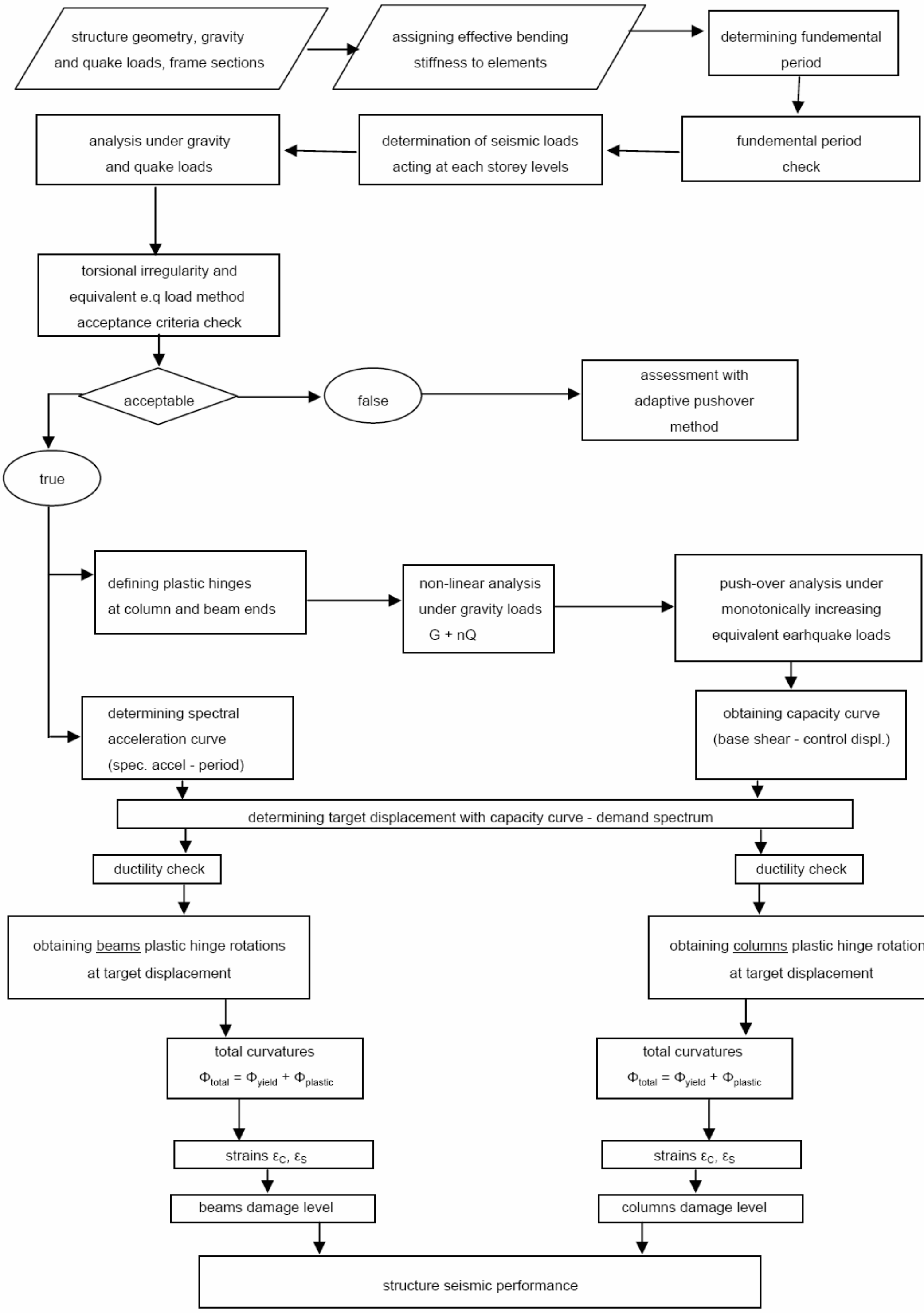

Figure 4.12 : Non-linear Static Analysis Flowchart

Plastic hinges are defined to the mathematical model at each column and beam ends. Mean values of mechanical properties are introduced when defining hinges yielding surfaces. Confidence factor confirmed from the knowledge level is multiplied to capacity values. Maximum compression strain of concrete material and maximum tension strain of reinforcement steel material are taken 0.003 and 0.01 respectively. Yielding surface data for each plastic hinge created is entered to the mathematical model as input.

Example Calculation for Column 101

Plastic cross-section surfaces are created by XTRACT [5] cross-section analysis software program. For Column 101, where the cross-section details are given in Figure 4.13, Mander unconfined concrete model is used at outer zone from the transverse reinforcement and Mander confined concrete model is used for the core material. Hardening of steel material is neglected. Maximum compression strain of concrete material and maximum tension strain of reinforcement steel material are taken 0.003 and 0.01 respectively.

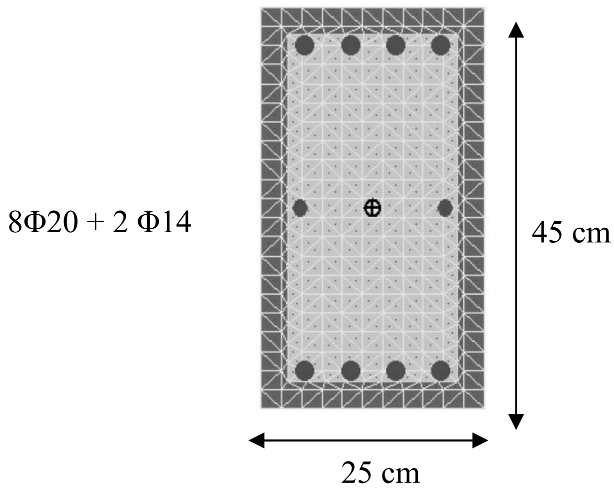

Figure 4.13 : Cross-sectional Detail of Column 101

The bending moment about x-axis and axial force interaction curve which is obtained from cross-section analysis is shown in Figure 4.14. An interaction surface is created and entered to the structural analysis program as input file after moment-axial force interaction curves performed for 0^{o}, 45^{o} and 90^{o}.

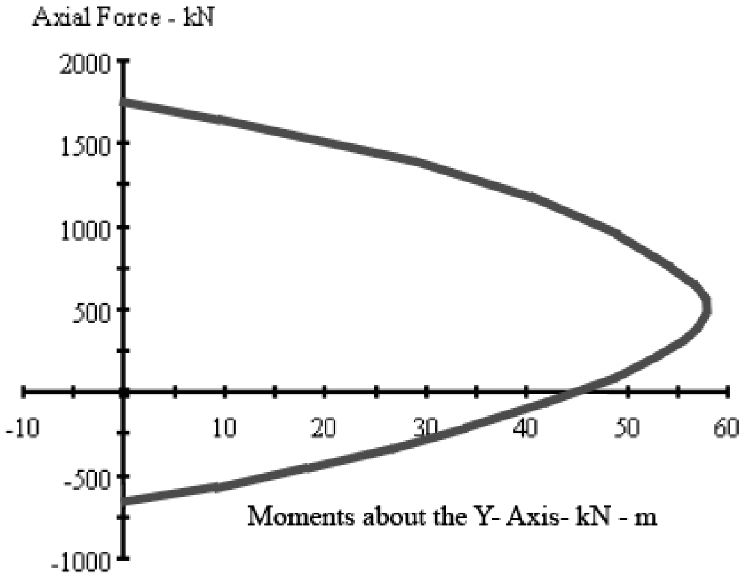

Figure 4.14 : Column 101 Moment – Axial Force Interaction Curve

Example Calculation for Beam 1101

Bending moment – curvature plots are created by XTRACT [5] cross-section analysis software program. For Beam 1101, where the cross-section details are given in Figure 4.15, Mander [31] unconfined concrete model is used at outer zone from the transverse reinforcement and Mander confined concrete model is used for the core material. Hardening of steel material is neglected. Maximum compression strain of concrete material and maximum tension strain of reinforcement steel material are taken 0.003 and 0.01, respectively.

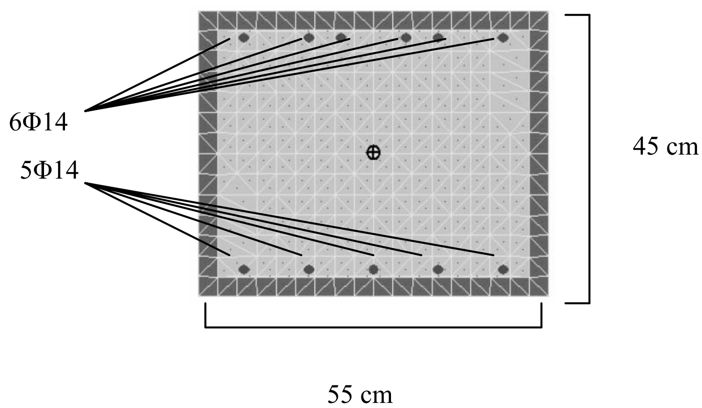

Figure 4.15 : Beam 1101 Left-end Cross-sectional Detail

At the end of cross-section analysis, yielding moment capacity values are determined as follows:

Positive bending moment (Tension at bottom)

$M_{pa}^{+} = 66.0$ kNm

Negative bending moment (Tension at top)

$M_{pa}^{-} = 79.2$ kNm

Internal force – deformation relation can be seen from Figure 4.16. Internal force-deformation relation is introduced to the structural analysis program which will perform the push-over analysis.

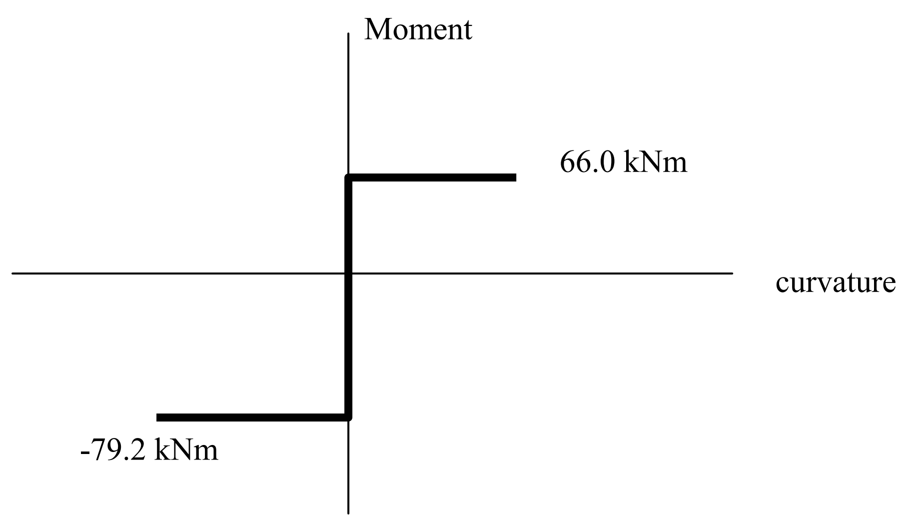

Figure 4.16 : Beam 1101 Left-end Moment-Curvature Graph

Defining yielding curves procedures for given examples are repeated for every point on system where a plastic hinge may develop. Plastic hinges are defined at every column and beam end points as well as at beam mid span in case of possible plastic deformation because of gravity loadings (Figure 4.17).

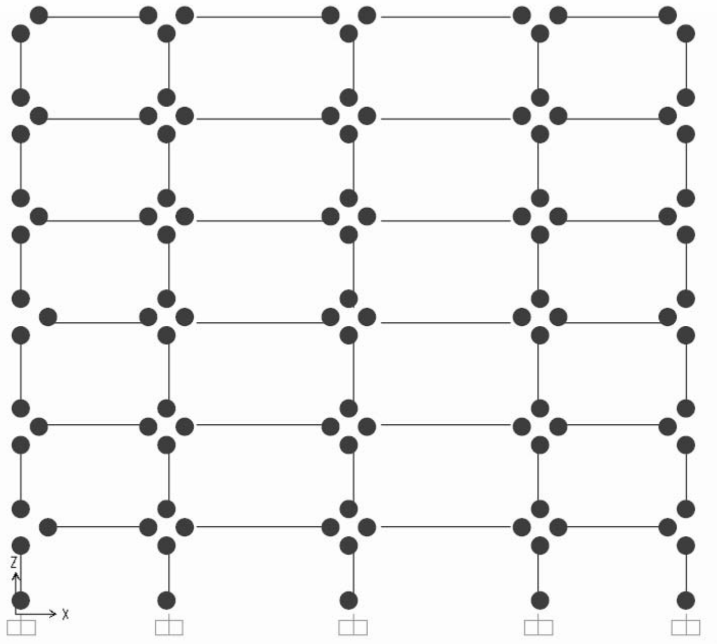

Figure 4.17 : Plastic Hinges Defined on Column and Beam Ends at C Axis

Structural system is pushed up to any predefined displacement under constant gravity loads and monotonically increasing equivalent seismic loads. Within scope of this analysis, modal mass and modal participation factor and modal shape of fundamental modes are also determined, Table 4.18 - Table 4.20

Table 4.18 : Modal Mass Ratios

Mode	Period [sec]	UX	UY
1	0.871	0.7729	0.0000
2	0.824	0.0000	0.7587
3	0.709	0.0008	0.0000
4	0.289	0.1119	0.0000
5	0.274	0.0000	0.1148
6	0.237	0.0001	0.0000

Table 4.19 : Modal Participation Factors

Mode	Period [sec]	UX [kNs^2]	UY [kNs^2]
1	0.871	22.290	0.011
2	0.824	-0.015	22.084
3	0.709	0.713	0.141
4	0.289	-8.480	-0.005
5	0.274	-0.007	8.591
6	0.237	-0.229	-0.058

Table 4.20 : Horizontal Displacements at Fundamental Periods

Storey	UX	UY
6	0.0583	0.0589
5	0.0544	0.05314
4	0.0455	0.04384
3	0.0339	0.0318
2	0.0201	0.0187
1	0.0065	0.00569

Base shear and control node displacement values are given in Table 4.21 for each step of push-over analyses separately for x and y directions.

Table 4.21 : Base Shear – Top Displacement Values

step	u_x [m]	base shear x [kN]	step	u_y [m]	base shear y [kN]
1	0.000	0.0	1	0.000	0.0
2	0.012	249.2	2	0.018	394.8
3	0.045	750.4	3	0.035	701.6
4	0.064	893.3	4	0.045	790.3
5	0.072	925.0	5	0.073	902.7
6	0.072	812.3	6	0.110	978.1
7	0.079	847.3	7	0.172	1029.4
8	0.079	847.3	8	0.185	1036.7
9	0.079	847.3	9	0.255	1065.4
10	0.079	841.3	10	0.315	1086.7
11	0.092	877.7	11	0.319	1087.7
12	0.092	875.8	12	0.319	1086.6
13	0.093	879.8	13	0.374	1109.9
14	0.117	934.7	14	0.374	1097.9
15	0.117	934.8	15	0.374	1101.8
16	0.118	939.8	16	0.375	1102.9
17	0.122	950.9	17	0.375	1103.5
18	0.122	951.5	18	0.375	1099.4
19	0.125	966.8	-	-	-
20	0.129	976.1	-	-	-

Under cover of Table 4.21, push-over curves are sketched. Push-over curves for x-direction and y-direction analyses are shown together at same coordinates in Figure 4.18.

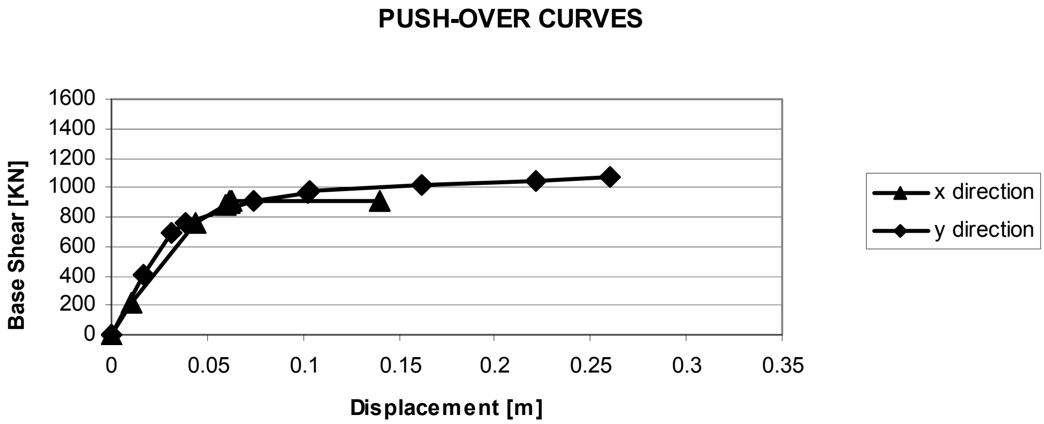

Figure 4.18 : Push-over Curves according to TEC'07

Coordinate conversion is applied for push-over curves to achieve modal displacement and modal acceleration values by using equations (2.2) and (2.3). Modal displacement and modal acceleration values are shown in for two separate push-over curves.

Table 4.22 : Modal Acceleration and Modal Displacement Values

x direction		y direction	
a_1 x	d_1 x	a_1 y	d_1 y
0.000	0.000	0.000	0.000
0.495	0.010	0.799	0.014
1.492	0.035	1.420	0.026
1.776	0.049	1.600	0.034
1.839	0.055	1.827	0.055
1.615	0.055	1.980	0.084
1.684	0.062	2.084	0.131
1.684	0.062	2.099	0.140
1.684	0.062	2.157	0.194
1.672	0.062	2.200	0.238
1.745	0.071	2.202	0.241
1.741	0.071	2.200	0.241
1.749	0.072	2.247	0.283
1.858	0.091	2.223	0.283
1.858	0.091	2.230	0.284
1.868	0.091	2.233	0.284
1.890	0.095	2.234	0.284
1.891	0.095	2.226	0.284
1.922	0.097	-	-
1.940	0.100	-	-
mass	503.080	mass	493.969
Φ_{i1}	0.058	Φ_{i1}	0.059
Γ_{X1}	22.133	Γ_{y1}	22.412

Elastic response spectrum curve is also converted to spectral acceleration – spectral displacement curve modal capacity curve of the structure is also shown at the same sketch. Linear part of the modal capacity curve is extended up to spectral acceleration curve and the gives the linear spectral displacement value as shown in

Figure 4.19 and Figure 4.20.

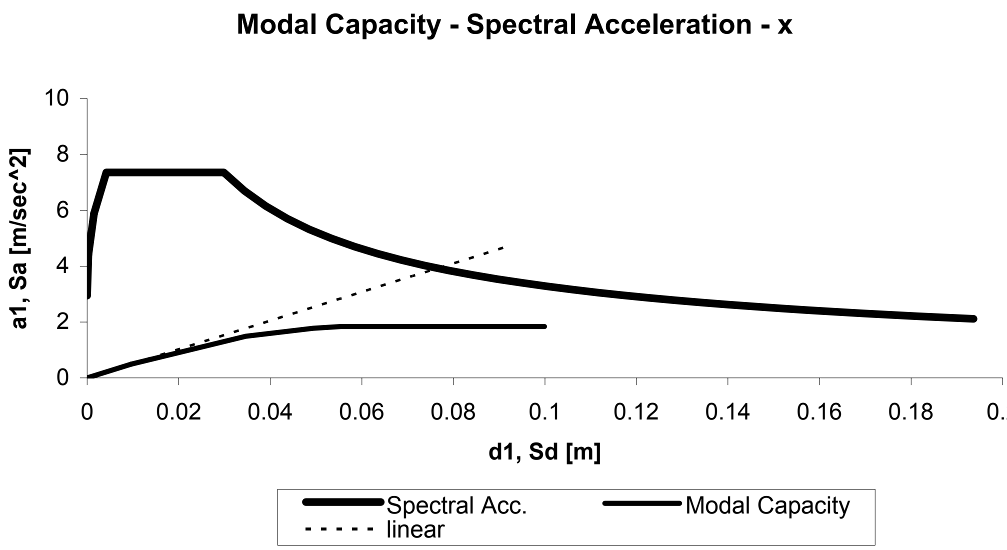

Figure 4.19 : Spectral Acceleration – Spectral Displacement Diagram (X-Dir.)

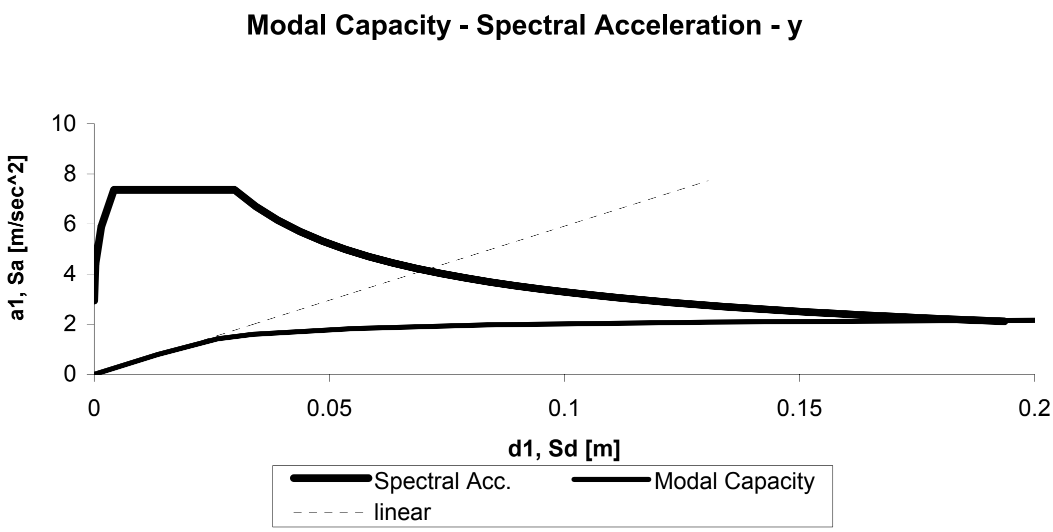

Figure 4.20 : Spectral Acceleration – Spectral Displacement Diagram (Y-Dir.)

Since the initial period in

Figure **4.19** is longer than the characteristic period T_B, non-linear spectral displacement is equal to the linear spectral displacement on the basis of *equal displacement rule*.

Modal displacement demand is equal to the non-linear spectral displacement value under scope of equations from (2.4) to (2.6).

Target displacements are calculated for two separate push-over analysis along x and y directions with equation (2.9) as 0.1019 m and 0.0974 m respectively.

Control node of the structure is pushed up to these target displacement and plastic rotation demands are investigated. Plastic hinges are shown on Figure 4.21 as for x direction and y direction push-over analyses.

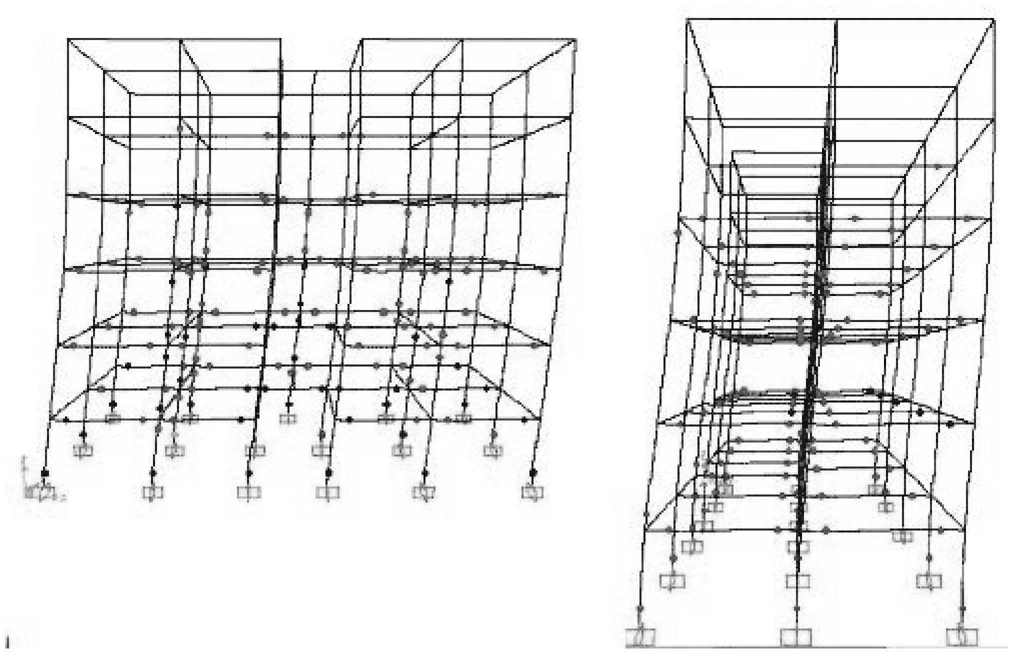

Figure 4.21 : Developed Plastic Hinges

Plastic curvatures are obtained by dividing plastic rotation by plastic hinge length as stated in equation (2.10).

Total curvature demand is calculated by adding plastic curvature with yield curvature value, Equation (2.11).

Curvature demands for beams at first storey are calculated with the procedure explained above and shown at Table 4.23.

Table 4.23 : Total Curvature Demands at Beam Ends

BEAM	HINGE	θ P [RAD]	h [m]	Lp [m]	K P	K yield	K total
1116	LEFT	0.00593	0.45	0.225	0.02636	0.00980	0.03616
1116	RIGHT	-0.00675	0.45	0.225	0.03000	0.00980	0.03980
1117	LEFT	0.00503	0.4	0.2	0.02515	0.01103	0.03618
1117	RIGHT	-0.00792	0.4	0.2	0.03960	0.01103	0.05063
1118	LEFT	0.00549	0.4	0.2	0.02745	0.01103	0.03848
1118	RIGHT	-0.00734	0.4	0.2	0.03670	0.01103	0.04773
1119	LEFT	0.00474	0.4	0.2	0.02370	0.01103	0.03473
1119	RIGHT	-0.00825	0.4	0.2	0.04125	0.01103	0.05228
1121	LEFT	0.00596	0.4	0.2	0.02980	0.01103	0.04083
1121	RIGHT	-0.00661	0.4	0.2	0.03305	0.01103	0.04408
1122	LEFT	0.00394	0.4	0.2	0.01970	0.01103	0.03073
1122	RIGHT	-0.00828	0.4	0.2	0.04140	0.01103	0.05243
1124	LEFT	0.00571	0.4	0.2	0.02855	0.01103	0.03958
1124	RIGHT	-0.00655	0.4	0.2	0.03275	0.01103	0.04378
1125	LEFT	0.00543	0.4	0.2	0.02715	0.01103	0.03818
1125	RIGHT	-0.00702	0.4	0.2	0.03510	0.01103	0.04613
1126	LEFT	0.00438	0.4	0.2	0.02190	0.01103	0.03293
1126	RIGHT	-0.00827	0.4	0.2	0.04135	0.01103	0.05238
1127	LEFT	0.00569	0.45	0.225	0.02529	0.00980	0.03509
1127	RIGHT	-0.00675	0.45	0.225	0.03000	0.00980	0.03980
1128	LEFT	0.00504	0.45	0.225	0.02240	0.00980	0.03220
1128	RIGHT	-0.00798	0.45	0.225	0.03547	0.00980	0.04527

Concrete compression strain and steel tension strain demand are achieved by using curvature demands in bending moment – curvature relations. It is observed that a linear relation exists between curvature and strain values at beams. Strain demands of unconfined concrete, confined concrete and steel materials are achieved and those demands are compared with strains defined for each corresponding damage limit in order to define the component damage level.

Table 4.24 : Damage Levels of Beams under x Direction Seismic Effect

BEAM	HINGE	ε s	ε cu	ε cc	damage
1101	LEFT	0.0176	0.0019	0.0008	moderate
1101	RIGHT	0.0145	0.0016	0.0006	moderate
1102	LEFT	0.0111	0.0012	0.0005	moderate
1102	RIGHT	0.0050	0.0005	0.0002	slight
1103	LEFT	0.0032	0.0003	0.0001	slight
1103	RIGHT	0.0156	0.0017	0.0007	moderate
1104	LEFT	0.0139	0.0015	0.0006	moderate
1104	RIGHT	0.0207	0.0022	0.0009	moderate
1105	LEFT	0.0182	0.0021	0.0008	moderate
1105	RIGHT	0.0159	0.0018	0.0007	moderate
1106	LEFT	0.0133	0.0015	0.0006	moderate
1106	RIGHT	0.0128	0.0015	0.0006	moderate
1107	LEFT	0.0211	0.0024	0.0010	moderate
1107	RIGHT	0.0034	0.0004	0.0002	slight
1108	LEFT	0.0034	0.0004	0.0002	slight
1108	RIGHT	0.0223	0.0025	0.0010	moderate
1109	LEFT	0.0196	0.0022	0.0009	moderate
1109	RIGHT	0.0189	0.0022	0.0009	moderate
1110	LEFT	0.0157	0.0018	0.0007	moderate
1110	RIGHT	0.0223	0.0025	0.0010	moderate
1111	LEFT	0.0178	0.0019	0.0008	moderate
1111	RIGHT	0.0127	0.0014	0.0005	moderate
1112	LEFT	0.0109	0.0012	0.0005	moderate
1112	RIGHT	0.0208	0.0022	0.0009	moderate
1114	LEFT	0.0184	0.0020	0.0008	moderate
1114	RIGHT	0.0162	0.0017	0.0007	moderate
1115	LEFT	0.0141	0.0015	0.0006	moderate
1115	RIGHT	0.0208	0.0022	0.0009	moderate

Table 4.25 : Damage Levels of Beams under y Direction Seismic Effect

BEAM	HINGE	ε s	ε cu	ε cc	damage
1116	LEFT	0.0118	0.0013	0.0005	moderate
1116	RIGHT	0.0129	0.0014	0.0006	moderate
1117	LEFT	0.0118	0.0013	0.0005	moderate
1117	RIGHT	0.0165	0.0018	0.0007	moderate
1118	LEFT	0.0125	0.0013	0.0005	moderate
1118	RIGHT	0.0155	0.0017	0.0007	moderate
1119	LEFT	0.0113	0.0012	0.0005	moderate
1119	RIGHT	0.0170	0.0018	0.0007	moderate
1121	LEFT	0.0126	0.0014	0.0006	moderate
1121	RIGHT	0.0136	0.0015	0.0006	moderate
1122	LEFT	0.0095	0.0011	0.0004	slight
1122	RIGHT	0.0161	0.0018	0.0007	moderate
1124	LEFT	0.0122	0.0014	0.0006	moderate
1124	RIGHT	0.0135	0.0015	0.0006	moderate
1125	LEFT	0.0118	0.0013	0.0005	moderate
1125	RIGHT	0.0142	0.0016	0.0006	moderate
1126	LEFT	0.0101	0.0012	0.0005	moderate
1126	RIGHT	0.0161	0.0018	0.0007	moderate
1127	LEFT	0.0108	0.0012	0.0005	moderate
1127	RIGHT	0.0123	0.0014	0.0006	moderate
1128	LEFT	0.0105	0.0011	0.0005	moderate
1128	RIGHT	0.0147	0.0016	0.0006	moderate

Same procedure for calculation of curvature demands is also valid for columns. Axial force – strain relation is sketched for each column end cross-section and defined strain limits for each damage limit is also represented on the same graph (Figure 4.22). In guidance axial force and total curvature demands, the damage level of columns are obtained.

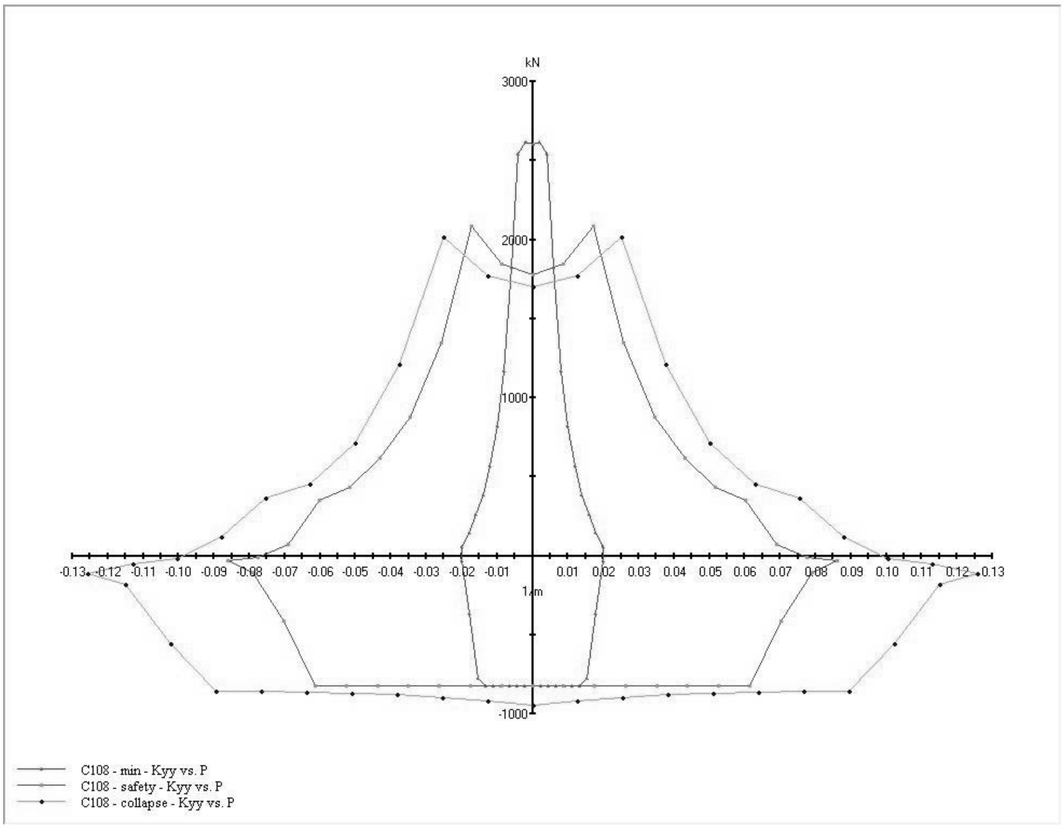

Figure 4.22 : Column 101 Axial Force – Curvature Graph with Damage Limits

Example for Column 101

Axial Force = 120.9 kN

Curvature = 0.085 1/m

Damage level: moderate

Table 4.26 : Damage Levels of Columns under x Direction Seismic Effects

COLUMN	HINGE	N [kN]	K total	Damage
101	BOTTOM	120.92	0.08484	moderate
101	TOP	120.92	0.01764	slight
102	BOTTOM	426.05	0.07244	moderate
102	TOP	426.05	0.018872	slight
103	BOTTOM	445.81	0.07844	moderate
103	TOP	445.81	0.04068	slight
104	BOTTOM	363.65	0.076664	moderate
104	TOP	363.65	0.01764	slight
105	BOTTOM	306.84	0.063832	moderate
105	TOP	306.84	0.01764	slight
106	BOTTOM	296.65	0.08164	moderate
106	TOP	296.65	0.01764	slight
107	BOTTOM	673.20	0.12964	heavy
107	TOP	673.20	0.03276	slight
108	BOTTOM	435.23	0.0346714	moderate
108	TOP	435.23	0.0063	slight
109	BOTTOM	409.91	0.0226714	moderate
109	TOP	409.91	0.0063	slight
110	BOTTOM	677.47	0.10484	collapse
110	TOP	677.47	0.01764	slight
111	BOTTOM	468.10	0.04764	slight
111	TOP	468.10	0.01764	slight
112	BOTTOM	151.51	0.08492	moderate
112	TOP	151.51	0.01764	slight
113	BOTTOM	379.61	0.0798	moderate
113	TOP	379.61	0.03284	slight
114	BOTTOM	310.24	0.06224	moderate
114	TOP	310.24	0.01764	slight
115	BOTTOM	119.87	0.08596	moderate
115	TOP	119.87	0.01764	slight
116	BOTTOM	410.05	0.07092	moderate
116	TOP	410.05	0.01764	slight
117	BOTTOM	326.03	0.062112	moderate
117	TOP	326.03	0.01764	slight

Table 4.27 : Damage Levels of Columns under y Direction Seismic Effects

COLUMN	HINGE	N [kN]	K total	Damage
101	BOTTOM	314.18	0.0134	slight
101	TOP	314.18	0.0098	slight
102	BOTTOM	500.76	0.0098	slight
102	TOP	500.76	0.0098	slight
103	BOTTOM	636.28	0.0098	slight
103	TOP	636.28	0.0098	slight
104	BOTTOM	504.30	0.0098	slight
104	TOP	504.30	0.0098	slight
105	BOTTOM	311.68	0.0134	slight
105	TOP	311.68	0.0098	slight
106	BOTTOM	392.93	0.0131	slight
106	TOP	392.93	0.0098	slight
107	BOTTOM	662.07	0.0195	slight
107	TOP	662.07	0.0098	slight
108	BOTTOM	498.29	0.0256	slight
108	TOP	498.29	0.0176	slight
109	BOTTOM	497.55	0.0255	slight
109	TOP	497.55	0.0176	slight
110	BOTTOM	671.57	0.0229	moderate
110	TOP	671.57	0.0098	slight
111	BOTTOM	390.77	0.0132	slight
111	TOP	390.77	0.0098	slight
112	BOTTOM	127.42	0.0173	slight
112	TOP	127.42	0.0098	slight
113	BOTTOM	283.95	0.0113	slight
113	TOP	283.95	0.0098	slight
114	BOTTOM	108.76	0.0173	slight
114	TOP	108.76	0.0098	slight
115	BOTTOM	104.14	0.0178	slight
115	TOP	104.14	0.0098	slight
116	BOTTOM	292.17	0.0116	slight
116	TOP	292.17	0.0098	slight
117	BOTTOM	127.52	0.0178	slight
117	TOP	127.52	0.0098	slight

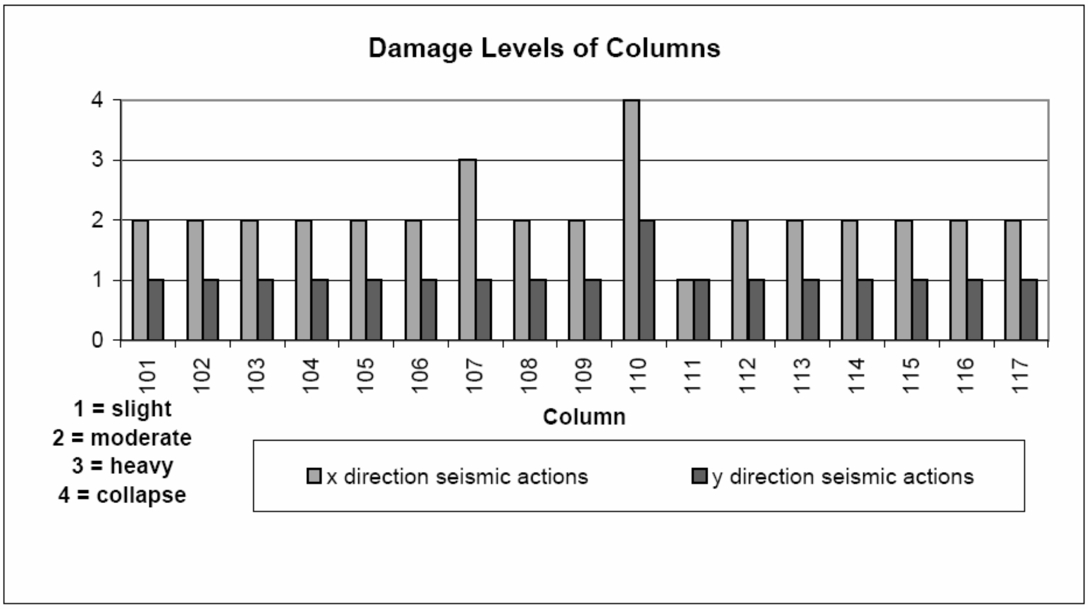

Figure 4.23 : Damage Levels of Columns

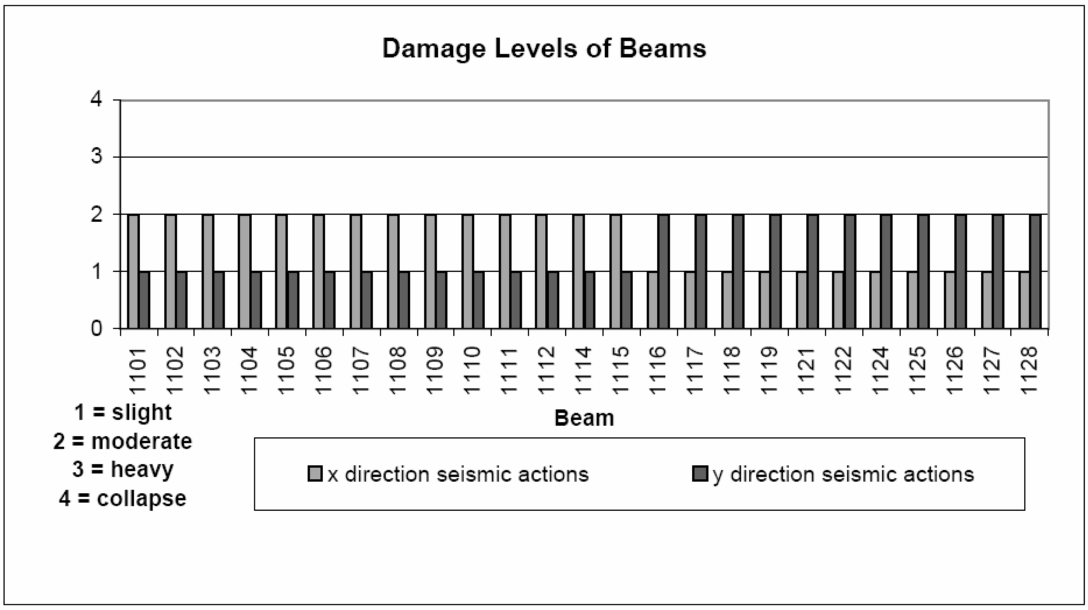

Figure 4.24 : Damage Levels of Beams

4.3 Assessment According to Eurocode 8

In this section, calculation steps for the case building according to *Lateral Load* (linear static analysis) and *Push-over* (non-linear static analysis) methods of analysis will be explained briefly on the basis of Eurocode 8.

Before investigating the seismic performance of the case building under different analysis types, on the following paragraphs, common seismic properties, which are valid both for linear static and non-linear static analysis procedures, are determined. Acceptance criteria for using the methods of analysis will also be checked.

Due to the comprehensiveness of obtained as-built information, knowledge level of the case study building is confirmed as "normal" and the corresponding confidence factor is 1.20 as given in Table 3.1. The confidence factor will be used for dividing mean values of material properties and also multiplying when determining force demand on brittle elements transferred by ductile ones.

Storey masses and building total masses considering the 30% participation of live loads are calculated and results are given in Table 4.28.

Table 4.28 : Storey Masses

storey	mass [kN.sec^2/m]
6	94.34
5	106.88
4	106.88
3	107.11
2	107.33
1	128.27
total	**650.81**

Eurocode recommends using elastic flexural and shear stiffness properties of concrete elements to be taken one-half of the corresponding stiffness of the uncracked elements unless a more accurate analysis of cracked elements is performed. The initial elastic stiffness values of beams and columns are multiplied with 0.50 in order to obtain the effective stiffness.

Fundamental periods for along x axis and y axis and rotation about z axis are obtained by SAP2000 [4] modal analysis after effective stiffness values are applied to the structural elements.

Tx	0.852 sec
Ty	0.794 sec
Tθ	0.689 sec

Lateral loads acting on each storey are determined on the following lines according to Lateral Load Method.

Ground Type : B

Soil Factor : 1.20

Lower limit of the period of the constant acceleration branch (T_B) : 0.15 sec

Upper limit of the period of the constant acceleration branch (T_C) : 0.50 sec

Beginning of the constant displacement response (T_D) : 2.00 sec

Damping correction factor: 1.00

Ordinate of the elastic spectrum at fundamental period $S_e(T_1)$

= peak ground acceleration x S x correction factor x 2.50 x (T_C / T_1)

$S_e(T_1)$ (x direction) = 0.30 x 9.81 x 1.20 x 1.00 x 2.50 x (0.50 / 0.852) = 5.18 m / sec^2

$S_e(T_1)$ (y direction) = 5.56 m / sec^2

Multiplying the ordinate value of the elastic spectrum with building mass gives the base shear. Multiplying also with correction factor 0.85 is also necessary since the building has more than two storeys and also has fundamental periods for both directions shorter than 1.00 sec.

Base shear for x direction = 2865.77 kN

Base shear for y direction = 3075.11. kN

For the distribution of horizontal forces along storeys, following formula is used:

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\Sigma s_j \cdot m_j} \quad \textbf{(4.7)}$$

where Fi is the horizontal force acting on storey i

F_b is the seismic base shear calculated above

m_i is the storey mass

s_i is the displacement of masses mi

Distributions of horizontal loads are given in Table 4.29.

Table 4.29 : Distribution of Horizontal Loads

STOREY	F_{ix} [kN]	F_{iy} [kN]
6	693.64	762.22
5	716.90	784.06
4	606.61	650.92
3	455.91	474.39
2	276.89	282.26
1	115.82	106.52
	2865.77	**3060.38**

The acceptance criteria for using Lateral Load method is depends on the fundamental period of the structure, irregularity condition on elevation and to the uniform distribution of elements which plastically deform.

The case study is regular in elevation because vertical elements and storey are continuous to the top, storey stiffness is also constant and there is no any existing setback.

The ratio of maximum to minimum value of demand-capacity ratio over all ductile members in a storey that go inelastic must not exceed the value of 2.50. Verification of this condition is shown in Table 4.30 and Table 4.31 separately for columns under seismic actions along x and y directions respectively.

Table 4.30 : Inelastic Behavior of Ductile Columns – X Direction

x - direction				
Column	Cap [kNm]	Demand [kNm]	DCR	DCR max
101	43	113.88	2.65	4.30
102	49	128.44	2.62	DCR min
103	47	129.03	2.75	2.29
104	48	128.34	2.67	DCR max / DCR min
105	46	116.78	2.54	1.88
106	48	109.95	2.29	
107	62	146.04	2.36	
108	225	968.52	4.30	
109	272	969.22	3.56	
110	62	239.98	3.87	
111	46	114.42	2.49	
112	43	114.53	2.66	
113	47	135.41	2.88	
114	48	114.54	2.39	
115	47	113.21	2.41	
116	47	134.21	2.86	
117	47	117.79	2.51	

Table 4.31 : Inelastic Behavior of Ductile Columns – Y Direction

y - direction				
Column	Cap [kNm]	Demand [kNm]	DCR	DCR max
101	105	210.57	2.01	3.03
102	114	217.74	1.91	DCR min
103	115	233.94	2.03	1.64
104	112	259.48	2.32	DCR max / DCR min
105	96	290.83	3.03	1.85
106	127	250.71	1.97	
107	192	314.21	1.64	
108	67	136.82	2.04	
109	66	143.15	2.17	
110	194	441.77	2.28	
111	128	344.05	2.69	
112	124	207.32	1.67	
113	127	212.85	1.68	
114	128	224.93	1.76	
115	127	233.74	1.84	
116	126	254.62	2.02	
117	124	288.13	2.32	

Since the overstrengths on columns are uniformly distributed, and there is not any existing irregularity in elevation, lateral force method is applicable for the investigated case study building.

4.3.1 Linear Analysis with Lateral Force Method

On previous section, minimum requirements for performing assessment using lateral force method are investigated and method's applicability is verified.

On following paragraphs, seismic performance assessment of the case study building will be performed with TEC'07 linear static analysis with *Lateral Force Method*.

Since the functionality purpose of case building is residence, on the basis of this thesis study, target seismic performance of the building will be Limit State of Significant Damage under seismic action with 475 years of return period. Therefore it will be convenient to compare seismic assessments using both TEC'07 and Eurocode 8 by using seismic actions with same return periods.

The chord rotation capacities of ductile primary elements corresponding to significant damage θ_{SD} is assumed to be 3/4 of the ultimate chord rotation θ_U .

For the calculation of ultimate chord rotation capacity of structural elements Equation (3.23) is used. Member-end rotation capacities for the critical storey vertical elements are given in Table 4.32 and Table 4.33.

Table 4.32 : Rotation Capacities of Column Ends – x Direction

x direction seismic effect							
column	θ yield [rad]	φ ult. [1/m]	φ yield [1/m]	L_{pl} [m]	L_V [m]	θ ult. [rad]	θ $_{SD}$[rad]
101	0.0017125	0.0451	0.0137	0.125	1.075	0.0036	0.0027
102	0.0015	0.037	0.012	0.125	1.075	0.0030	0.0022
103	0.001375	0.036	0.011	0.125	1.075	0.0029	0.0022
104	0.0015	0.037	0.012	0.125	1.075	0.0030	0.0022
105	0.0017125	0.0451	0.0137	0.125	1.075	0.0036	0.0027
106	0.0015	0.038	0.012	0.125	1.075	0.0030	0.0023
107	0.00125	0.031	0.01	0.125	1.075	0.0025	0.0019
108	0.00119	0.015	0.0034	0.35	1.075	0.0031	0.0023
109	0.00119	0.015	0.0034	0.35	1.075	0.0031	0.0023
110	0.00125	0.031	0.01	0.125	1.075	0.0025	0.0019
111	0.0015	0.038	0.012	0.125	1.075	0.0030	0.0023
112	0.001625	0.044	0.013	0.125	1.075	0.0035	0.0026
113	0.00145	0.0375	0.0116	0.125	1.075	0.0030	0.0022
114	0.0016875	0.044	0.0135	0.125	1.075	0.0035	0.0026
115	0.0016875	0.044	0.0135	0.125	1.075	0.0035	0.0026
116	0.00145	0.0375	0.0116	0.125	1.075	0.0030	0.0022
117	0.001625	0.044	0.013	0.125	1.075	0.0035	0.0026

Table 4.33 : Rotation Capacities of Column Ends – x Direction

y direction seismic effect							
column	θ yield [rad]	φ ult. [1/m]	φ yield [1/m]	L_{pl} [m]	L_V [m]	θ ult. [rad]	θ $_{SD}$[rad]
101	0.001395	0.029	0.0062	0.225	1.075	0.00399209	0.0030
102	0.0014625	0.02	0.0065	0.225	1.075	0.00278808	0.0021
103	0.001368	0.0195	0.00608	0.225	1.075	0.00271434	0.0020
104	0.0014625	0.02	0.0065	0.225	1.075	0.00278808	0.0021
105	0.001395	0.029	0.0062	0.225	1.075	0.00399209	0.0030
106	0.001395	0.021	0.0062	0.225	1.075	0.00291767	0.0022
107	0.0011	0.0155	0.0044	0.25	1.075	0.00236822	0.0018
108	0.00175	0.048	0.014	0.125	1.075	0.00383527	0.0029
109	0.00175	0.048	0.014	0.125	1.075	0.00383527	0.0029
110	0.0011	0.0155	0.0044	0.25	1.075	0.00236822	0.0018
111	0.001395	0.021	0.0062	0.225	1.075	0.00291767	0.0022
112	0.001395	0.029	0.0062	0.225	1.075	0.00399209	0.0030
113	0.0014625	0.02	0.0065	0.225	1.075	0.00278808	0.0021
114	0.00140175	0.0272	0.00623	0.225	1.075	0.00375082	0.0028
115	0.00140175	0.0272	0.00623	0.225	1.075	0.00375082	0.0028
116	0.0014625	0.02	0.0065	0.225	1.075	0.00278808	0.0021
117	0.001395	0.029	0.0062	0.225	1.075	0.00399209	0.0030

Bending moment demands are obtained from analysis with load combination of gravity and seismic loads. Yielding moment and yielding curvature and so the yielding rotation values for each column is calculated by cross-sectional analysis.

Demand rotations on member ends are determined on the basis of *Equal Displacement Rule*. Demand rotation, and the limit state of significant damage rotation capacities are compared in Table 4.34.

Table 4.34 : Demand Capacity Ratio of Columns

COLUMN	x direction			y direction		
	Ɵ SD	Ɵ DEMAND	DCR	Ɵ SD	Ɵ DEMAND	DCR
101	0.0027	0.0045	1.68	0.0030	0.0028	0.93
102	0.0022	0.0039	1.77	0.0021	0.0028	1.34
103	0.0022	0.0038	1.75	0.0020	0.0028	1.37
104	0.0022	0.0040	1.81	0.0021	0.0034	1.62
105	0.0027	0.0043	1.61	0.0030	0.0042	1.41
106	0.0023	0.0034	1.51	0.0022	0.0028	1.26
107	0.0019	0.0029	1.58	0.0018	0.0018	1.01
108	0.0023	0.0051	2.23	0.0029	0.0036	1.24
109	0.0023	0.0042	1.85	0.0029	0.0038	1.32
110	0.0019	0.0048	2.60	0.0018	0.0025	1.41
111	0.0023	0.0037	1.64	0.0022	0.0037	1.71
112	0.0026	0.0043	1.64	0.0030	0.0023	0.78
113	0.0022	0.0042	1.86	0.0021	0.0025	1.17
114	0.0026	0.0040	1.53	0.0028	0.0025	0.88
115	0.0026	0.0041	1.54	0.0028	0.0026	0.92
116	0.0022	0.0041	1.84	0.0021	0.0030	1.41
117	0.0026	0.0041	1.54	0.0030	0.0032	1.08

Rotation demand-capacity ratios in Table 4.34 show that few columns are compatible with limit state of significant damage when seismic actions are acting along y axis. Other columns fail for the acceptance criteria under the regarding limit state. Observing the ratio values also gives the idea that the vertical elements may also not be suitable for collapse prevention performance level.

4.3.2 Non-linear Static Analysis

At the very first step of non-linear static (pushover) analysis procedure, the structure is subjected to gravity loads, and then horizontal forces defined in Equation (4.8) are statically applied.

$$F_i = \alpha \cdot m_i \cdot \phi_i \qquad \textbf{(4.8)}$$

Uniform and modal patterns are defined for vertical distributions of the lateral loads. The uniform pattern is based on the lateral loads that are proportional to mass regardless of elevation.

$$\phi_i = 1$$

Table 4.35 : Vertical Distribution of Lateral Loads Using Uniform Pattern

storey	mass [kN.sec^2/m]	unit force [kN]
6	94.34	0.74
5	106.88	0.83
4	106.88	0.83
3	107.11	0.84
2	107.33	0.84
1	128.27	1.00

The modal pattern is applied proportional to lateral forces consistent with the lateral force distribution in the direction under consideration determined in elastic analysis.

Table 4.36 : Vertical Distribution of Lateral Loads Using Modal Pattern

STOREY	Fix [kN]	unit force x [kN]	Fiy [kN]	unit force y [kN]
6	693.64	0.97	762.22	0.97
5	716.90	1.00	784.06	1.00
4	606.61	0.85	650.92	0.83
3	455.91	0.64	474.39	0.61
2	276.89	0.39	282.26	0.36
1	115.82	0.16	106.52	0.14

The process is carried-out under monotonically increasing horizontal loads to investigate the displacement of control node relation. The control node is set to the center of mass of the top storey. Two different distributions of the lateral loads along the height of the building (uniform and inverted triangular) were used. The push-over curves which base shears are traced against control displacements are given in Figure 4.25 and Figure 4.26.

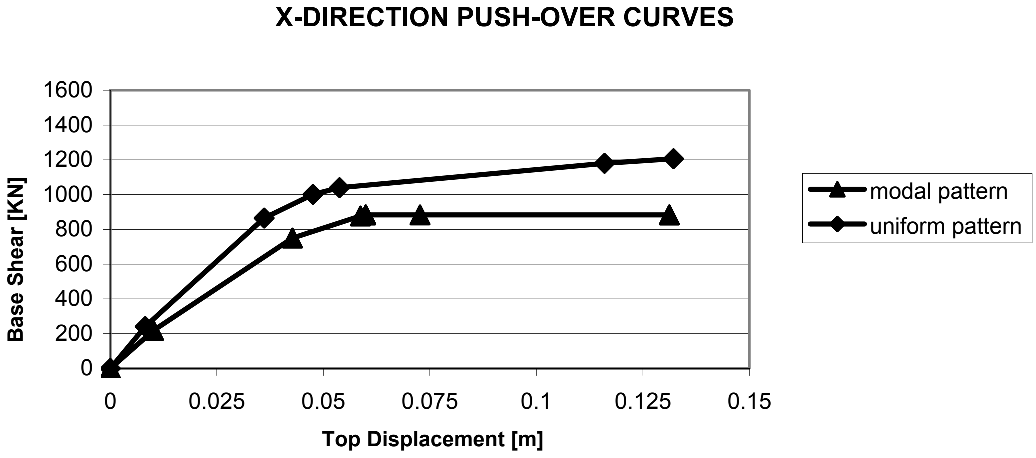

Figure 4.25 : X Direction Push-over Curves

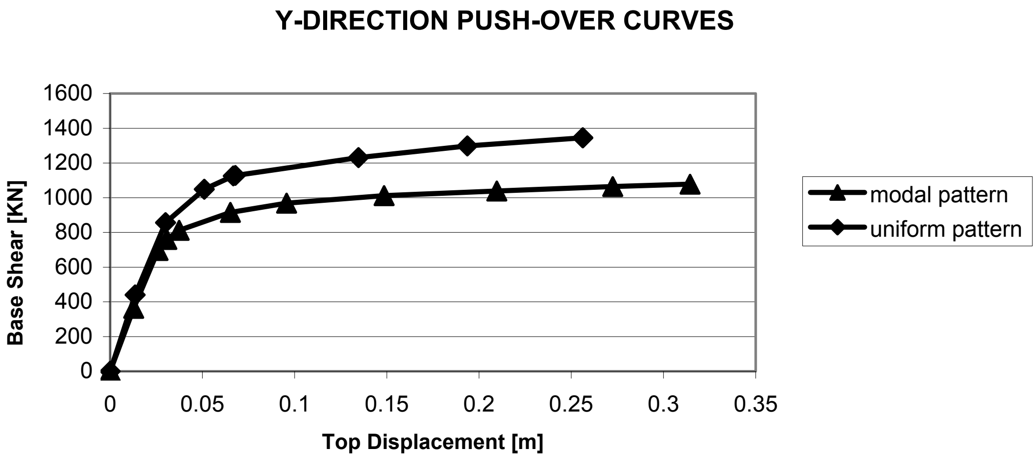

Figure 4.26 : Y Direction Push-over Curves

Previous studies on the topic states that uniform pattern for the lateral forces provides a good simulation of seismic response in bottom parts of structures, whereas a triangular distribution is better suited for top storey of buildings [32]. Though deformation energy capacities along both directions is lower while modal pattern, plastic rotations in first storey is higher when using the uniform distribution. Comparison of column top-end plastic rotations under triangular and uniform distribution of lateral loads is given in Table 4.37. Determination of seismic performance of the structure by using uniform distribution of lateral loads is more convenient since higher plastic deformations are obtained at first storey which is also critical storey of the structure.

Table 4.37 : Plastic Rotations at First Storey Columns

column	plastic rotations [rad]			
	x direction		y direction	
	modal pattern	uniform pattern	modal pattern	uniform pattern
101	0.0089	0.0125	0.0000	0.0058
102	0.0060	0.0099	0.0006	0.0058
103	0.0077	0.0106	0.0016	0.0053
104	0.0070	0.0103	0.0024	0.0057
105	0.0019	0.0054	0.0040	0.0066
106	0.0077	0.0117	0.0001	0.0072
107	0.0063	0.0099	0.0011	0.0064
108	0.0053	0.0133	0.0015	0.0057
109	0.0073	0.0113	0.0021	0.0058
110	0.0058	0.0098	0.0043	0.0087
111	0.0025	0.0060	0.0048	0.0083
112	0.0084	0.0125	0.0006	0.0083
113	0.0070	0.0105	0.0011	0.0084
114	0.0019	0.0060	0.0025	0.0087
115	0.0087	0.0128	0.0037	0.0089
116	0.0058	0.0100	0.0040	0.0089
117	0.0023	0.0069	0.0056	0.0154

The target displacement shall be defined as the seismic demand derived from elastic response spectrum in terms of the displacement of an equivalent single-degree-of-freedom (SDOF) system. Transformation to an equivalent single-degree-of-freedom system is explained below.

Coordinate conversion is applied for push-over curves to achieve modal displacement and modal acceleration values by using equations (2.2) and (2.3). Modal displacement and modal acceleration values are shown in Figure 4.27 and Figure 4.28 for two separate push-over curve.

Table 4.38 : Modal Acceleration and Modal Displacement Values

x direction		y direction	
a_1 x	d_1 x	a_1 y	d_1 y
0.000	0.000	0.000	0.000
0.479	0.006	0.890	0.010
1.720	0.028	1.734	0.023
1.989	0.037	2.121	0.039
2.066	0.042	2.278	0.051
2.346	0.090	2.285	0.051
2.398	0.102	2.492	0.102
-	-	2.628	0.147
-	-	2.724	0.194
-	-	2.724	0.194
mass	503.080	mass	493.969
Φ_{i1}	0.058	Φ_{i1}	0.059
Γ_{X1}	22.133	Γ_{y1}	22.412

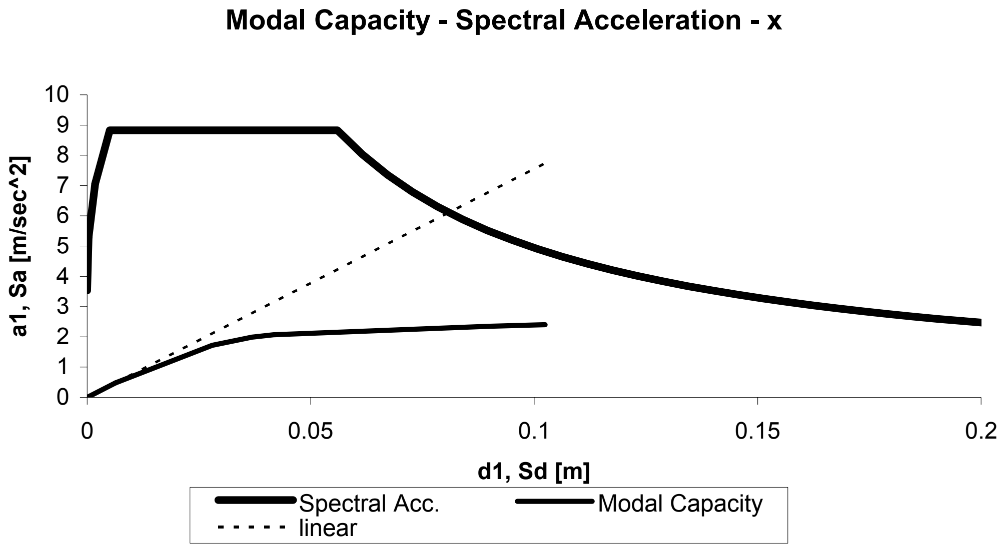

Figure 4.27 *:* Spectral Acceleration – Spectral Displacement Diagram (X Dir.)

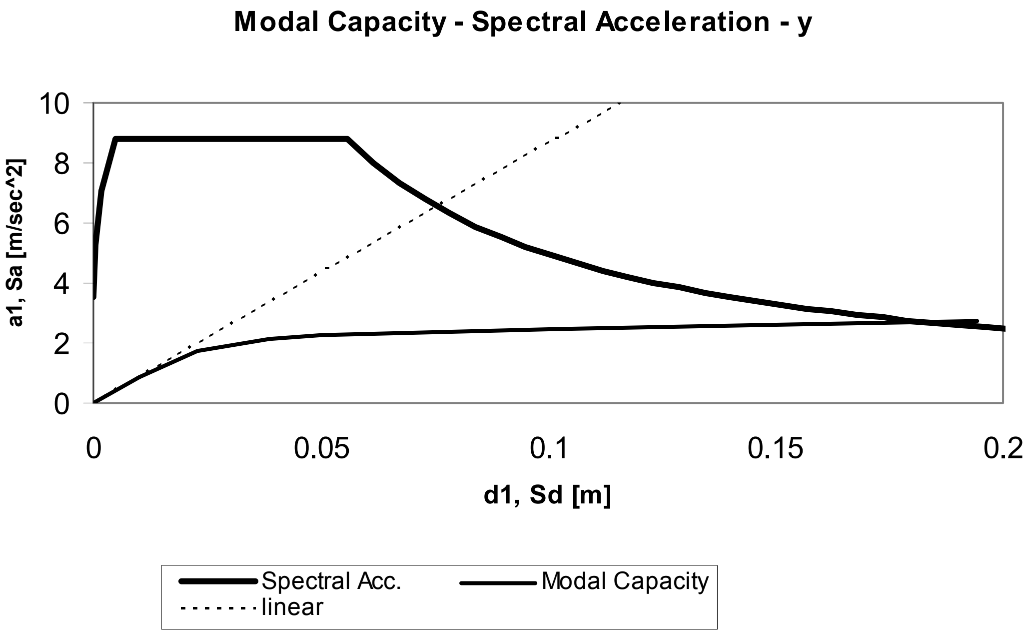

Figure 4.28 : Spectral Acceleration – Spectral Displacement Diagram (Y Dir.)

Since the initial periods in Figure 4.27 and Figure 4.28 are longer than the characteristic period T_C, non-linear spectral displacement is equal to the linear spectral displacement on the basis of *equal displacement rule*.

Modal displacement demand is equal to the non-linear spectral displacement value under scope of equations from (2.4) to (2.6).

Target displacements are calculated for two separate push-over analysis along x and y directions with equation (2.9) as 0.1083 m and 0.1034 m respectively.

Control node of the structure is pushed up to these target displacement and plastic rotation demands are investigated. Plastic hinges are shown on Figure 4.29 and Figure 4.30 as for x direction and y direction push-over analyses.

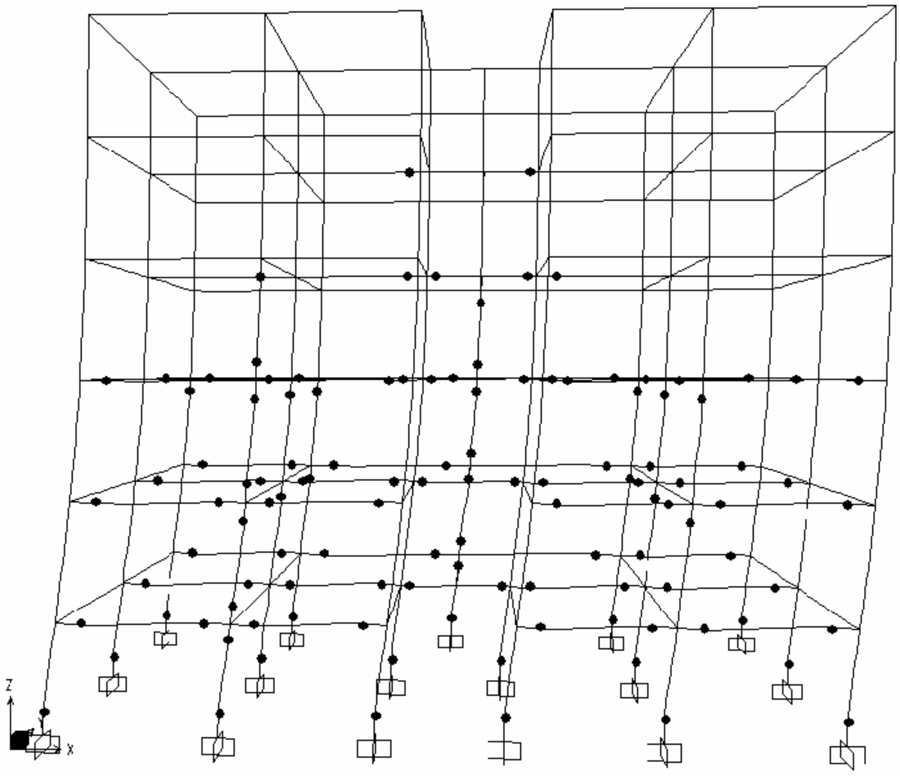

Figure 4.29 : Developed Plastic Hinges – X Direction Push-over Analysis

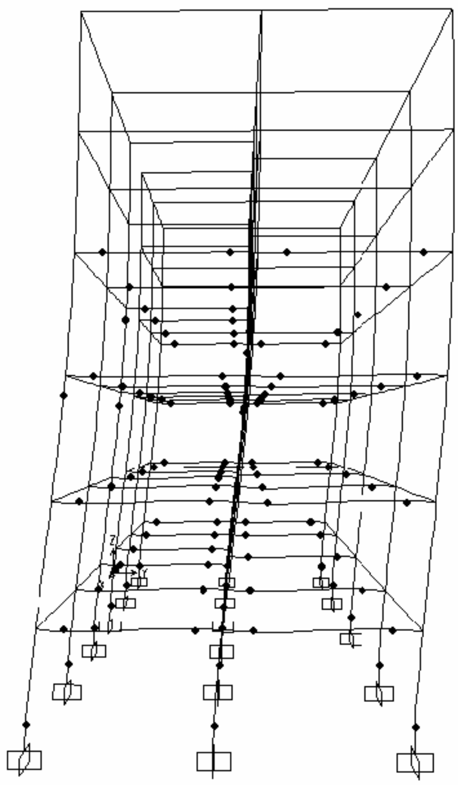

Figure 4.30 : Developed Plastic Hinges – Y Direction Push-over Analysis

Plastic rotation demands which are obtained from the analysis are summed with elastic rotations to determine total rotation values. Allowed rotation amounts for limit state of significant damage is compared with rotation demands for each column at first storey in Table 4.39 and Table 4.40.

Table 4.39 : Rotations at Columns under X Direction Push-over Analysis

column	elastic rot. [rad]	plastic rot. [rad]	total rot. [rad]	allowed (SD) [rad]
101	0.0017	0.0125	0.0142	0.0027
102	0.0015	0.0099	0.0114	0.0022
103	0.0014	0.0106	0.0120	0.0022
104	0.0015	0.0103	0.0118	0.0022
105	0.0017	0.0054	0.0071	0.0027
106	0.0015	0.0117	0.0132	0.0023
107	0.0013	0.0099	0.0112	0.0019
108	0.0012	0.0133	0.0145	0.0023
109	0.0012	0.0113	0.0125	0.0023
110	0.0013	0.0098	0.0111	0.0019
111	0.0015	0.0060	0.0075	0.0023
112	0.0016	0.0125	0.0141	0.0026
113	0.0015	0.0105	0.0120	0.0022
114	0.0017	0.0060	0.0077	0.0026
115	0.0017	0.0128	0.0145	0.0026
116	0.0015	0.0100	0.0115	0.0022
117	0.0016	0.0069	0.0085	0.0026

Table 4.40 : Rotations at Columns under Y Direction Push-over Analysis

column	elastic rot. [rad]	plastic rot. [rad]	total rot. [rad]	allowed (SD) [rad]
101	0.0014	0.0058	0.0072	0.0030
102	0.0015	0.0058	0.0073	0.0021
103	0.0014	0.0053	0.0067	0.0020
104	0.0015	0.0057	0.0072	0.0021
105	0.0014	0.0066	0.0080	0.0030
106	0.0014	0.0072	0.0086	0.0022
107	0.0011	0.0064	0.0075	0.0018
108	0.0018	0.0057	0.0075	0.0029
109	0.0018	0.0058	0.0076	0.0029
110	0.0011	0.0087	0.0098	0.0018
111	0.0014	0.0083	0.0097	0.0022
112	0.0014	0.0083	0.0097	0.0030
113	0.0015	0.0084	0.0099	0.0021
114	0.0014	0.0087	0.0101	0.0028
115	0.0014	0.0089	0.0103	0.0028
116	0.0015	0.0089	0.0104	0.0021
117	0.0014	0.0154	0.0168	0.0030

Rotation demand-capacity comparisons in Table 4.39 and Table 4.40 show that all columns fail for the acceptance criteria under the regarding limit state. Observing the values also gives the idea that the vertical elements may also not be suitable for collapse prevention performance level.

Allowed rotation amounts for limit state of significant damage is compared with rotation demands for each column at first storey in Table 4.39 and Table 4.40.

Table 4.41 : Rotations at Beams under X Direction Push-over Analysis

beam	elastic rot. [rad]	plastic rot. [rad]	total rot. [rad]	allowed (SD) [rad]
1101	0.0008	0.0126	0.0134	0.0116
1102	0.0008	0.0093	0.0101	0.0118
1103	0.0008	0.0116	0.0124	0.0118
1104	0.0008	0.0145	0.0153	0.0116
1105	0.0009	0.0125	0.0134	0.0104
1106	0.0009	0.0154	0.0163	0.0103
1107	0.0009	0.0143	0.0152	0.0094
1108	0.0009	0.0155	0.0164	0.0094
1109	0.0009	0.0140	0.0149	0.0103
1110	0.0009	0.0149	0.0158	0.0104
1111	0.0010	0.0129	0.0139	0.0116
1112	0.0008	0.0150	0.0158	0.0115
1114	0.0008	0.0136	0.0144	0.0115
1115	0.0008	0.0149	0.0157	0.0116

Table 4.42 : Rotations at Beams under Y Direction Push-over Analysis

beam	elastic rot. [rad]	plastic rot. [rad]	total rot. [rad]	allowed (SD) [rad]
1116	0.0008	0.0131	0.0139	0.0116
1117	0.0008	0.0142	0.0150	0.0117
1118	0.0009	0.0144	0.0153	0.0104
1119	0.0009	0.0145	0.0154	0.0105
1121	0.0009	0.0136	0.0145	0.0104
1122	0.0009	0.0149	0.0158	0.0105
1124	0.0009	0.0136	0.0145	0.0104
1125	0.0009	0.0143	0.0152	0.0104
1126	0.0009	0.0150	0.0159	0.0105
1127	0.0008	0.0136	0.0144	0.0116
1128	0.0008	0.0148	0.0156	0.0117

4.4 Comparison of Results Obtained from Static Linear and Non-linear Analysis According to TEC’07 and EC8

4.4.1 Comparison of Horizontal Elastic Response Spectrum Curves

The computed base shear value according to Eurocode is much higher than the Turkish Earthquake Code while the selected ground conditions represent the same characteristics (Figure 4.31). The main reason is that the ordinate of the horizontal elastic response spectrum for Eurocode 8 is increased by the soil factor.

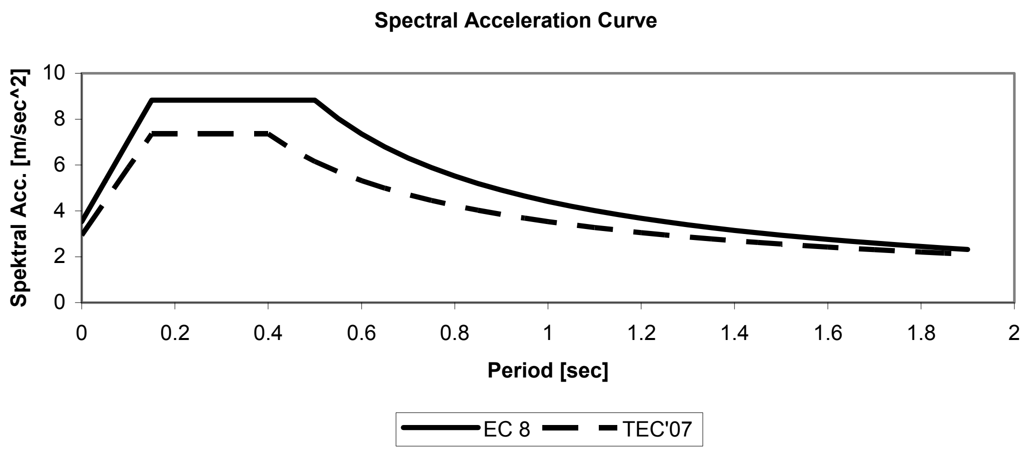

Figure 4.31 : Horizontal Elastic Response Spectrum Curves

4.4.2 Comparison of Push-over Curves

Push-over curves obtained under seismic loads in x and y directions according to Eurocode 8 and Turkish Earthquake Code are compared in Figure 4.32 and Figure 4.33.

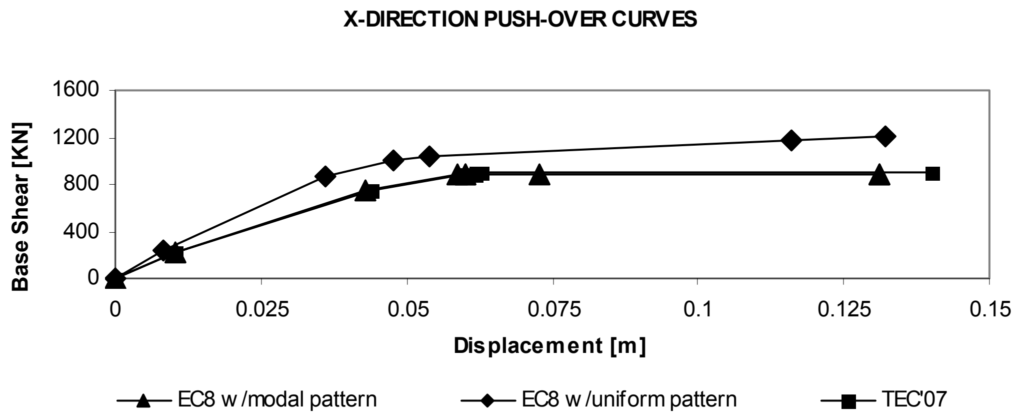

Figure 4.32 : X-Direction Push-over Curves

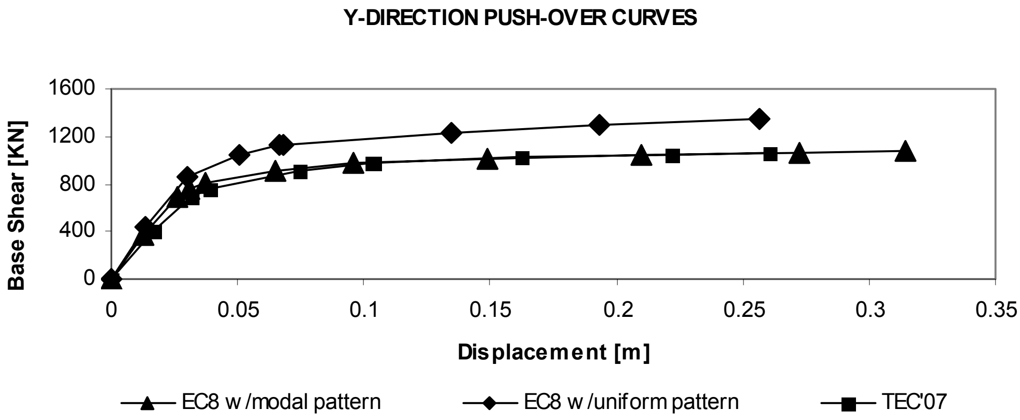

Figure 4.33 : Y-Direction Push-over Curves

4.4.3 Comparison of Curvatures

Demand curvatures obtained from linear and non-linear methods of analysis of Eurocode 8 together with TEC'07 is given in Figure 4.34. Curvatures from linear methods of analysis are determined on basis of equal displacement rule.

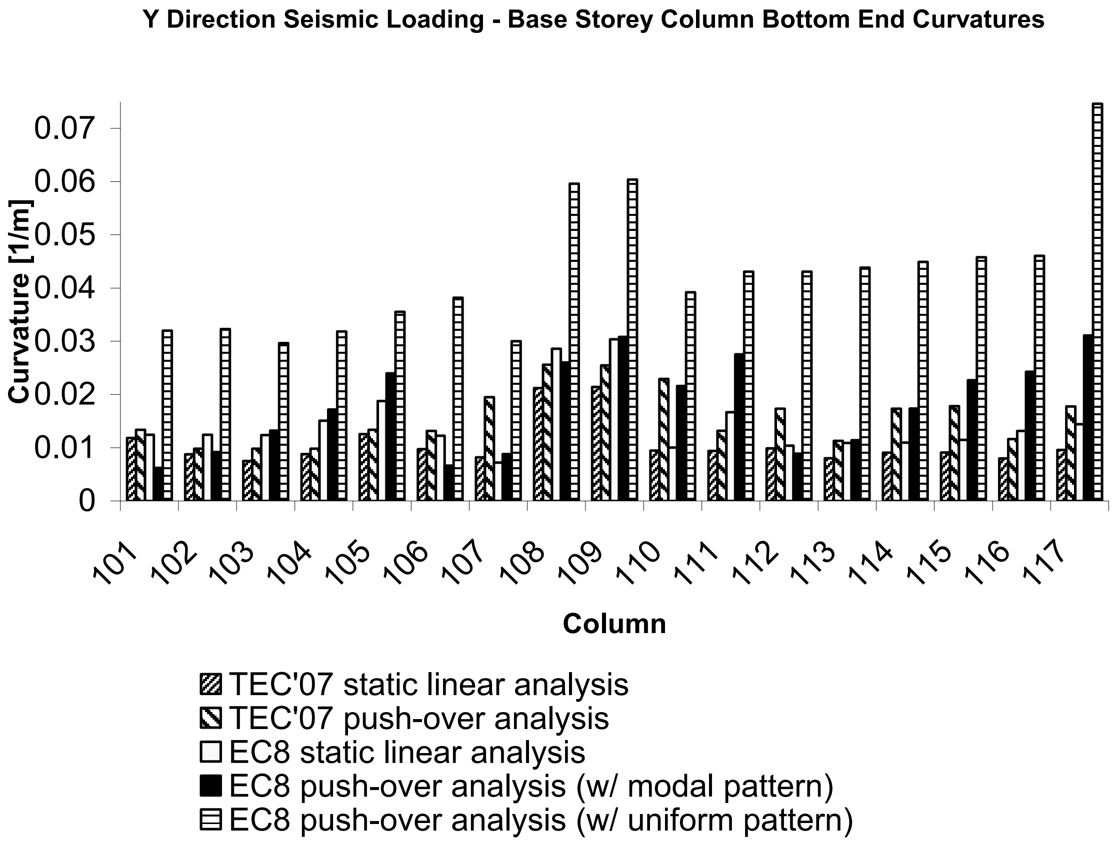

Figure 4.34 : Curvatures at Critical Storey Columns

4.4.4 Comparison of Top Displacements

Top storey displacements obtained from linear and non-linear methods of analysis of Eurocode 8 together with TEC'07 is given in Figure 4.35. Top storey displacements regarding to Eurocode 8 Push-over analysis is determined by using modal pattern of lateral load distribution.

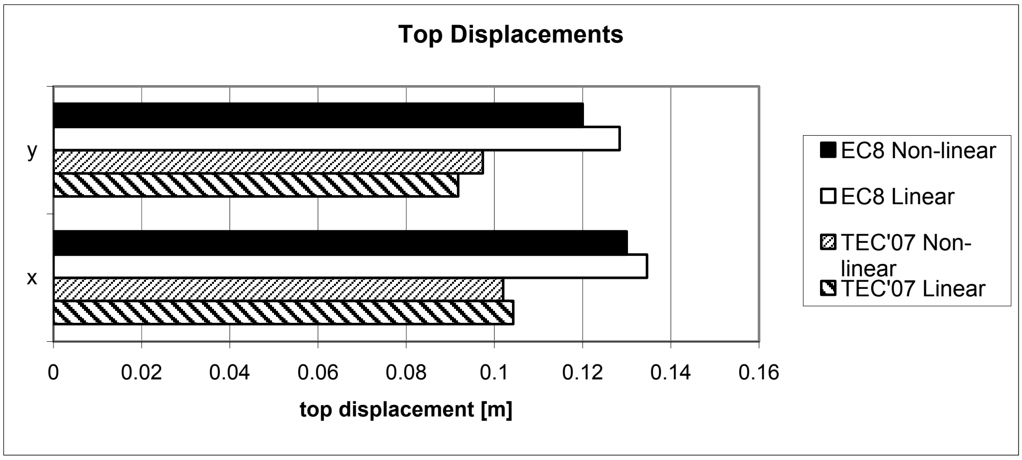

Figure 4.35: Top Displacements

5. CONCLUSION

In this thesis study, performance based assessment methods and basic principles given in TEC'07 and Eurocode 8 are investigated. After the linear elastic approach and non-linear approach are outlined as given in two codes, the procedures of seismic performance evaluations for existing RC buildings according to Eurocode 8 and TEC'07 are applied on a real three dimensional case study building and the results are compared.

Conclusion remarks regarding seismic performance assessments using static linear and static non-linear methods of analysis according to Eurocode 8 and TEC'07 are given below.

1) The computations show that the performing methods of analysis with linear and non-linear approaches using either Eurocode 8 or TEC'07 independently produce a very similar performance levels for the critical storey of the structure. The case study building is found to be as in collapse performance level.

2) The computed base shear value according to Eurocode is much higher than the Turkish Earthquake Code while the selected ground conditions represent the same characteristics. The main reason is that the ordinate of the horizontal elastic response spectrum for Eurocode 8 is increased by the soil factor.

3) According to the displacement-based non-linear assessment described in TEC'07, the strains at plastic cross-sections are to be verified; however, the chord rotations of primary ductile elements must be checked for Eurocode safety verifications.

4) The demand curvatures obtained from linear and non-linear methods of analysis are almost similar. Providing same levels of deformation in linear and non-linear analyses supports the *"Equal Displacement Rule"*. It is observed that rotations

with EC8 procedure are resulted with higher values than TEC’07 since the ordinate of the horizontal elastic response spectrum for EC8 is higher.

Investigated existing case study building within scope of this thesis study has no structural irregularities neither in elevation not in plan. Irregular structures can be examined with further studies to investigate essential differences between two codes if any.

Adaptive push-over analysis using different distribution of lateral loads at each step related to plastic deformations on the structure also using the participation of higher modes can be offered to observe the efficiencies of non-linear between each other.

REFERENCES

[1] **Fajfar, P. And Krawingler, H.,** 1997. *Seismic Design Methodologies for the Next Generation of Codes,* Proceedings of International Workshop held in Bled (Slovenia), Balkema, The Netherlands.

[2] **Priestley, M.J.N.,** 2000, *Performance Based Design, 12th World Conference on Earthquake Engineering*, Auckland, New Zeland, January-February 2000.

[3] **Özer, E.,** 2007. *Non-linear Analysis of Structural Systems Lecture Notes*, www.ins.itu.edu.tr/eozer.

[4] **SAP 2000**, 2006. Structural Analysis Program, Computers and Structures Inc., Berkeley, California.

[5] **XTRACT**, 2002, Cross-section Analysis Program, Imbsen Software Systems.

[6] **ATC-40**, 1996. Seismic *Evaluation and Retrofit of Concrete Buildings*, Applied Technology Council, California.

[7] **FEMA-273**, 1997. *NEHRP Guidelines for Seismic Rehabilitation of Buildings*, Federal Emergency Management Agency, Washington.

[8] **FEMA-356**, 2000. *Prestandard and Commentary for Seismic Rehabilitation of Buildings*, Federal Emergency Management Agency, Washington.

[9] **FEMA-440,** 2005. *Improvement of Nonlinear Static Seismic Analysis Procedures*, Federal Emergency Management Agency, Washington.

[10] **European Committee for Standardization**, 2004. *Design of Structures for Earthquake Resistance-Assessment and Retrofitting of Buildings*, Eurocode 8-3.

[11] **Fardis, M.N.**, 2003. *Seismic Assessment and Retrofitting of Existing Buildings according to Eurocode 8*, Fifth National Conference on Earthquake Engineering, Istanbul.

[12] **Priestley, M.J.N., Calvi, G.M., Kowalsky, M.J.**, 2007. *Displacement-Based Seismic Design of Structures*, IUSS Press, Pavia.

[13] **Ministry of Public Works,** *Turkish Seismic Design Code*, Official Gazette, March 2007.

[14] **Kuran, F., Demir, C., Koroglu, O., Kocaman, C., İlki, A.**, 2007. *Seismic Safety Analysis of an Existing 1502 Type Disaster Building Using New Version of Turkish Seismic Design Code*, ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering, Greece.

[15] **Çetin, C.**, 2007. *A Numerical Study on Seismic Performance Evaluation of Existing Reinforced Concrete Buildings by Non-linear Methods of 2007 Turkish Seismic Code*, Master Thesis, ITU Institute of Science and Technology, Istanbul.

[16] **Yılmaz, H.E.**, 2006. *A Comparative Numerical Study on Seismic Performance Evaluation of Existing Reinforced Concrete Buildings on FEMA 356 and 2006 Turkish Seismic Code Non-linear Analysis Approaches*, Master Thesis, ITU Institute of Science and Technology, Istanbul.

[17] **Bal, I.E., Tezcan, S. S., Gülay, F.G.**, 2007. *P25 Rapid Screening Method to Determine the Collapse Vulnerability of R/C Buildings*, Sixth National Conference on Earthquake Engineering, Istanbul.

[18] **Gülay, F.G., Bal, I.E., Gökçe, T.**, 2007. *Correlation Between Detailed and Preliminary Assessment Techniques in the Light of Real Damage States*, Journal of Earthquake Engineering (Accepted Paper).

[19] **Önem, G., Aydınoğlu**, M.N., 2007. *Evaluation of Incremental Response Spectrum Analysis*, ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering, Crete.

[20] **Kalkan, E., Kunnath, S.K.**, 2004. *Method of Modal Combinations for Push-over Analysis of Buildings*, 13th World Conference on Earthquake Engineering, Canada.

[21] **Falcao, S., Bento, R.**, 2002. *Analysis Procedures for Performance-Based Seismic Design*, 12th European Conference on Earthquake Engineering, London.

[22] **Özer, E., Pala, S., Orakdöğen, E., Girgin, K.**, 1999. *Deprem Bölgelerindeki Mevcut Betonarme Yapıların Deprem Güvenliklerinin Belirlenmesi ve Rehabilitasyonu*, Technical Report, TDV/TR 028-45.

[23] **Chopra, A.K., Goel, R.K.**, 2002. *A Modal Pushover Analysis for Estimating Seismic Demands for Buildings*, Earthquake Engineering and Structural Dynamics, **31(3)**, 561-582.

[24] **Aydınoğlu, M.N.**, *An Incremental Response Spectrum Analysis Based on Inelastic Spectral Displacements for Multi-mode Seismic Performance Evaluation*, Bulletin of Earthquake Engineering, **1(1)**, 3-36.

[25] **Aydınoğlu, M.N.**, 2004. *An Improved Pushover Procedure for Engineering Practice: Incremental Response Spectrum Analysis (IRSA), International Workshop on "Performance-based Seismic Design: Concepts and Implementation"*, Bled, Slovenia, PEER Report 2004/05: 345-356.

[26] **Carr, A.J.**, 2008. *Advanced Non-linear Seismic Analysis Lecture Notes*, ROSE SCHOOL Centre for Post-Graduate Training and Research in Earthquake Engineering & Engineering Seismology, Pavia, Italy.

[27] **Fardis, M. N.**, *Designer's Guide to EN 1998-1 and EN 1998-5*, MPG Books, 2005.

[28] **TS-500**, 2000. *Requirements for Design and Construction of Reinforced Concrete Structures*, Turkish Standards Institute, Ankara.

[29] **Özer, E., Pala, S.**, 1998. *PTT Evleri Block 18 Yaşar Apartment Retrofitting Report*, ITU Structural and Earthquake Research Center, Istanbul.

[30] **TS-498**, 1997. *Design Loads for Buildings*, Turkish Standards Institute, Ankara.

[31] **Mander, J.B., Priestley, M.J.N., Park, R.**, 1988. *Theoretical Stress-Strain Confined Concrete*, Journal of Structural Division (ASCE), **114(8)**, 1804-1826.

[32] **Fajfar, P.,** *Seismic Performance Analysis of Infilled Reinforced Concrete Frames, Sixth National Conference on Earthquake Engineering,* Istanbul, 2007.

CURRICULUM VITAE

Ahmet Emre Toprak was born in 1983. He had his elementary education at Lutfu Banat Elementary School and his secondary education at Beşiktaş Atatürk Anatolian High School in Istanbul.

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