QUANTITATIVE EVALUATION OF ASSESSMENT METHODS IN THE 2007 TURKISH EARTHQUAKE CODE

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ABSTRACT

QUANTITATIVE EVALUATION OF ASSESSMENT METHODS IN THE 2007 TURKISH EARTHQUAKE CODE

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Turkey is located on a seismically active region and ranks high among all countries that have suffered losses of life and property due to earthquakes over many centuries. In addition, its building stock has usually poor construction quality with seismically improper structural systems. These lead to a need for rapid and reliable assessment and retrofit procedures.

In the 2007 Turkish Earthquake Code, a new chapter is included for assessment and retrofit of existing buildings. The assessment procedures proposed in the Code are classified as linear elastic and nonlinear procedures. An engineer is allowed to choose one of these two procedures without any restriction.

In this study, a research was undertaken in order to clarify the differences between the seismic assessment procedures in the 2007 Turkish Earthquake Code. For this purpose, two pairs of existing and retrofitted residential buildings were assessed according to the principles of both procedures proposed in the 2007 Turkish Earthquake Code. The assessment results were also compared with the actual performance observations from a 5-storey building which suffered damage during the 1999 Düzce earthquake. In addition, an anchorage design methodology was developed for the exterior coupled shear wall retrofit solution, and tested on a 6-storey case study building.

Keywords: 2007 Turkish Earthquake Code, assessment, retrofit, nonlinear procedure, linear elastic procedure, global performance, acceptance criteria.

ÖZ

2007 DEPREM YÖNETMELİĞİNDE YER ALAN MEVCUT BİNA DEĞERLENDİRME YÖNTEMLERİNİN DEĞERLENDİRİLMESİ

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Türkiye sismik olarak aktif bir bölgede konumlanmış ve yüzyıllar boyunca deprem sebebiyle can ve mal kaybı yaşayan ülkeler arasında yüksek sıralarda yer almıştır. Buna ek olarak, ülkenin bina stoğu, kötü inşaat kalitesi ve deprem açısından yetersiz statik sistemlerden oluşmaktadır. Bunun sonucu olarak hızlı ve güvenilir değerlendirme ve güçlendirme yöntemlerininin ihtiyacı doğmuştur.

2007 Deprem Yönetmeliği mevcut binaların değerlendirilmesi ve güçlendirilmesi konulu yeni bir başlık içermektedir. Yönetmelikte yer alan değerlendirme yöntemleri doğrusal elastik ve doğrusal olmayan yöntemler olarak ayrılabilir. Mühendis bu iki yöntemden birini hiçbir sınırlama olmaksızın kullanabilir.

Bu çalışmada, 2007 Deprem Yönetmeliğinde yer alan mevcut bina değerlendirme ve güçlendirme yöntemlerinin aralarındaki farklılıkları irdelemek amaçlı bir araştırma gerçekleştirilmiştir. Bu amaçla, 2007 Deprem Yönetmeliğindeki yöntemler kullanılarak iki çift mevcut ve güçlendirilmiş konut binasının değerlendirmesi yapılmıştır. Değerlendirme sonuçları, beş katlı örnek binanın maruz kaldığı 1999 Düzce depremindeki gerçek hasar durumu ile karşılaştırılmıştır. Ek olarak, altı katlı örnek binanın dıştan perde ile güçlendirilmesi seçeneğinde ankraj hesabı yöntemi önerilmiştir.

Anahtar kelimeler: 2007 Türk Deprem Yönetmeliği, Doğrusal olmayan analiz yöntemleri, doğrusal elastik analiz yöntemleri, global performans, yöntem kabul değerlendirmesi

To my family

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CHAPTER I

INTRODUCTION

1.1 General

Turkey is on a seismically active region and most regions of the country are under risk of strong ground motion shaking. The recent earthquakes in Turkey have highlighted the structural inadequacy of the building stocks under earthquake. There are thousands of apartment buildings vulnerable to severe damage in a moderate or larger earthquake. Typically three to seven storey high, they consist of relatively poorly detailed and constructed reinforced concrete frame members infilled to various extents by unreinforced masonry walls (JICA, 2002). In many cases structural framing systems do not follow a rational layout from the perspective of resisting lateral loads. Commonly observed configuration problems include soft-stories, caused by a combination of increased ground floor interstorey heights, weak columns and strong beams, and masonry infill walls at first floor and upper stories. Additionally, improper construction techniques are one of the primary reasons of increased loss of life after an earthquake. In order to decrease the risk and repair damaged structures, assessment and retrofit guidelines became a necessity for Turkey. The 2007 Turkish Earthquake Code (TEC, 2007) is the fundamental document to fulfill the needs. It includes a chapter for assessment and retrofit of existing buildings.

New Turkish Earthquake Code offers two alternative procedures for the assessment of existing buildings. These are the linear elastic procedure and the nonlinear procedure. Code allows engineers to select one of these two procedures. Both analysis types have their own advantages and disadvantages. Therefore, an engineer can select the method by considering several factors such as building type, knowledge level, project duration…etc.

The primary decision is whether to choose the inelastic procedure over more conventional linear elastic analysis. In general, linear procedures are applicable when the structure is expected to remain nearly elastic for the level of ground motion of interest or when the design results in an almost uniform distribution of nonlinear response throughout the structure. In these cases, the level of uncertainty associated with linear procedures is relatively low. As the performance objective of the structure implies greater inelastic demands, the uncertainty with linear procedures increases to a point that requires a high level of conservatism in demand assumptions and/or acceptability criteria to avoid unintended performance. Inelastic procedures facilitate a better understanding of actual performance. This can lead to a design that focuses upon the critical aspects of the building, leading to more reliable and efficient solutions. However, the elastic analysis procedure described in the 2007 Turkish Earthquake Code is complemented with the capacity analysis. The main purpose of this linear analysis procedure is to obtain reliable results by combining linear analysis with capacity principles in order to determine seismic performance of buildings appropriately.

Since the Code allows an engineer to select one of the two different assessment procedures, the difference between the two procedures must be examined and understood clearly.

1.2 Review of Past Studies

There are several different procedures for performance assessment in the literature. Although they have variations, their logic for the assessment is the same: comparison of demands with capacity related limits. This logic is valid for both linear and nonlinear procedures. Some well-known and most widely used procedures leading to the development of seismic assessment regulations were summarized briefly in the following paragraphs.

Design of a building to the earthquake-induced lateral loading is a relatively new subject. Special regulations for buildings exposed to lateral loading are implemented to Uniform Building Code in 1930 although it was first published in 1927. In every three-year period, new regulations for earthquake engineering are added to the UBC. In the year 2000, Uniform Building Code was named as International Building Code (IBC, 2000) and it has been adopted by most of the states of USA and some other countries. The 1998 Turkish Earthquake Code also has similarities with UBC. All of the codes mentioned above are used for only design purposes with employing linear elastic analysis.

Towards the end of the last decade, researches have focused on performance-based engineering methods that rely on nonlinear static analysis procedures (NSPs). In 1996, ATC published the ATC-40 report, Seismic Evaluation and Retrofit of Concrete Buildings. In a larger project funded by the Federal Emergency Management Agency (FEMA), ATC prepared the FEMA 273 Guidelines for the Seismic Rehabilitation of Buildings, and the companion FEMA 274 Commentary, which were issued in 1997 by the Building Seismic Safety Council. FEMA 356 report, Prestandard and Commentary for the Seismic Rehabilitation of Buildings was published in 2000 as a successor to FEMA 273/274. Both FEMA 356 and ATC-40 are essentially the same when it comes to generating a “pushover” curve to represent the inelastic force-deformation behavior of a building. They differ, however, in the technique used to calculate the inelastic displacement demand for a given representation of ground motion. FEMA 356 (FEMA, 2000) uses a procedure known as the Coefficient Method, and ATC-40 (ATC, 1996) details the Capacity-Spectrum Method. Other variations and versions of these two procedures have been suggested, but all are related fundamentally to either displacement modification or equivalent linearization. Both approaches use nonlinear static analysis (pushover analysis) to estimate the lateral force-deformation characteristics of the structure. In both procedures, the global deformation (elastic and inelastic) demand on the structure is computed from the response of an equivalent single-degree-of-freedom system having the load-deformation properties determined from the pushover analysis. They differ, however, in the technique used to estimate the maximum deformation demand.

These procedures use estimates of ductility to calculate effective period and damping. The Coefficient Method is fundamentally a displacement modification procedure that is presented in FEMA 356. Displacement modification procedures estimate the total maximum displacement of the oscillator by multiplying the elastic response, assuming initial linear properties and damping, by one or more coefficients. The coefficients are typically derived empirically from series of nonlinear response history analyses of oscillators with varying periods and strengths. The process begins with an idealized force-deformation curve (pushover curve) relating base shear to roof displacement to generate an estimate of the maximum global displacement, which is termed the target displacement.

A form of equivalent linearization known as the Capacity-Spectrum Method, which is based primarily on the work of Freeman et al. (1975), is documented in ATC-40. The basic assumption in equivalent linearization techniques is that the maximum inelastic deformation of a nonlinear SDOF system can be approximated from the maximum deformation of a linear elastic SDOF system that has a period and a damping ratio that are larger than the initial values of those for the linear system. It was found that (ATC, 2005) the Capacity-Spectrum Method implemented in ATC-40 leads to very large overestimations of the maximum displacement for relatively short-period systems (periods smaller than about 0.5 s).

Another important procedure is described in Eurocode 8 Part 3 Strengthening and Repair of Buildings (EN 1998-3, 2005). In 1975, the Commission of the European Community decided on an action programme in the field of construction. The objective of the programme was the elimination of technical obstacles to trade and the harmonization of technical specifications. Eurocode 8, including the Strengthening and Repair of Buildings part, was prepared to serve this purpose.

Eurocode 8 elaborates; equivalent lateral force analysis, multi-modal response spectrum analysis, non-linear static analysis, and non-linear time history analysis.

Linear analysis procedures of Eurocode 8 are similar to those of FEMA 356 but acceptance limits are different. Capacities are modified according to the expected limit state, and demands are obtained from analysis of a linear system and from equilibrium conditions, considering the strength and ductility capacities of members.

Nonlinear analysis procedures of Eurocode 8 propose a displacement based assessment for ductile members, whereas a force based procedure is imposed for brittle members. The damage levels of all ductile members are assessed according to the chord rotation values that the member undergoes at the performance point of the building under the given ground motion. The chord rotation values obtained from the structural analysis are compared with the capacities for each limit state defined in the Eurocode 8. Different formulations are available for different limit states.

Damage states of Eurocode 8 correspond to the damage states of FEMA-356 and ATC-40 with different names. Damage Limitation (DL) of Eurocode 8 is Immediate Occupancy (IO) of FEMA-356 and ATC-40 whereas Significant Damage (SD) corresponds to Life Safety (LS). Near Collapse (NC) damage state is Collapse Prevention (CP) in FEMA-356 and ATC-40.

As it is described above, recent guidelines and earthquake codes prefer to include both linear and nonlinear procedures for assessment because it can not be said that one is superior to the other. Traditional design and analysis procedures that use linear elastic techniques can predict performance only implicitly because structural damage implies inelastic behavior. By contrast, the objective of inelastic seismic analysis procedures is to directly estimate the magnitude of inelastic deformations and distortions. The practical objective of inelastic seismic analysis procedures is to predict the expected behavior of the structure in a future earthquake. The history of nonlinear analysis procedures, used in these guidelines, belongs to 1960's. The studies of Jacobsen (1960) and Rosenblueth and Herrera (1964) are two of the examples.

Other important studies like Priestley (1997) and Park (1998) help the development of assessment and analysis procedures. Priestley suggested using mechanism considerations to calculate the displacement ductility and using realistic material strengths for determination of member strengths. Displacement demand is calculated by using the effective stiffness and equivalent viscous damping from the code displacement spectra. In the assessment procedure, substitute-structure approach of Shibata and Sozen (1976) is used.

Park (1998) stated the common deficiencies found in older frames and suggested a force-based procedure for seismic assessment of reinforced concrete structures. In this procedure, realistic material strength values are used to determine the member strengths and the type of inelastic mechanism. Shear failure checks are performed for members using the capacity principles. Finally, interstory drift is controlled. After determination of the inelastic mechanism, horizontal force capacity of the structure is calculated. Using the elastic period (based on cracked section stiffness) and horizontal force capacity, the global ductility demand is calculated. However ductility demands are based on curvature ductilities.

1.3 Objective and Scope

The majority of buildings in regions of high seismicity in Turkey do not meet current seismic code requirements, and many of these buildings are vulnerable to damage and collapse in an earthquake. Concerns for seismic rehabilitation of existing buildings have grown considerably following the 1971 San Fernando earthquake and resulted in several programs to identify and mitigate seismic risks around the world. Especially, after 1999 Kocaeli earthquake, studies in Turkey have been focused on the development of earthquake code in order to overcome retrofit design and assessment problems.

In this study, after presenting an overview of basic concepts of the new version of Turkish Earthquake Code, a case study was conducted with a residential building constructed in 1991 and located in Düzce. The investigated building was a reinforced concrete structure with 5-stories and the structural system was composed of shear walls with moment-resisting frames. The building experienced 12 November 1999 Düzce Earthquake and was slightly damaged. After the earthquake, it was retrofitted with additional shear walls and new columns. Both existing and retrofitted cases were assessed considering 2007 TEC and also by using the real damage observations after earthquake, global performance level was concluded. Linear and nonlinear analysis procedures were used for assessment. In addition to this, an existing 6-story residential building in Bakırköy İstanbul, which was built before 1975, was analyzed using the linear and nonlinear analysis procedures given in the 2007 version of the Turkish

Earthquake Code. As a retrofit technique, exterior coupled shear wall system was selected and retrofitted case was also assessed by using both the linear and nonlinear analysis procedures. Also, an anchorage design methodology was introduced for both analysis procedures.

The main objective of the study is to make a broad comparison between the linear and nonlinear assessment procedures of the 2007 Turkish Earthquake Code on two case study buildings before and after retrofitting and to evaluate these assessment methods with real damage observation.

This thesis is composed of seven main chapters and an appendix. Brief contents are given as follows:

Chapter 1	Statement of the problem and literature survey on analysis procedures and assessment methods.
Chapter 2	Description of the assessment procedures in 2007 Turkish Earthquake Code.
Chapter 3	Seismic Assessment of an existing residential building with linear and nonlinear procedures with reference to actual damage observations.
Chapter 4	Retrofitting of the building defined in Chapter 3 with shear walls and assessment of the retrofitted system with linear and nonlinear procedures.
Chapter 5	Seismic assessment of an existing residential building with linear and nonlinear procedures.
Chapter 6	Retrofitting of the building defined in Chapter 5 with exterior coupled shear walls and assessment of the retrofitted system with linear and nonlinear Procedures.
Chapter 7	A brief summary, discussions and conclusions.

CHAPTER II

DESCRIPTION OF ASSESSMENT PROCEDURES IN THE 2007 TURKISH EARTHQUAKE CODE

Seismic engineering is one of the most rapidly evolving disciplines in the civil/structural engineering profession. Recent seismic events in Turkey and around the world have provided new insight into the way structures perform when subjected to earthquake related ground motion. However, many structural engineers have limited experience concerning the dynamic behavior of structures subjected to strong ground motion. This makes seismic design codes key references for all. Assessment procedures of the 2007 Turkish Earthquake Code are explained briefly in following topics.

2.1 Linear Elastic Assessment Procedures

Linear assessment procedure basically has three steps; analysis, capacity calculation, performance evaluation. Each step is explained in the following topics. A brief step-by-step description of the linear assessment procedure of 2007 Turkish Earthquake Code is presented below.

1-) 3-D model of the building is prepared.

2-) Beam moment and shear capacities are calculated.

3-) Capacity related column axial loads are calculated.

4-) Member failure mode is defined (ductile or brittle).

5-) Demand to capacity ratio “r” is defined for ductile members from moment and for brittle members from shear.

6-) By comparing r values with r_{Limit} values, member acceptance is decided.

7-) Building global performance level is defined based on the member acceptance.

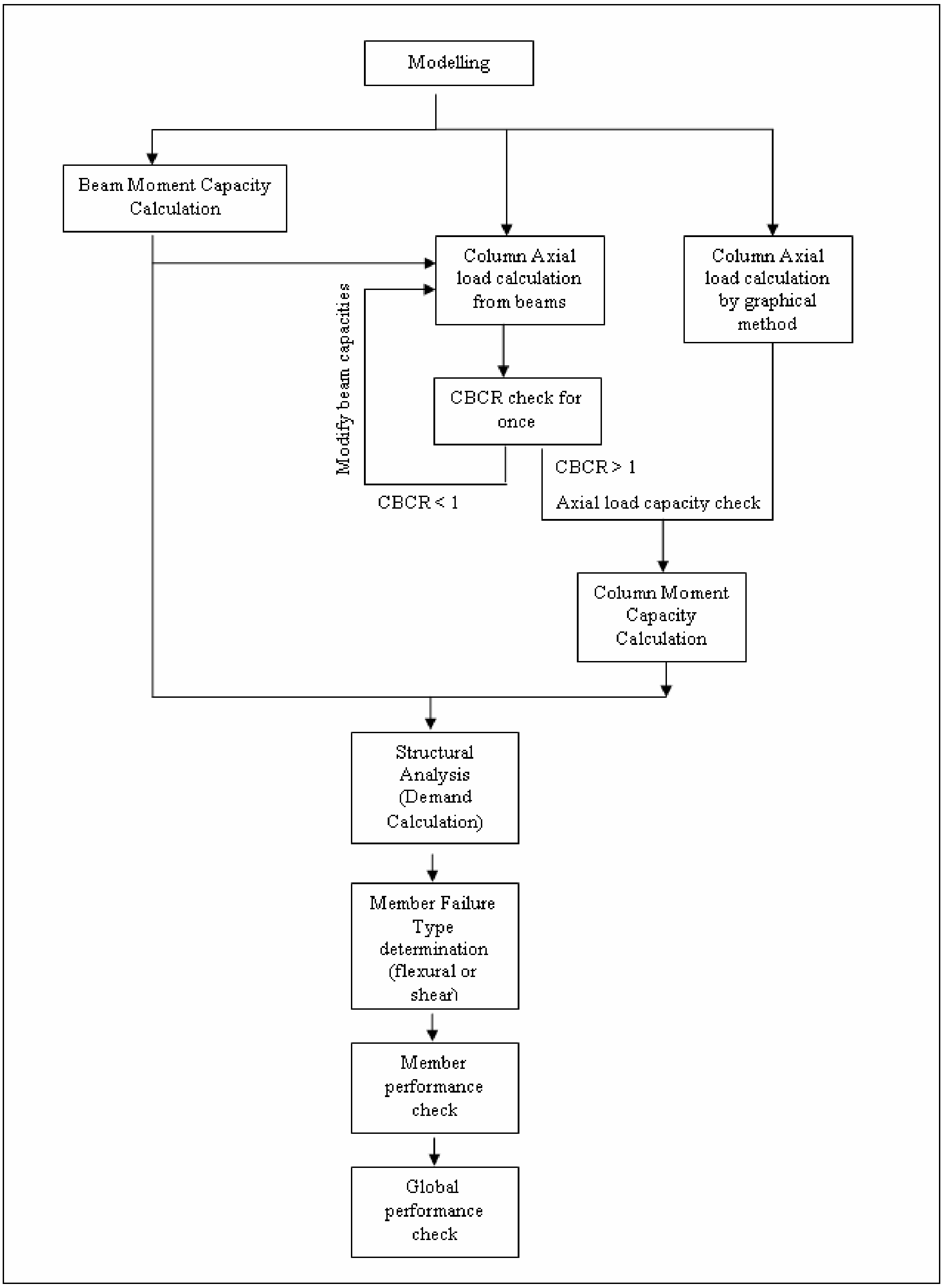


Figure 2.1 : Linear assessment procedure

At the beginning of the assessment, the project performance level must be selected. Performance level will determine limit values for section and system. Moreover Code spectrum is modified according to the selected performance level. Usage and type of the structure define the performance level that it should satisfy. Required performance levels for different structure types are given in Table 2.1.

Table 2.1 : Required seismic performance levels for design earthquakes

Building usage and type	Probability of exceedance of the earthquake		
	50% in 50 years	10% in 50 years	2% in 50 years
Buildings to be utilized after the earthquake	-	IO	LS
Intensively and long-term occupied buildings	-	IO	LS
Intensively and short-term occupied buildings	IO	LS	-
Buildings containing hazardous materials	-	IO	CP
Other buildings	-	LS	-

Where;

IO : Immediate Occupancy

LS : Life Safety

CP : Collapse prevention

As the probability of occurrence decreases, the magnitude of earthquake increases. Thus, an earthquake having the probability of exceedance of 2% in 50 year period stands for CP whereas an earthquake having the probability of exceedance of 50% in 50 year period stands for IO for a building. It is uneconomical to design or repair a structure for an earthquake having the probability of exceedance of 2% in 50 year period with a performance level of IO.

By using the spectrum for the probability of exceedance of 10% in 50 year period as a base, modification for different probabilities can be made as; taking the half of spectral ordinates for obtaining the probability of exceedance of 50% in 50 year period, or multiplying with 1.5 for obtaining the probability of exceedance of 2% in 50 year period. Code spectrum for different performance levels is shown in Figure 2.2.

2.1.1 Modeling for Linear Elastic Analysis

In Chapter 7 of the 2007 Turkish Earthquake Code, there are two alternative assessment procedures, one is linear elastic and the other one is nonlinear procedure. Both procedures have their own modeling criteria with some similarities.

While using linear elastic methodology, the code requires that rigid-end zones are correctly defined and assigned to the members. Rigid end zones have the effect on stiffness of the members, thus on the natural period of the system. Another influence is on the member demand. For instance, in a symmetric system two identical members with one symmetric to other are expected to have close demands. However if rigid end zones are not assigned correctly, one member can be unacceptable while the other one is acceptable according to the acceptance limits.

Next important point of modeling is not to use mass eccentricity. For the assessment projects, as-built information is available. Thus, load and mass configuration is known and there is no need to use accidental eccentricity in the analysis.

Last issue is to use cracked section for all members. In the 2007 Turkish Earthquake Code cracked section coefficients are defined separately for beams, columns and shear walls. Column and shear wall section modifier depends on axial load on the member while beams have one coefficient for all.

The cracked section stiffness of beams are taken as 40% of uncracked section stiffness and cracked section stiffness of columns and shear walls are calculated according to their axial load level:

Beams	: 0.4 EI		
Column and Shear Wall	: 0.4 EI	if	$N_D/(A_c*f_{cm}) \leq 0.1$
	: 0.8 EI	if	$N_D/(A_c*f_{cm}) \geq 0.4$

where;

N_D : Axial load under gravity loading

A_c : Gross section area

f_{cm} : Existing concrete compressive strength

E : Modulus of elasticity

I : Uncracked moment of inertia

For the values of $N_D/(A_c*f_{cm})$ between 0.1 and 0.4, interpolation is needed to calculate cracked stiffness.

For the seismic load calculations both building importance factor (I) and strength reduction factor (R) are taken as 1. Member demands under the load combination of 1G+nQ (n depends on the building type) and earthquake are needed for further steps.

2.1.1.1 Equivalent Static Analysis

Equivalent lateral load distribution can be applied to the structures if the total height above ground level is less than 25m and torsional irregularity factor η_{bi} is less than 1.4. Number of floors except basement must be less than or equal to 8 in order to use Equivalent Static Analysis.

Total horizontal force acting on the system is calculated by Equation 2.1, then the right side of the equation is multiplied with λ which is 1 for structures with one or two floors and 0.85 for others.

$$V_t = \frac{WA(T_1)}{R(T_1)} \geq 0.1\, A_0\, IW \tag{2.1}$$

Here, W represents the total weight of the building calculated with the combination of dead loads and live loads multiplied by the “live load participation factor” which is defined according to the type of the structure. Both importance factor (I) and seismic load reduction factor (R) are equal to 1. A_0 is effective ground acceleration coefficient and varies between 0.4 and 0.1 depending on seismic zone that the structure is located in. $A(T_1)$ is defined by Equation 2.2:

$$A(T) = A_0\, I\, S(T) \tag{2.2}$$

S(T) is the Spectrum Coefficient depending on the site conditions and building natural period.

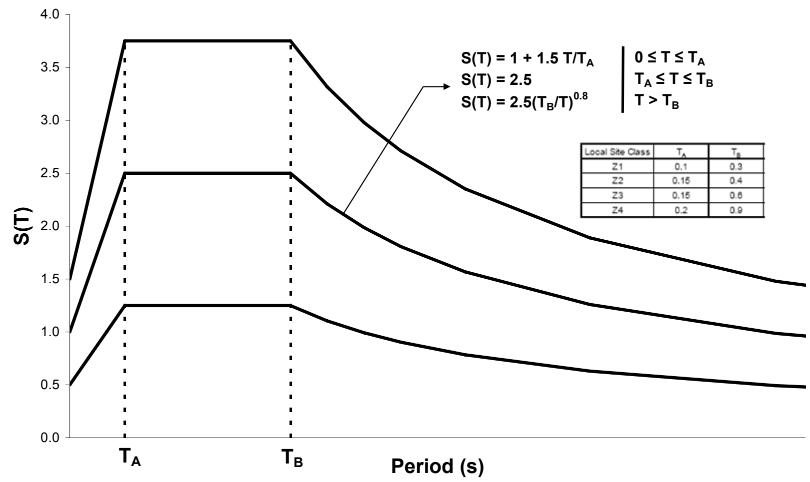


Figure 2.2 : Code spectra for different performance levels

Total seismic load is distributed along the stories according to the following equations;

$$V_t = \Delta F_N + \sum_{i=1}^{N} F_i \tag{2.3}$$

$$\Delta F_N = 0.0075 N V_t \tag{2.4}$$

$$F_i = (V_t - \Delta F_N) \frac{w_i H_i}{\sum_{j=1}^{N} w_j H_j} \tag{2.5}$$

2.1.1.2 Mode Superposition

In this method, maximum internal forces and displacements are determined by the combination of the maximum contributions obtained from each of the sufficient number of natural vibration modes considered. Sufficient number of vibration modes, Y, accounts for the sum of effective participating masses calculated for each mode in each direction, shall in no case be less than 90% of the total mass (TEC, 2007).

$$\sum_{n=1}^{Y} M_{xn} = \sum_{n=1}^{Y} \frac{\{\sum_{i=1}^{N} (m_i \phi_{xin})\}^2}{M_n} \geq 0.9 \sum_{i=1}^{N} m_i \tag{2.6}$$

$$\sum_{n=1}^{Y} M_{yn} = \sum_{n=1}^{Y} \frac{\{\sum_{i=1}^{N} (m_i \phi_{yin})\}^2}{M_n} \geq 0.9 \sum_{i=1}^{N} m_i \tag{2.7}$$

where:

$$M_n = \sum_{i=1}^{N} (m_i \phi_{xin}^2 + m_i \phi_{yin}^2 + m_{\theta i} \phi_{\theta in}^2) \tag{2.8}$$

Maximum contributions of response quantities calculated for each vibration mode, such as the base shear, storey shear, internal force components, displacements and storey drifts, are combined with either Square Root of Sum of Squares (SRSS) or Complete Quadratic Combination (CQC). In the case where any two natural vibration modes $T_m < T_n$ if $T_m / T_n > 0.8$ then SRSS is inapplicable.

2.1.2 Member Capacities

In order to decide if the member can resist to earthquake load, capacities must be known. Ductile and brittle members will have different limits and member performance definitions but shear check is needed for evaluating a member as brittle or ductile.

2.1.2.1 Shear Capacities

Shear capacities of the members are calculated by the rules of TEC (2007) and TS500 (2000).

Columns:

$$V_r = 0.52 f_{ctd} b_w d (1 + \gamma \frac{N_D}{A_c}) + \frac{A_{sw}}{s} f_{ywd} d \tag{2.9}$$

Beams:

$$V_r = 0.52 f_{ctd} b_w d + \frac{A_{sw}}{s} f_{ywd} d \tag{2.10}$$

Shear Walls:

$$V_r = A_{ch} (0.65 f_{ctd} + \rho_{sh} f_{ywd}) \tag{2.11}$$

2.1.2.2 Moment Capacities

2007 Turkish Earthquake Code has a new approach on column capacities; hence, they are dependent of axial load on the member in addition to section properties. By using the "Capacity Control Method" (Günay, 2003), axial loads under earthquake loading can be predicted more accurately. On the other hand, for this purpose beam capacities must be calculated first, in order to transfer shear from beams to columns as axial load.

Beam end moment capacities are calculated by using the existing concrete and reinforcement strength and section properties. Any available material model can be used such as the one suggested in Appendix 7B of the 2007 Turkish Earthquake Code (Mander et al., 1988). For both ends, top and bottom moment capacities are calculated separately. Considering the direction of moment induced by earthquake loading it is decided which moment capacity is of concern.

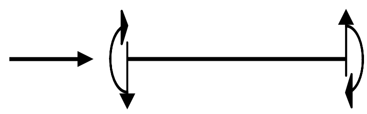


Figure 2.3 : Sign convention for beams

The sign convention shown in Figure 2.3 indicates that a positive moment induces tension in bottom reinforcement at the left and tension in top reinforcement at the right. For negative moment, it is vice versa. Residual moment (ΔM_K) is calculated by subtracting 1G + nQ demands (M_D) vectorially from moment capacities (M_K).

$$\Delta M_K = M_K - M_D \qquad (2.12)$$

Residual moments are preferred because during the 1G + nQ loading, gravity loads of beams are distributed to the columns. Vectorial subtraction of M_D has the ease to consider lateral loading condition and remaining capacity of the member.

Using the residual moment, capacity related shear demands are calculated by dividing the sum to the clear length (L_n) as shown in Figure 2.4.

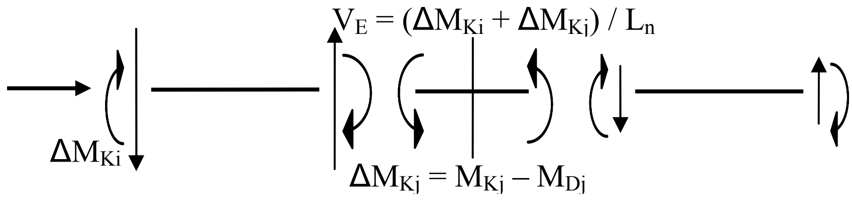


Figure 2.4 : Beam end shear calculation

V_E values are maximum axial loads that can be transferred to the columns under earthquake. Thus, this makes the method, capacity controlled. However these values must be checked with V_E demand of analysis. If V_E from analysis is larger than V_E calculated from the capacities, capacity related V_E is valid. If V_E from analysis is smaller than the V_E calculated from capacities, then V_E is obtained from analysis. Latter is the expected case for especially upper floors. V_E are added vectorially to the adjoining columns in order to calculate the axial forces due to earthquake loading (N_E).

Total axial load of columns (N_K) are the summation of axial load resultant of 1G + nQ loading (N_D) and loads transferred from all beams above and joining to that column (N_E). This process is shown schematically in Figure 2.5.

$$N_K = N_D + N_E \tag{2.13}$$

From these N_K values, the moment capacities (M_K) of column ends are estimated from the interaction diagrams, as shown in Figure 2.6.

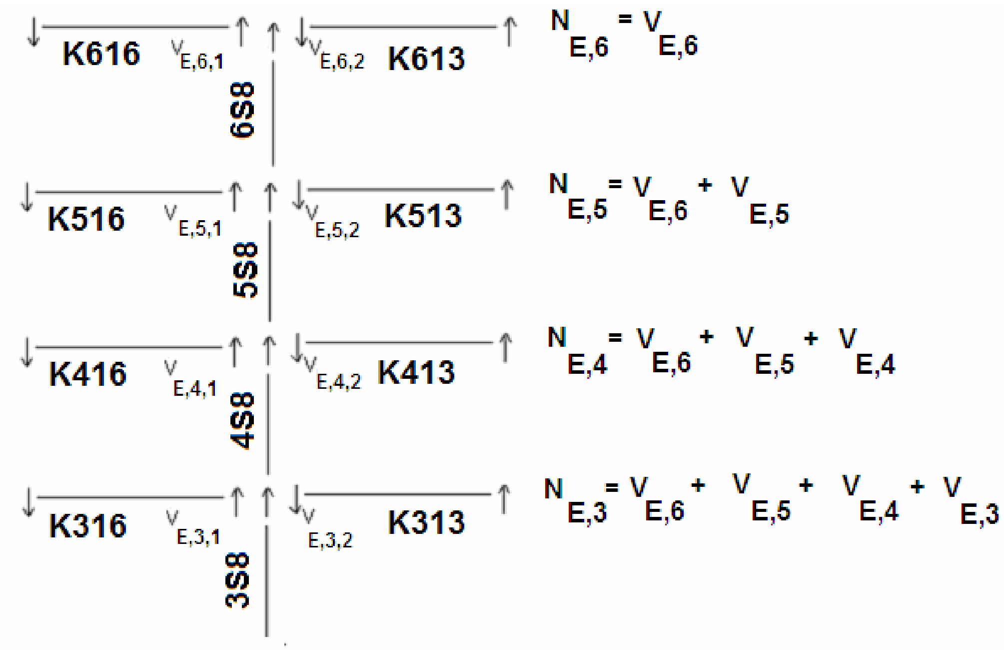


Figure 2.5 : Column axial load transferred from beams

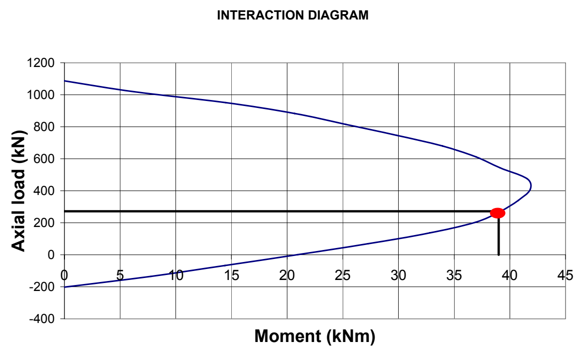


Figure 2.6 : Moment capacity of a column for a given axial load

Steps until this point are based on the assumption that all beams are weaker than columns, and hence they yield under earthquake loading. However, this assumption may not always hold. In order to check this, the column to beam capacity ratios (CBCR) are calculated from the M_K values of beam ends and column ends at every beam-column joints as follows:

$$\mathrm{CBCR} = \frac{\text{Total column moment capacities at joint } (M_{K\,bot} + M_{K\,top})}{\text{Total beam moment capacities at joint } (M_{Kj} + M_{Ki})} \qquad (2.14)$$

The symbols shown in Equation 2.14 are shown in Figure 2.7.

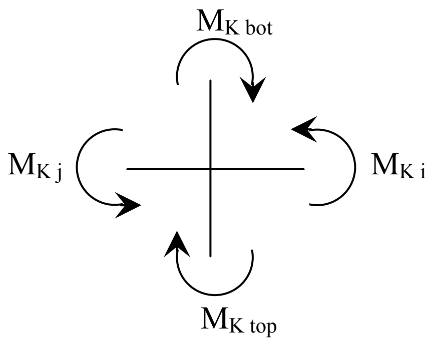


Figure 2.7 : Capacities of members at a joint

There are two possibilities for CBCR.

1. If CBCR > 1 then the assumption is valid, so the beams at that joint may yield. Adjoining member capacities can be accepted as they are calculated.

2. If CBCR < 1 then the assumption is wrong. Columns connected to that joint may yield before the beams. This situation requires a modification for the beam capacities that will affect column total axial load. M_{Kj} and M_{Ki} are multiplied by the CBCR values in order to modify the beam end capacities.

By using the new beam capacities, column axial load and moment capacity calculation steps are repeated. This iteration is done for once. The corresponding moment capacities of the columns are calculated by using the final N_K values. Modified beam capacities are only for the column N_K calculations, original capacities will be used for other purposes.

N_K sets the limit for axial load that can be taken by the member during earthquake. Code requires a comparative process for axial load calculation. A graphical method is described in Appendix 7A of the 2007 Turkish Earthquake Code. According to this method, axial load of a member under earthquake is the intersection of the line between gravity loading value and elastic analysis response of the member with interaction diagram. Procedure is illustrated in Figure 2.8. The smaller axial load obtained from the two analysis is used for the calculation of moment capacity of a column. Axial load calculated by the graphical method can not exceed the capacity controlled axial load.

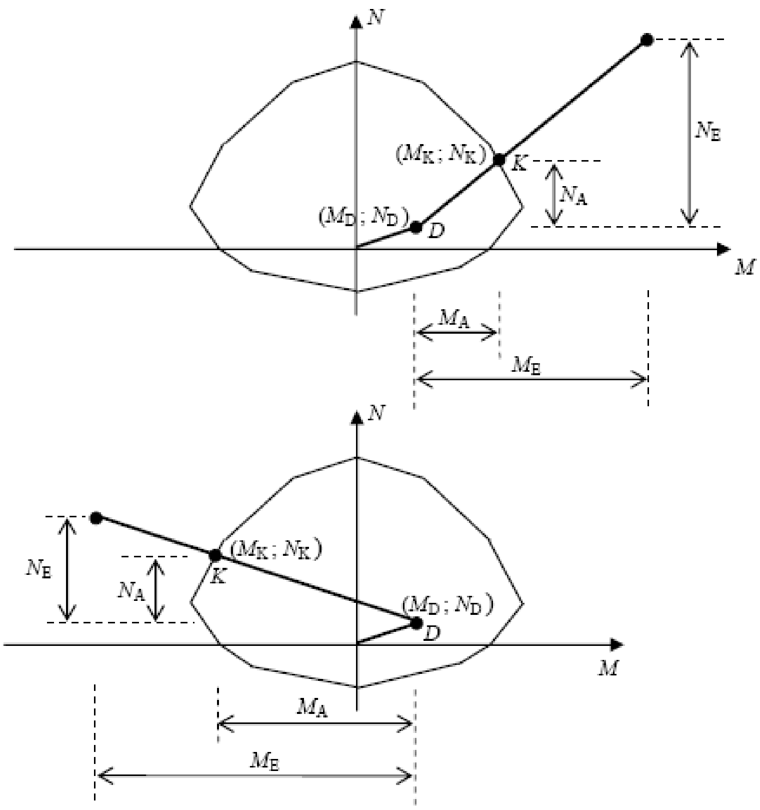


Figure 2.8 : Axial load calculation with graphical procedure

Member failure type is defined according to the shear capacity and shear demand. Shear capacities (V_r) for beam end sections and column mid sections are calculated according to section 2.1.2.1. Shear demands are the minimum of V_E from earthquake and V_E related to capacity. For columns, strong column – weak beam check must be done while calculating capacity related shear demand.

If V_E is greater than V_r, then the member is shear critical and defined as brittle, otherwise it is moment critical and ductile.

2.1.3 Performance Assessment of Members

For ductile members, earthquake induced moments (M_E) are divided by residual moment capacities (ΔM_K) and the demand to capacity ratios (r) are calculated. By using residual moment, comparison of earthquake demand with remaining capacity is achieved. Some portion of capacity is occupied with gravity loading and only remaining part can be used by earthquake. This is illustrated in Figure 2.9. For brittle members, shears are used instead of moments.

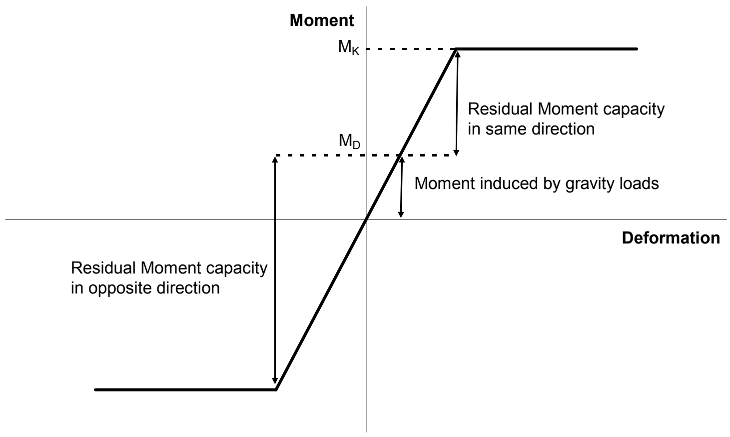


Figure 2.9 : Residual moment capacity

Calculated demand to capacity ratios are checked with the limit r values (r_{Limit}) obtained from the 2007 Turkish Earthquake Code for the target performance level.

For columns, r_{Limit} values (Table 2.2) are related to the axial force, confinement of member ends and shear force. r_{Limit} values are referred to the ductility of the sections, thus confinement has significant effect on curvature ductility and accordingly on the r_{Limit}. In addition to this, axial force is another criterion influencing ductility. As the axial force increases, ductility decreases. This is why for higher axial load levels r_{Limit} values are lower. Effect of axial force on ductility can be observed in Figure 2.10. Finally shear is important for member performance because for higher shear demands, member approaches to brittle behavior and this result in a lower value for r_{Limit}.

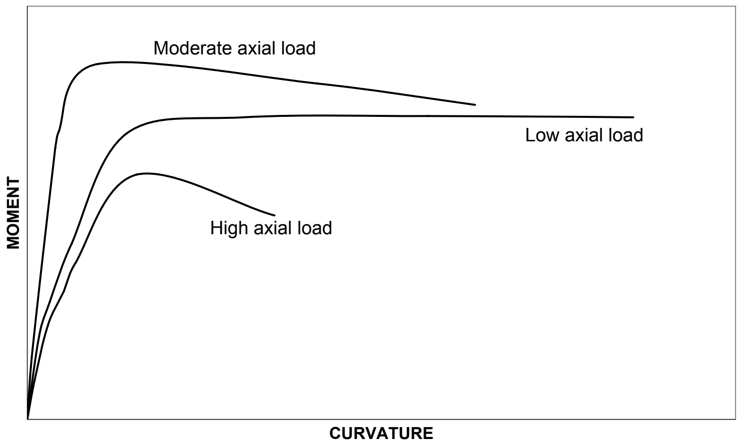


Figure 2.10 : Axial load effect on ductility

Table 2.2 : r_{Limit} values for columns

COLUMNS					
Ductile Columns			**Performance Level**		
$\frac{N_K}{A_c f_{cm}}$	**Confinement**	$\frac{V_E}{b_w d f_{cm}}$	**IO**	**LS**	**CP**
≤ 0.1	C	≤ 0.65	3	6	8
≤ 0.1	C	≥ 1.30	2.5	5	6
≥ 0.4 and ≤ 0.7	C	≤ 0.65	2	4	6
≥ 0.4 and ≤ 0.7	C	≥ 1.30	1.5	2.5	3.5
≤ 0.1	NC	≤ 0.65	2	3.5	5
≤ 0.1	NC	≥ 1.30	1.5	2.5	3.5
≥ 0.4 and ≤ 0.7	NC	≤ 0.65	1.5	2	3
≥ 0.4 and ≤ 0.7	NC	≥ 1.30	1	1.5	2
≥ 0.7	-	-	1	1	1

Here N_K is the total axial load calculated by Equation 2.13. V_E is the capacity related shear demand shown in Figure 2.4.

For beams, r_{Limit} values (Table 2.3) are related to the reinforcement ratio, confinement and shear force of member ends.

Table 2.3 : r_{Limit} values for beams

BEAMS					
Ductile Beams			**Performance Level**		
$\frac{\rho - \rho'}{\rho_b}$	**Confinement**	$\frac{V_E}{b_w d f_{cm}}$	**IO**	**LS**	**CP**
≤ 0.0	C	≤ 0.65	3	7	10
≤ 0.0	C	≥ 1.30	2.5	5	8
≥ 0.5	C	≤ 0.65	3	5	7
≥ 0.5	C	≥ 1.30	2.5	4	5
≤ 0.0	NC	≤ 0.65	2.5	4	6
≤ 0.0	NC	≥ 1.30	2	3	5
≥ 0.5	NC	≤ 0.65	2	3	5
≥ 0.5	NC	≥ 1.30	1.5	2.5	4

Reinforcement ratio has a significant effect on member performance for beams. To take into account of this property it is included in tables as follows:

$$\frac{\rho - \rho^{'}}{\rho_b} \tag{2.15}$$

where:

ρ is the tension reinforcement ratio.

ρ′ is the compression reinforcement ratio.

ρ_b is the balance reinforcement ratio.

Here it should be noticed that tension and compression reinforcement depends on the member behavior under earthquake loading. If the positive moment for left end is induced then the bottom reinforcement is in tension while top is in compression. For the right end, it is vice versa.

If $\rho - \rho' < \rho_b$ then the section will be under-reinforced which effects the ductility of the section. Hence for increasing values of $\frac{\rho - \rho^{'}}{\rho_b}$ ductility decreases and so r_{Limit} decreases.

Table 2.4 : r_{Limit} values for shear walls

SHEAR WALLS			
	Performance Level		
Confinement	**IO**	**LS**	**CP**
C	3	6	8
NC	2	4	6

The parameters used for the decision of member behavior for shear wall are its height to length ratio, existence of wall caps and reinforcement ratio with shear check. Shear demand of walls are calculated according to Equation 2.16 where; elastic analysis shear demand (V_D) is multiplied with moment capacity (M_K) to elastic moment demand (M_D) ratio.

$$V_E = \frac{M_K}{M_D} V_D \tag{2.16}$$

If shear demand (V_E) is smaller than shear capacity (V_r) of the section and height of the wall (H_w) is larger than two times the length (L_w) then section is accepted to be ductile. Otherwise, it is brittle.

Before final acceptance step, r values can be normalized with r_{Limit} so that a normalized acceptance level can be assigned to all members. Normalized r values can be calculated by simply dividing r with r_{Limit}. Although this is not required in the 2007 Turkish Earthquake Code, this practical representation will be used in the rest of the thesis.

- If $r / r_{Limit} \leq 1$ then member end is acceptable.
- If $r / r_{Limit} > 1$ then member end is unacceptable.

If one of the ends of the member is unacceptable this results in member to be unaccepted. If both ends are acceptable then the member is acceptable for that performance level.

2.2 Nonlinear Assessment Procedures

Second alternative for assessment offered in the 2007 Turkish Earthquake Code is nonlinear Static (Pushover) Analysis. Nonlinear behavior of the structure can be understood and hinge mechanism of the system is identified by this methodology. Equivalent Static Analysis and Mode Superposition will be explained in the following sections.

Procedure can be summarized as follows:

1-) 3-D model is prepared and gravity load demands of 1G+nQ combination are calculated.

2-) Cracked sections are defined by using axial loads of Step 1, assigned to the members and new demands are calculated.

3-) Considering new demands, moment-plastic rotation relations and interaction diagrams are defined for column and shear walls. Moment-plastic rotation relations are defined for beam. These plastic sections are assigned at the clear span ends for beams and columns, and at the bottom of the shear walls.

4-) Equivalent lateral load distribution is calculated by modal properties and assigned to the mass center of all floors.

5-) Two pushover analysis is done for each orthogonal direction. First one is the force controlled for vertical loading and the next is displacement controlled for lateral loading, until system reaches its collapse point.

6-) Roof Displacement – Base Shear curve is converted into Spectral Acceleration – Spectral Displacement curve and target displacement is calculated for the desired performance level.

7-) Force and rotation demands are obtained at this target displacement for all members.

8-) Shear demands are controlled with shear capacities and section failure type is defined by this check. If $V_E > V_r$ for any end of the member, it is called as brittle and not acceptable. If $V_E < V_r$ then member is ductile and need further process.

9-) If the member is ductile, then total curvature is calculated and strain values for that curvature is obtained for the section.

10-) Strain values calculated at Step 9 are compared with strain limits and member acceptance is defined.

A schematic summary can be found in Figure 2.11. Details of the steps will be discussed in following topics.

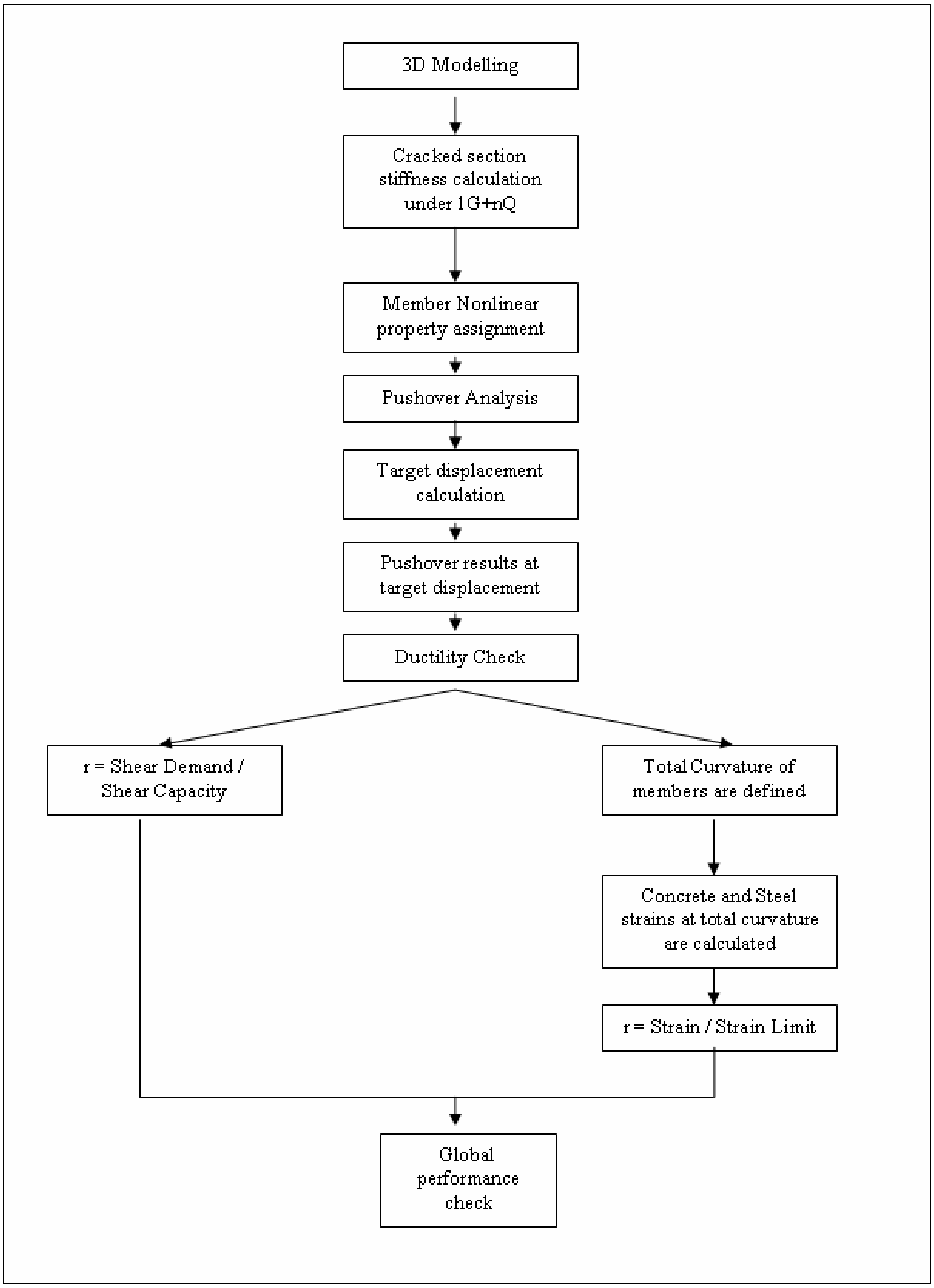


Figure 2.11 : Nonlinear analysis procedure

2.2.1 Modeling for Pushover Analysis

Three dimensional models were constructed by any 3D analysis software primarily for establishing the fundamental static and dynamic characteristics of the structure. For this study SAP2000 (CSI, 1998) is used and details are explained according to the features of SAP2000. By using vertical loads, cracked section stiffness are calculated and assigned to the members same as linear elastic model. Section 2.1.1 rules are also valid for nonlinear analysis. Nonlinear properties of the sections are calculated by the help of Response2000 (Bentz E.C., 2000) and a VisualBasic Macro with MS Excel. For beam sections moment – curvature relations, for column and shear wall sections both moment – curvature relations and interaction diagrams are used as nonlinear property.

For the calculation of moment – curvature relations, Modified Kent & Park concrete model (Appendix A) and steel model as suggested in the 2007 Turkish Earthquake Code are selected. It is important to use a concrete material model which takes into account the effect of confinement.

In order to estimate the yield and ultimate values for moment and curvatures, the curve must be bi-linearized. Priestley (2003) method is used for bi-linearization. The method can be summarized as follows, and shown in Figure 2.12.

- Find the ultimate curvature (ϕ_u) and moment (M_u) from the moment-curvature curve.
- Yield curvature ($\phi_{y'}$) and yield moment (M_y) is the curvature when tension steel first yields ($\varepsilon_y = f_y / E_s$) or concrete extreme fiber attains 0.002 which can be accepted as yield strain of unconfined concrete.
- ϕ_y is modified with M_n / M_y factor where M_n is the moment at the point that the reinforcement tension strain reaches 0.015 or the concrete extreme compression fiber strain reaches 0.004, whichever occur first.

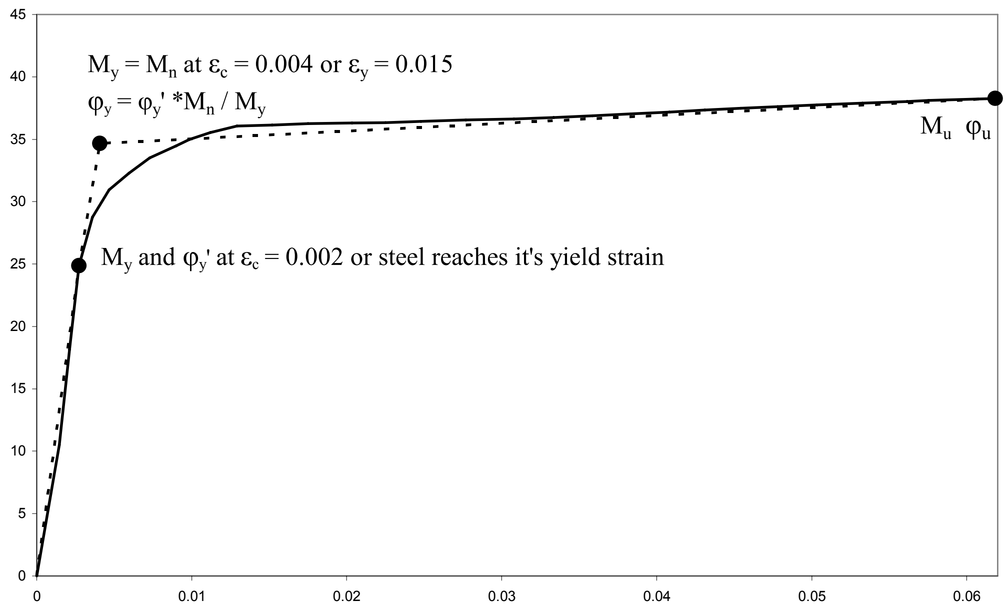


Figure 2.12 : Yield and ultimate curvature calculation

SAP2000 user defined hinge properties of M3 type is selected for beam sections. However, software requires moment – rotation relation rather than moment – curvature. So, a conversion must be done.

$$\theta_y = (\phi_y * L_n) / 6 \tag{2.17}$$

$$\theta_u = (\phi_u - \phi_y) * L_p + \theta_y \tag{2.18}$$

where:

ϕ_y = yield curvature of moment -curvature relation

ϕ_u = ultimate curvature of moment -curvature relation

L_p = plastic hinge length of section

L_n = clear length of the member

θ_y = yield rotation

θ_u = ultimate rotation

Although different formulas can be found in literature for the calculation of plastic hinge length, it is taken as half of the cross-section depth ($L_p = h / 2$).

In addition to moment – curvature, interaction diagrams must be assigned to the vertical members. SAP2000 includes PMM hinge type for this purpose. It has default hinge properties whereas for the sake of accuracy user defined hinge property is desired. As the model is 3D and for considering torsional effects interaction surface must be defined rather than interaction curve. SAP2000 requires minimum of 5 curves for a section for one quadrant. Other quadrants can be defined as symmetric to the one defined. Axial force–moment capacity curves corresponding to bending about two major axis of each column section are obtained. From these major curves, the other axial force - moment capacity curves (Figure 2.13) are obtained by the following equation proposed by Parme et al (1966):

$$\left(\frac{Mux}{Muxo}\right)^{\left(\frac{\log(0.5)}{\log(\beta)}\right)} + \left(\frac{Muy}{Muyo}\right)^{\left(\frac{\log(0.5)}{\log(\beta)}\right)} = 1 \qquad (2.19)$$

where:

Muxo = uniaxial flexural strength about x-axis

Muyo = uniaxial flexural strength about y-axis

Mux = component of biaxial flexural strength on the x-axis at required inclination

Muy = component of biaxial flexural strength on the y-axis at required inclination

β = parameter dictating the shape of interaction surface

$\beta = 0.7$ for ground, first and 2nd floor columns

$\beta = 0.6$ for 3rd and 4th floor columns

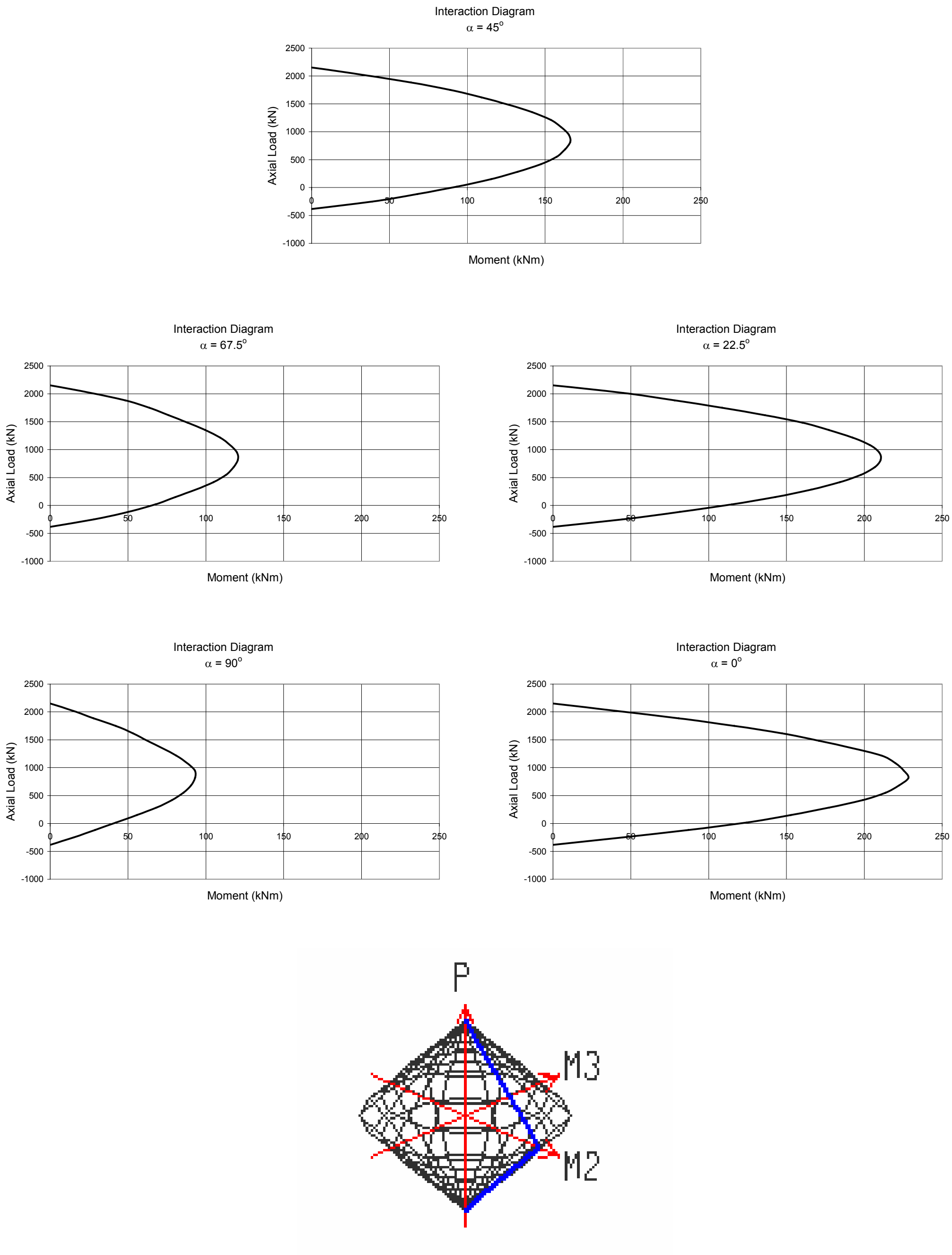


Figure 2.13 : Interaction diagrams and interaction surface

2.2.1.1 Equivalent Static Procedure

This method can not be applied if the building does not satisfy the following initial checks.

- Excluding basement floors, maximum 8 stories is allowed.
- Torsional irregularity coefficient must be less than 1.4
- Building height, measured from the ground level, must be less than 25m.
- Mass participation ratio for the first mode must be at least 70%

It is assumed that the Equivalent Static Lateral Load distribution does not change with plastic hinge formations. Load distribution is calculated by multiplication of first mode shape (Φ_1) with mass of the storey. Calculated loads are assigned to the mass center of the storey for two lateral orthogonal directions and rotation about vertical direction.

Pushover analysis is a step-by-step load increment procedure starting with a gravity loading and continues until required top displacement is monitored.

2.2.1.2 Mode Superposition

This procedure can be applied to any structure for considering higher mode effects and including sufficient number of modes in the assessment of global performance of the structure. Cumulative mass participation ratios of modes included in the analysis must be at least 90%.

Procedure simply involves demand and drift calculation of different modes at corresponding target displacements and combination of these demands with a proper combination rule. Square Root of Sum of Squares (SRSS) or Complete Quadratic Combination (CQC) can be used for combination.

2.2.2 Displacement Demand of the Structure

After the pushover analysis is finalized and Roof Displacement – Base Shear curve (Capacity Curve) is obtained, target displacement for selected performance level is needed to be assessed. The method described in the 2007 Turkish Earthquake Code will be explained here.

Both Code spectrum and capacity (pushover) curves are converted to displacement (d, S_d) vs. Spectral Acceleration (a, S_a) curves.

$$S_d = \frac{g\,S_a\,T^2}{4\,\pi^2} \qquad \text{For Code spectrum conversion} \qquad (2.20)$$

$$\left.\begin{array}{l} S_d = \delta * PF1 * \phi_{r1} \\ S_a = \text{Base Shear} / M' \end{array}\right\} \quad \text{For capacity curve conversion} \qquad (2.21)$$

Where:

PF1 : Modal participation ratio

ϕ_{r1} : Roof displacement for the first mode.

M’ = m*α1

m : Total mass

α1 : Modal mass participation ratio for the first mode

Slope of the linear elastic Spectral Displacement (S_{de}) vs. Spectral Acceleration (S_{ae}) is frequency of the corresponding period (ω^2), and is equal to $(\frac{2\pi}{T})^2$. Thus S_{de} is calculated from S_{ae} as shown in Figure 2.14.

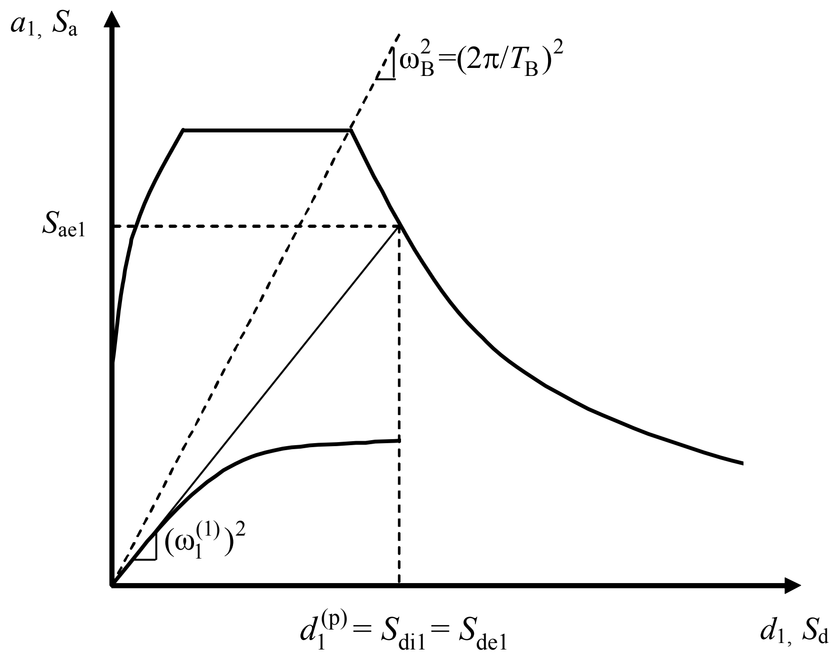


Figure 2.14 : Demand calculation

If the natural vibration period of the corresponding direction is greater than the corner period of spectrum (T_B) then the equal displacement rule is valid, and thus inelastic spectral displacement can be accepted as equal to the elastic spectral displacement.

$$S_{die} = S_{de} \tag{2.22}$$

Else, an iterative procedure is performed. Corresponding Elastic Spectral Acceleration (S_{ae1}) and Elastic Spectral Displacement (S_{de}) are the initial point of iteration.

$$R_{y1} = \frac{S_{ae1}}{\alpha_{y1}} \quad (2.23)$$

Here,

a_{y1}^{o} **:** is calculated by maintaining equal areas under capacity curve and bi-linearized capacity curve.

$$C_{R1} = \frac{1 + (R_{y1} - 1)T_B/T_1}{R_{y1}} \geq 1 \quad (2.24)$$

For the first iteration C_{R1} can be taken as 1 but then it will be calculated from the formula given above.

$$S_{die1} = C_{R1} * S_{de} \quad (2.25)$$

Iteration will be repeated until S_{die1} results are close enough (Figure 2.15).

Final S_{die1} value is the inelastic displacement demand which is also called target displacement.

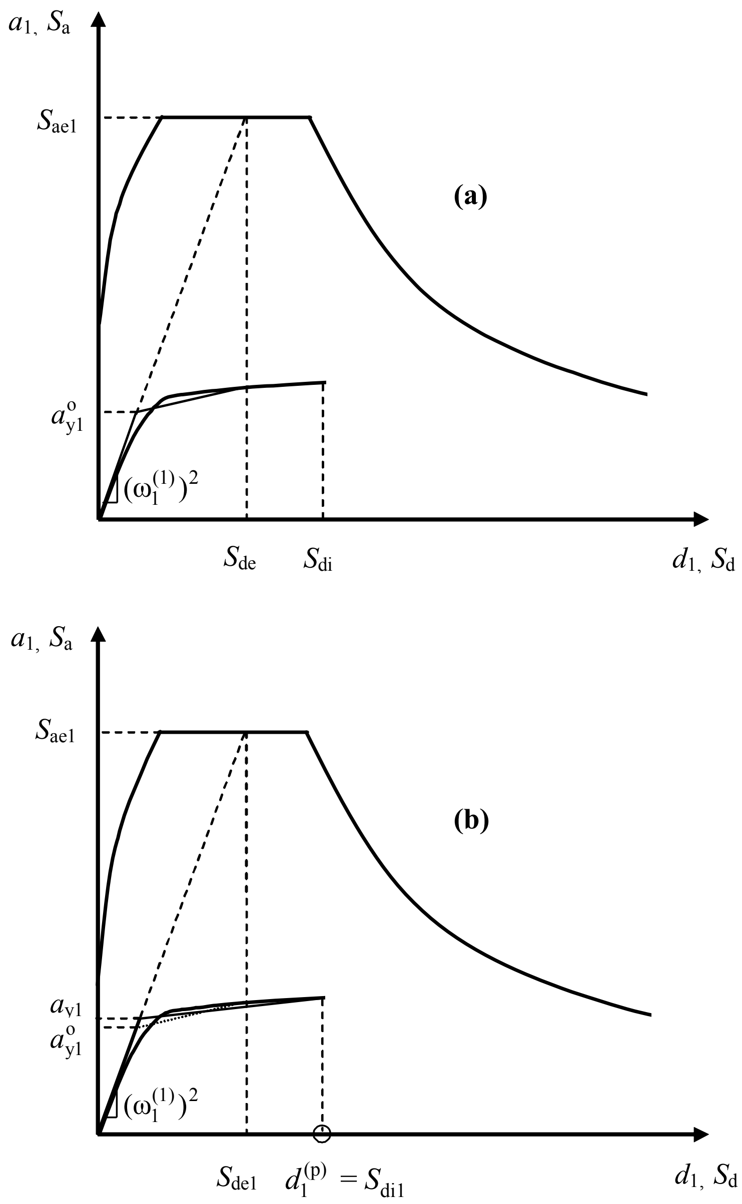


Figure 2.15 : Target displacement iteration

2.2.3 Demand and Capacity of Members

For the calculated target displacement, member demands are obtained. Both shear forces and plastic rotation demands will be used in further calculations. Also axial loads are needed. All values are taken from the last step of pushover analysis at target displacement.

Similar to linear elastic method, first the ductility check is performed. If V_E corresponding to the last step of pushover analysis at target displacement is greater than shear capacity V_r, then the member is called brittle. Otherwise, it is ductile. If the member is brittle than the capacity – demand ratio r is the ratio of shear demand V_E to shear capacity V_r. However, if the member is ductile, different evaluation is needed to be done.

Rather than moments, strain values for concrete and steel at the total curvature is in concern for inelastic assessment.

2.2.3.1 Ductile Members

If the member is categorized to be ductile, then acceptance check is done by comparison of concrete and reinforcement strains with strain limits of expected performance level. For different damage levels, different strain limits are available. Concrete strain limit depends on confinement whereas reinforcement strain limits are directly estimated. Strain limits are given in Table 2.5.

For IO, concrete strain corresponds to the strain of outermost fiber of the section, whereas for LS and CP performance levels strain of outermost fiber of confined region is used. It should be noticed that ρ_s is transverse reinforcement that fulfill the requirements described in section 3.2.8 of the 2007 Turkish Earthquake Code.

Table 2.5 : Strain limits

	Concrete Strain Limit	Steel Strain Limit
Immediate Occupancy	0.0035	0.01
Life Safety	$0.0035 + 0.010(\rho_s / \rho_{sm}) \leq 0.0135$	0.04
Collapse Prevention	$0.0040 + 0.014(\rho_s / \rho_{sm}) \leq 0.018$	0.06

Strain values at total curvature of the section are compared with the limits given in Table 2.5 and member acceptance is decided. Total curvature is calculated using the following equations.

$$\phi_p = \frac{\theta_p}{L_p} \tag{2.26}$$

$$\phi_t = \phi_y + \phi_p \tag{2.27}$$

2.3 Global Performance Estimation

After determining member performance and acceptability, building global performance acceptance is checked. Three basic criteria are used in order to control the building performance for both linear and nonlinear assessment procedures.

- Percentage of total shear of unaccepted columns to storey total shear force.
- Percentage of unaccepted beams to all beams in the direction considered.
- Relative interstorey displacement.

These criteria are checked with different limits for different performance levels. Performance levels are described below.

Immediate Occupancy Performance Level:

Structural performance level Immediate Occupancy is defined as the post-earthquake damage state that remains safe to occupy, essentially retains the pre-earthquake design strength and stiffness of the structure, and is in compliance with the acceptance criteria specified here for this structural performance level.

For any storey, for all vertical members, immediate occupancy limits should be satisfied. None of exceedance is allowed. On the other hand, 10% of the beams at most are allowed to exceed the immediate occupancy limits for beams. Relative storey displacement limit is 0.01. In case of retrofitting the brittle members, and satisfying all these conditions, the building is accepted for Immediate Occupancy Performance Level.

Life Safety Performance Level:

Structural performance level Life Safety is defined as the post-earthquake damage state that includes damage to structural components but retains a margin against onset of partial or total collapse in compliance with the acceptance criteria specified here for this structural performance level.

A building can be accepted in Life Safety performance level by satisfying the following conditions and retrofitting brittle members, if exists.

For any storey, for vertical members life safety limits should be satisfied but 20% of total storey shear is allowed to be taken by vertical elements of severe damage. This limit is 40% for top storey. Shear taken by the columns with both ends exceeded the immediate occupancy limits are limited with the 30% of total storey shear. This rule excludes the columns with CBCR is larger than 1.2

for both ends. Moreover, 30% of the beams at most are allowed to exceed the life safety limits for beams. Relative interstorey displacement limit is 0.03.

Collapse Prevention Performance Level:

Structural performance level Collapse Prevention is defined as the post-earthquake damage state that includes damage to structural components such that the structure continues to support gravity loads but retains no margin against collapse in compliance with the acceptance criteria specified here for this structural performance level.

Accepting the brittle members in the collapse damage level, building that satisfies the following conditions is accepted to be in Collapse Prevention performance level.

For any storey, 20% of the beams at most are allowed to exceed the collapse prevention limits for beams. All other members are allowed to be in the limits of collapse prevention, life safety or immediate occupancy. Shear taken by the columns with both ends exceeded the immediate occupancy limits are limited with the 30% of total storey shear. This rule excludes the columns with CBCR is larger than 1.2 for both ends. Relative storey displacement limit is 0.04.

Collapse Performance Level:

If any building can not satisfy Collapse Prevention performance level, then the building will be at Collapse Performance Level.

Table 2.6 : Global performance limits

	Performance Level		
	Immediate Occupancy	**Life Safety**	**Collapse Prevention**
Column Shear %	100% of Minimum damage	20% of Severe Damage	None of Collapse
Beam %	10% of Significant Damage	30% of Severe Damage	20% of Collapse
Relative Interstorey Displacement	0.01	0.03	0.04

CHAPTER III

CASE STUDY 1: ASSESSMENT OF AN EXISTING 5-STOREY RESIDENTIAL BUILDING IN DÜZCE

In this case study, a five-storey existing residential building was assessed using both linear elastic and nonlinear procedures described in the 2007 Turkish Earthquake Code. Moreover, actual performance evaluation of the building was performed according to the damage demand during the 1999 Düzce Earthquake.

3.1 Structural Properties of the Building

Case study building is composed of five stories with varying properties. All floors are 2.8m in height and floor areas are 432.97m^2. Overhangs exist about 1.5m long, starting from the first floor. In the long direction, full framing was provided along the length of the building only at the two exterior axes and at the first storey. Some of the exterior beams in these frames were offset towards the edge of the overhangs in the upper stories; therefore, the second and upper stories contained only partial framing along both the external and the internal axes. Moreover, beam K123 exists only at the first storey. In the short direction, partial symmetry and framing is provided. Building photo, typical floor plan and 3D model can be found in Figure 3.1 through Figure 3.4. Member dimensions are not uniform through the height of the building. Typical section dimensions for frame 1 are given in Table 3.1. Example calculations for procedures will be given for the column 3S8 and beam K333 located on frame 1.

Figure 3.1 : Building photo

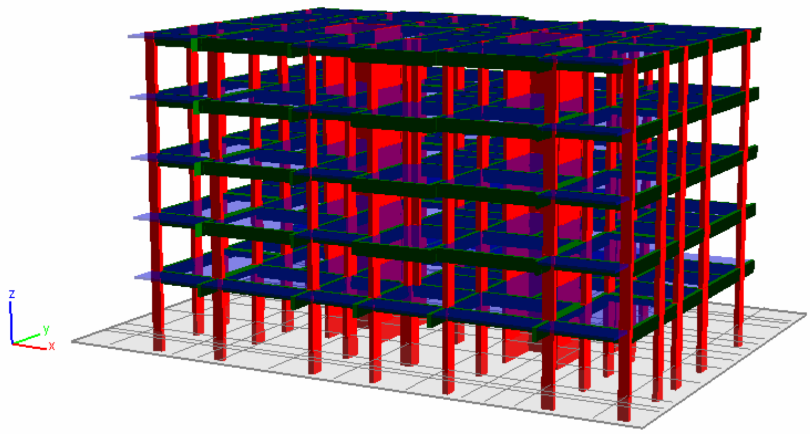


Figure 3.2 : 3D model of the building

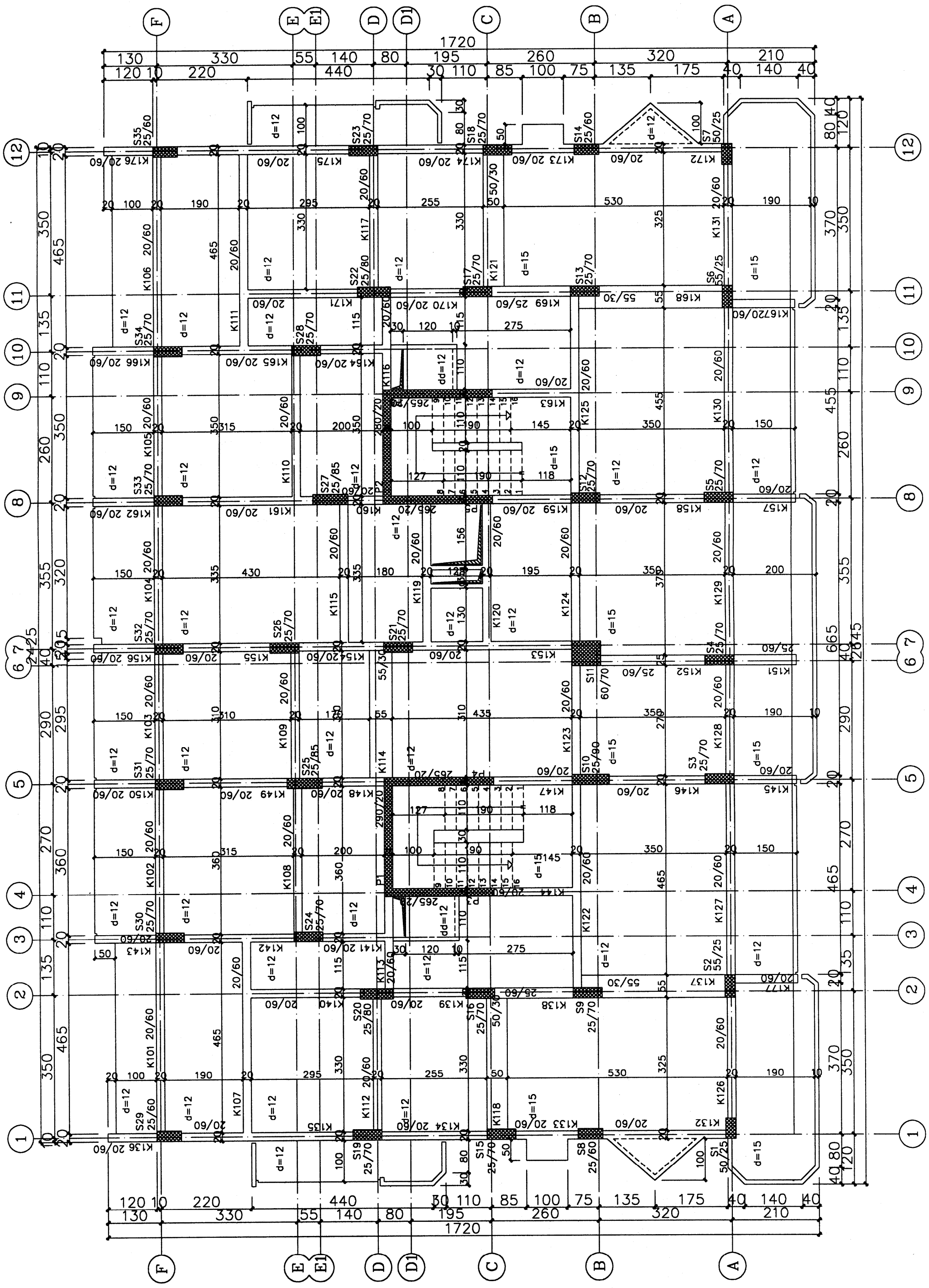


Figure 3.3 : Floor plan of 1st storey

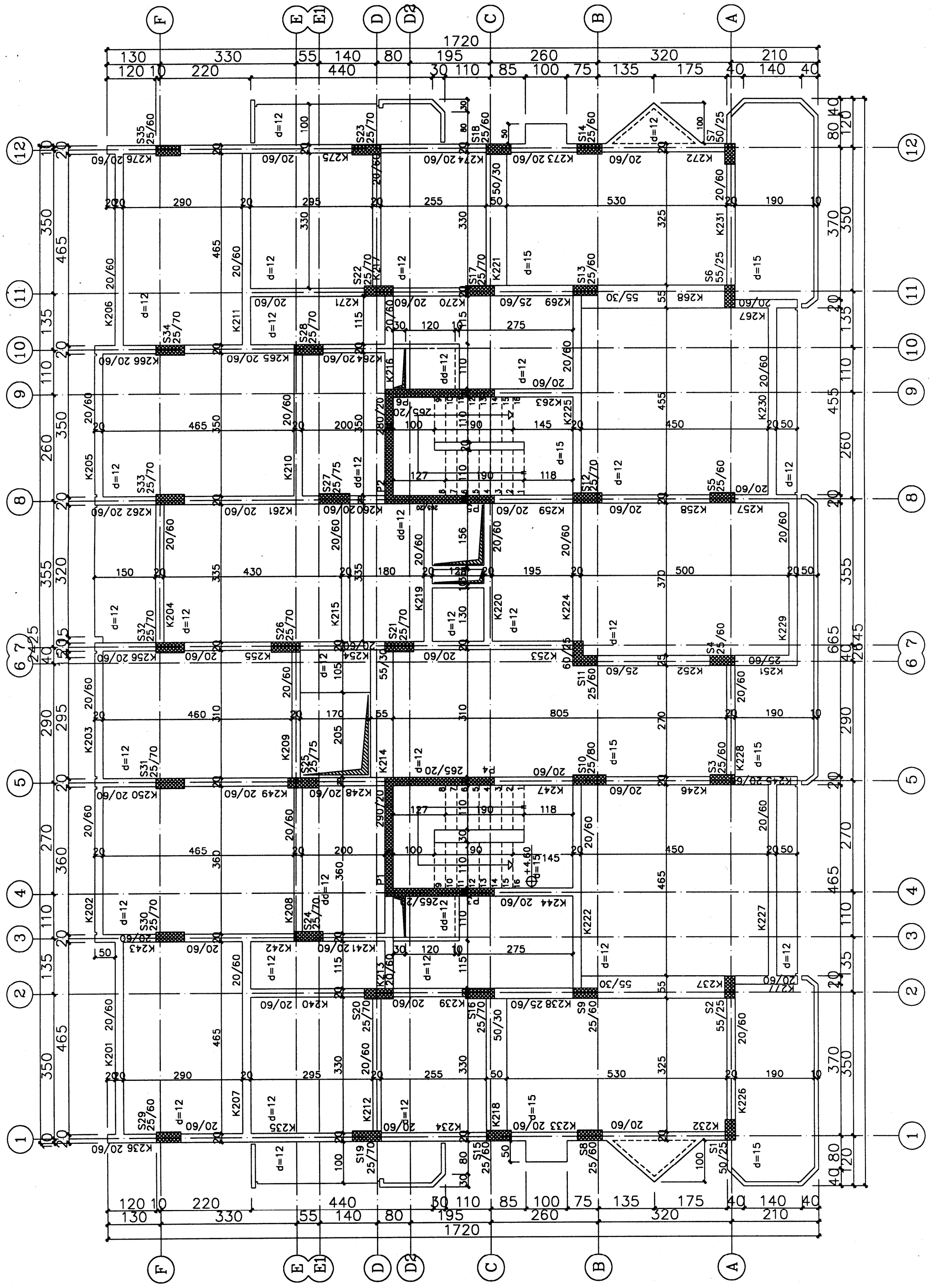


Figure 3.4 : Floor plan of 2nd storey and higher

All beams are in 20cm x 60 cm on frame 1. Dimensions of columns in frame 1 are given in Table 3.1. Existing building properties and parameters for the 2007 Turkish Earthquake Code are tabulated in Table 3.2.

Table 3.1 : Column dimensions for frame 1

	S1		**S8**		**S15**		**S19**		**S29**	
Storey	**bx (mm)**	**by (mm)**	**bx (mm)**	**by (mm)**	**bx (mm)**	**by (mm)**	**bx (mm)**	**by (mm)**	**bx (mm)**	**by (mm)**
1	500	250	250	600	250	700	250	700	250	600
2	500	250	250	600	250	600	250	700	250	600
3	500	250	250	600	250	600	250	600	250	600
4	500	250	250	600	250	500	250	600	250	500
5	500	250	250	600	250	500	250	500	250	500

Table 3.2 : Existing properties and code parameters of the building

Existing Building Properties

Project available?	Yes
Knowledge level	Extensive
Knowledge level factor	1
Existing concrete strength	12 MPa (E_c = 25000 MPa)
Existing reinforcement strength	220 MPa

2007 Turkish Earthquake Code Parameters

Seismic Zone	1
Seismic Zone Factor (A_o)	0.4
Building Importance Factor	1
Soil Class	Z3
Live Load Participation Factor	0.3
Target Performance Level	Life Safety

3.2 Performance of the Building During the 1999 Düzce Earthquake

The case study building was constructed in 1991 and located at Düzce, Turkey. The building was exposed to two destructive earthquakes in 1999. The first one is the 17 August 1999 Marmara Earthquake with a magnitude of M_w = 7.4. Its epicenter was located at the east end of the Gulf of İzmit, near the town of Gölcük, which was about 40km away from the city of Düzce. The second one is the 12 November 1999 Düzce Earthquake with a magnitude of M_w = 7.1. Its epicenter was located 8km away from the city center of Düzce. Ground acceleration record for the Düzce earthquake is available from the strong motion station located 250m to the north of the building. Peak ground acceleration values were 0.410g in North-South direction, 0.513g in East-West direction and 0.34g in the vertical.

The building has survived from both earthquakes with some damages, and it was evacuated after the Marmara Earthquake. First damage investigation was done in July 2000 and continued until the summer of 2001. Only the first floor was accessible for the damage investigation because owners of the upper floors were not available. This is the reason why only first floor damage will be evaluated in this study.

At first glance, building was thought to have large amount of damage because of widespread spilled plaster and damaged partition walls. However, detailed investigations on structural system revealed that the columns sustained almost no damage. Beams exhibited thin transverse or diagonal cracks. Shear walls suffered from diagonal cracks and some horizontal cracks. Examples of damage observations are shown in Figure 3.5 to Figure 3.8.

First storey damage observations are illustrated in Figure 3.9 with the damage level that the members are assigned. For the global performance assessment of the first storey, shear demands of columns and shear walls from the linear elastic analysis were used. This performance evaluation is given in Table 3.3.

Figure 3.5 : Flexural crack at the end of beam K154, where it spans into the column S04

Figure 3.6 : Diagonal crack passing through beam K131, column S6 and the adjacent walls, at the first story

Figure 3.7 : Diagonal cracks on the web portion of shear wall P1 indicating southward movement

Figure 3.8 : Horizontal cracks on the northern flange of shear wall P1, at the slab level

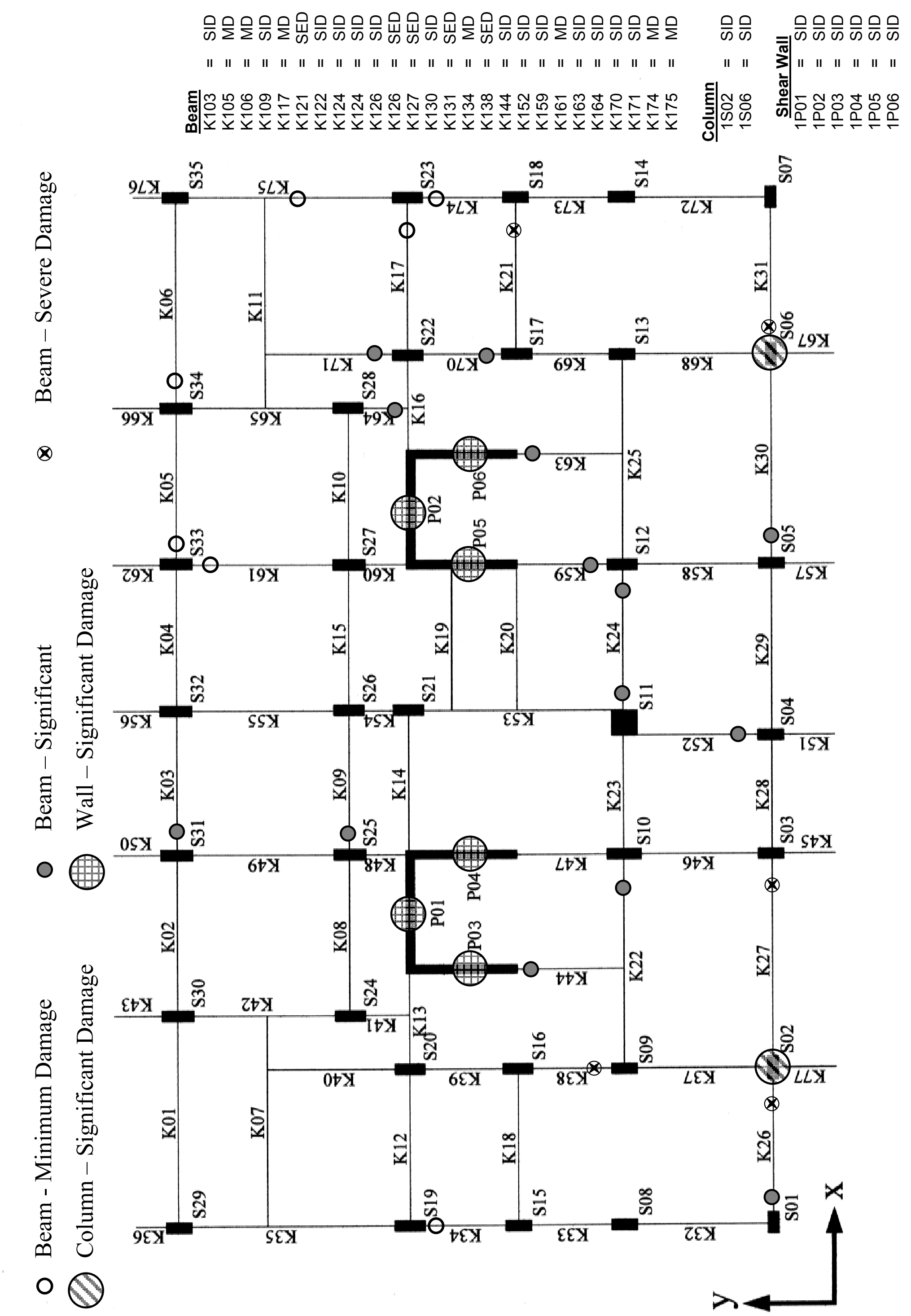


Figure 3.9 : Damage observation at the first storey

Table 3.3 : Performance evaluation of first storey

OBSERVATION : COLUMNS						
1st Storey	Number of Columns +X	Number of Walls +X	% Shear Columns & Walls +X	Number of Columns +Y	Number of Walls +Y	% Shear Columns & Walls +Y
No Damage	33	0	17.6	33	0	0.4
Minimum Damage	0	0	0	0	0	0
Significant Damage	**2**	**2**	**82.4**	**2**	**4**	**99.6**
Severe Damage	0	0	0	0	0	0
Collapse	0	0	0	0	0	0
Total	35	2	100	35	4	100
Performance Level	**Life Safety**			**Life Safety**		

OBSERVATION : BEAMS				
1st Storey	Number of Beams +X	% Beams +X	Number of Beams +Y	% Beams +Y
No Damage	19	61.3	22	64.7
Minimum Damage	3	9.7	4	11.8
Significant Damage	5	16.1	7	20.6
Severe Damage	**4**	**12.9**	**1**	**2.9**
Collapse	0	0.0	0	0.0
Total	31	100	34	100
Performance Level	**Life Safety**		**Life Safety**	

Life Safety +X

Columns +X : % Shear exceeding Significant Damage = 0 < 20%

Beams +X : % Beam exceeding Significant Damage = 12.9 < 30%

Life Safety +Y

Columns +Y : % Shear exceeding Significant Damage = 0 < 20%

Beams +Y : % Beam exceeding Significant Damage = 2.9 < 30%

According to the 2007 Turkish Earthquake Code, this building satisfies **"Life Safety"** performance level.

3.3 Assessment of the Building by Linear Elastic Procedure

3.3.1 Modeling and Analysis

Linear assessment procedure starts with correct modeling of the building for which cracked section stiffness was used for all members. Uncracked section stiffness of beams are reduced by 40% in order to obtain the cracked section stiffness, whereas for columns and shear walls, section modifiers are calculated with respect to axial load level of 1G+nQ load combination. Rigid end zone description, and the sample column and beam are shown in Figure 3.10 for frame 1. Modal properties were found as given in Table 3.4 by performing an eigenvalue analysis. Equivalent static loads were calculated and tabulated in Table 3.5 by using the first mode period and Code spectrum.

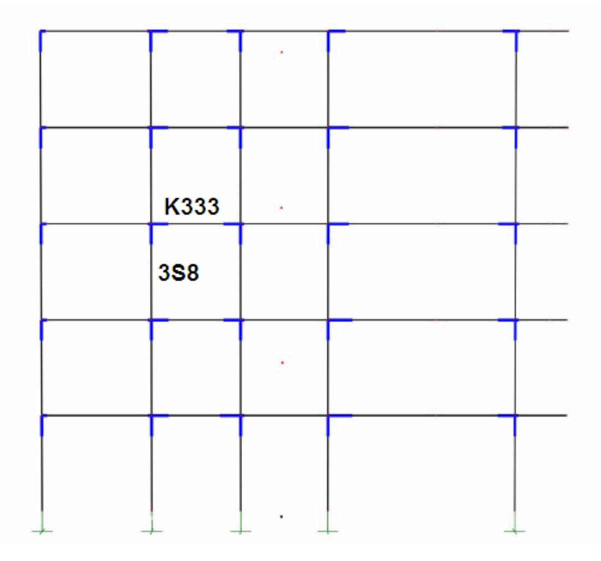


Figure 3.10 : Frame 1 with rigid end zones

Table 3.4 : Modal properties of the building

MODE	PERIOD	INDIVIDUAL MODE (PERCENT)		CUMULATIVE SUM (PERCENT)	
		UX	UY	UX	UY
1	0.514671	35.046	0.0031	35.046	0.0031
2 (X)	0.426501	41.373	0.0057	76.4191	0.0088
3 (Y)	0.374317	0.0001	76.0294	76.4192	76.0383
4	0.154411	4.7025	0.0013	81.1216	76.0396
5	0.118802	12.7542	0.0014	93.8759	76.0409
6	0.104618	0.0009	16.1041	93.8768	92.145

Using these modal properties equivalent static loads were calculated as follows:

$R_a(T_1)$ =1, I=1 , A_o = 0.4 and W = 20,689 kN

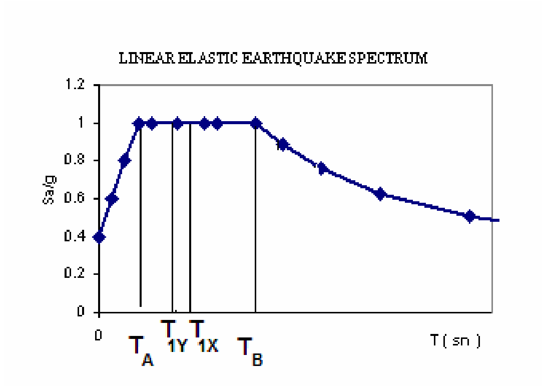
For T_x = 0.43s and T_y = 0.37s, S(T) values were calculated as 2.5g in both directions. From these values $A(T_1)$ values are 1g.

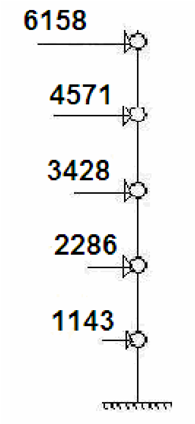
λ =0.85, V_t = 17858 kN

ΔF_N = 0.0075 N V_t = 0.0075 x 5 x 17585 = 659 kN

Table 3.5 : Equivalent static lateral load distribution

St	Storey Mass (t)	Storey Weight (W_i)(kN)	Storey Height (m)	H_i (m)	W_i H_i (kNm)	F_i (kN)
1	425	4169.25	2.8	2.8	11673.9	1142.769
2	425	4169.25	2.8	5.6	23347.8	2285.538
3	425	4169.25	2.8	8.4	35021.7	3428.307
4	425	4169.25	2.8	11.2	46695.6	4571.076
5	409	4012.29	2.8	14	56172.06	6158.207





$T_A = 0.15s$, $T_B = 0.6s$, $T_{1x} = 0.43s$, $T_{1y} = 0.37s$

Figure 3.11 : Earthquake design spectrum and lateral load distribution

Code spectrum can be used for the equivalent lateral load calculation but for the assessment purpose, 1999 Düzce earthquake ground motion data was preferred so that building can be assessed under the ground motion it was exposed to. Geometric mean of East-West and North-South components of the 1999 Düzce ground motion spectra is plotted with elastic Code Spectrum and shown on the same figure (Figure 3.12 and Figure 3.13).

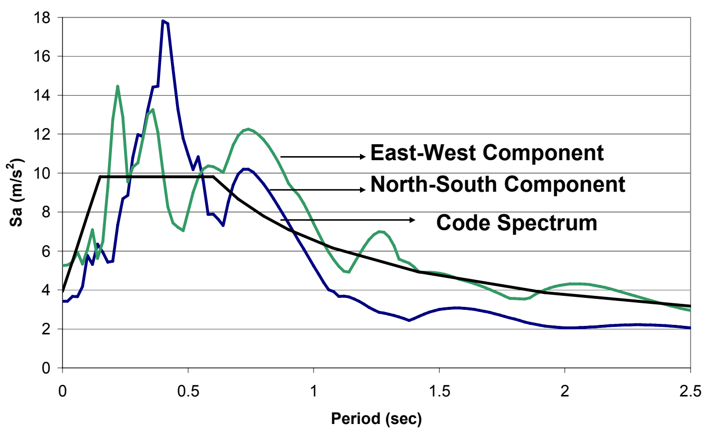


Figure 3.12 : 1999 Düzce earthquake spectral components and the Code spectrum

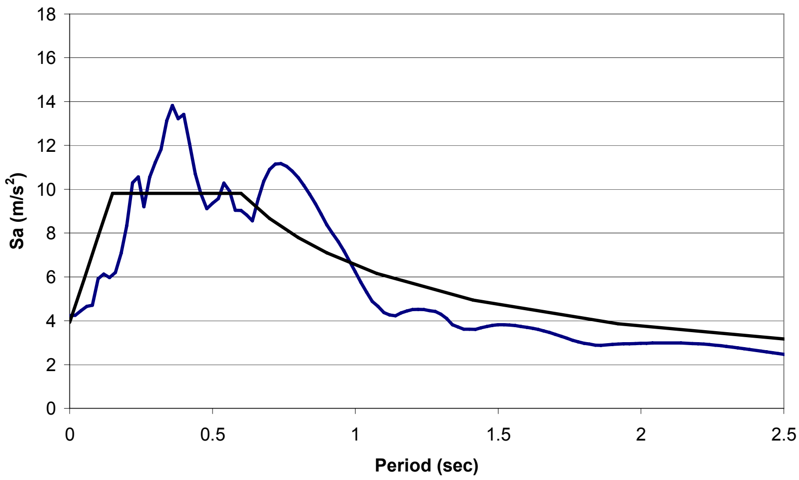


Figure 3.13 : 1999 Düzce mean earthquake spectrum and the Code spectrum

Before starting the assessment process, building model has been controlled for the appropriateness of the equivalent static load procedure. The building has;

Total height of 14m which is less than 25m.

Total number of floors is 5 which is less than 8.

Maximum torsional irregularity coefficient (η_{bi}) as shown in Table 3.6 was 1.18, which is less than 1.4.

These three checks showed that building satisfies the requirements of equivalent static load procedure but while performing spectral analysis, modal superposition procedure including five modes will be used for +X direction in order to account for the higher mode effects and to be consistent with the nonlinear solution for which modal superposition is a must because of the inappropriate modal mass participation ratio which is 41%.

Table 3.6 : Torsional irregularity check

	+X				+Y			
Storey	**(Δ_i)max**	**(Δ_i)min**	**(Δ_i)mean**	**η_{bi}**	**(Δ_i)max**	**(Δ_i)min**	**(Δ_i)mean**	**η_{bi}**
1	0.012	0.009	0.010	1.164	0.006	0.006	0.006	1.002
2	0.020	0.014	0.017	1.177	0.011	0.010	0.011	1.006
3	0.022	0.016	0.019	1.158	0.012	0.012	0.012	1.011
4	0.021	0.016	0.018	1.130	0.012	0.011	0.012	1.012
5	0.018	0.015	0.017	1.095	0.010	0.010	0.010	1.013

3.3.2 Calculation of Member Capacities

Beam moment capacities are calculated using a simple section analysis procedure. Example beam K333 (20cm x 60cm) has ϕ8/25cm transverse reinforcement with no confined region. Longitudinal reinforcement area and calculated moment capacities are given in Table 3.7.

Table 3.7 : End moment capacity of beam K333

	Moment capacity at top		**Moment capacity at bottom**	
	i	***j***	***i***	***j***
A_s (cm^2)	7.2	7.2	3.6	3.6
M_K (kNm)	89.9	89.9	53.0	53.0
				

Column moment capacity calculation will be carried out by using the procedure described in Section 2.1.2.2. A sample calculation is given for the column 3S8.

Axial load under lateral forces will be transferred from the beams joining to the columns 3S8, 4S8 and 5S8, and N_E notation will be used for these axial loads. N_D is the axial load of the column under 1G+nQ loading.

Capacity control procedure starts with the calculation of shear transmitted from beams. M_K represents the moment capacity of the beam and M_D is the moment induced by vertical loading which was reduced to 85% of the value. Third storey beams K332 and K333 join to the column 3S8 and shear transmitted to the column is $V_{E,3}$. The procedure for calculating axial force is given in Figure 3.14.

Beam K333

$M_{K,i}$(bottom)	: 53.0 kNm	$M_{K,j}$(top)	: -89.9 kNm
$M_{D,i}$	: -6.93 kNm	$M_{D,j}$	: -1.68 kNm
$\Delta M_{E,i} = M_{Ki}(bot) - M_{D,i}$		$\Delta M_{E,j} = M_{Kj}(top) - M_{D,j}$	
$= 53.0 - (-6.93)$	: 59.93 kNm	$= -89.9 - (-1.68)$	: -87.92 kNm

$$V_{E,3,2} = (\Delta M_{E,i} - \Delta M_{E,j}) / l_n$$

$$= (59.93 - (-87.92)) / 1.6 = 92.4 \text{ kN}$$

Same calculations were applied to the beam K332 and $V_{E,3,1}$ was found to be 45.6 kN in the opposite direction. Total shear transferred to the column 3S8 at storey 3 was;

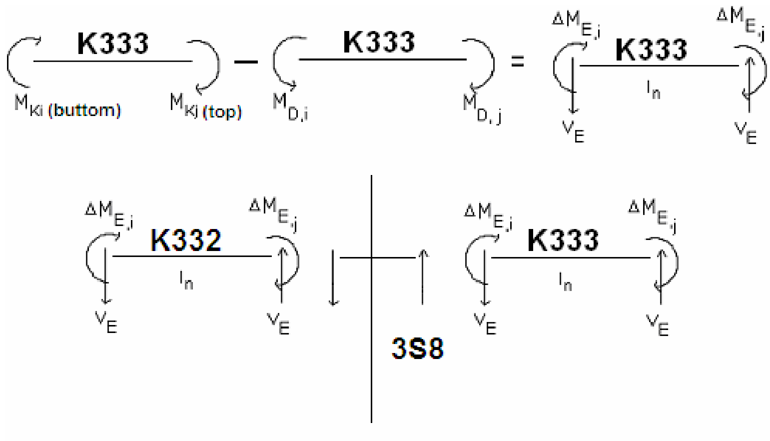


Figure 3.14 : Shear force transmitted from K332 and K333 to column 3S8

$V_{E,3} = V_{E,3,1} + V_{E,3,2} = 45.6 - 92.4 = -46.8$ kN (tension)

Similarly;

$V_{E,4} = -48.5$ kN , $V_{E,5} = -49.5$ kN

$N_{E,3}$ is the total shear force transmitted to the column 3S8 from the beams joining to the S8 columns above 3S8.

$N_{E,3} = V_{E,3} + V_{E,4} + V_{E,5} = -49.8 – 48.5 – 49.5 = -147.8$ kN (tension)

$N_D = 139.9$ kN

$N_D + N_{E,3} = 139.9 – 147.8 = -8.9$ kN (tension)

Moment capacity of the column 3S8 was calculated using the axial load level of -8.9 kN

$M_K = 87.5$ kNm

Calculating the moment capacity of 4S8 as 88.0 kNm column to beam capacity ratio (CBCR) was found as follows;

$$CBCR = \frac{\left(M_{Kbot} + M_{Ktop}\right)}{\left(M_{Ki(bot)} + M_{Kj(top)}\right)} = \frac{(88.0 + 87.5)}{(53 + 89.9)} = 1.23$$

CBCR values are used to identify the hinge mechanism and if it is smaller than 1, moment capacities of beams joining to that joint are reduced by CBCR. Other values of CBCR of 4S8 and 5S8 were 1.23, 0.62 respectively. On the adjacent column axis A CBCR values were 1.14, 1.12, 0.55 for columns 3S1, 4S1 and 5S1 respectively. Other adjacent column axis C had CBCR values of 1.55, 1.14 and 0.53 for columns 3S19, 4S19 and 5S19 respectively.

Beam capacities were modified with the CBCR values of smaller than 1.

Table 3.8 : Beam moment modification with CBCR

Beam	**Moment Capacity (bottom / top)**	**CBCR**	**Modified Moment Capacity**	**Shear**
K532	53.0 (end-i) 89.9 (end-j)	0.55 0.62	29.2 55.7	26.0
K533	53.0 (end-i) 89.9 (end-j)	0.62 0.53	32.9 47.6	55.8

Using modified beam capacities and thus shear forces, shear transferred from beams to the columns were recalculated. Only 5th storey CBCR values were smaller than 1, thus axial load value of 5th storey was changed only.

$V_{E,5} = V_{E,5,1} + V_{E,5,2} = 26 - 55.8 =$ **-29.8 kN**.

Axial loads were;

5th storey 5S8 column $N_{E,5} = V_{E,5} = -29.8 =$ **-29.8 kN**

4th storey 4S8 column $N_{E,4} = V_{E,4} + V_{E,5} = -48.5 - 29.8 =$ **-78.3 kN**

3rd storey 3S8 column $N_{E,3} = V_{E,3} + V_{E,4} + V_{E,5} = -46.8 - 48.5 - 29.8 =$ **-125.1 kN** which was -147.8 kN before modification.

$N_D + N_{E,3} = 139.9 – 125.1 = 13.8$ kN (compression)

These axial loads constitute limit values for the axial loads calculated from graphical procedure.

Required parameters for the calculation of axial load by graphical procedure for column 3S8 were given below (Figure 3.15).

N_D	: 139.9 kN	Axial load due to gravity loading.
$M_{D\text{-}bot}$	: -1.31 kNm	Bottom moment due to gravity loading.
$M_{D\text{-}top}$	: 1.53 kNm	Top moment due to gravity loading.
N_E	: -386.3 kN	Axial load of earthquake loading with R=1.
$M_{E\text{-}bot}$	: -253.6 kNm	Bottom moment of earthquake loading with R=1.
$M_{E\text{-}top}$	: 244.7 kNm	Top moment of earthquake loading with R=1.

$N_{K\text{-}bot}$,$M_{K\text{-}bot}$: Moment and axial load capacities calculated from the interaction diagram.

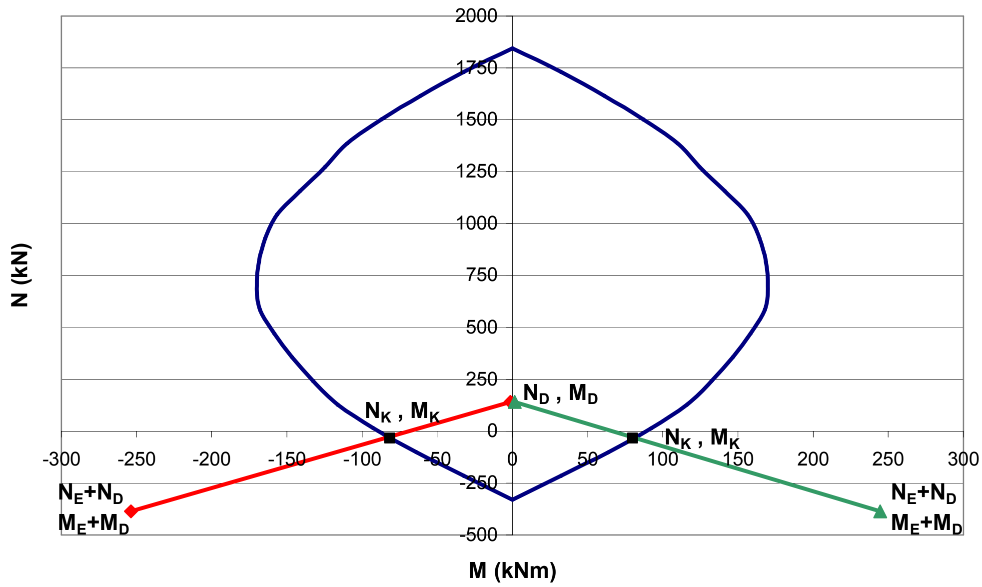
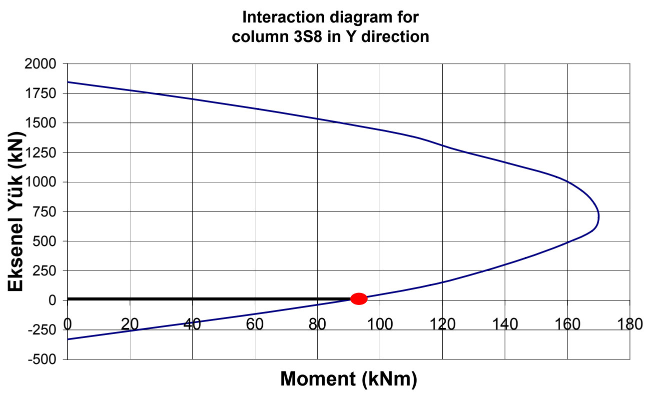
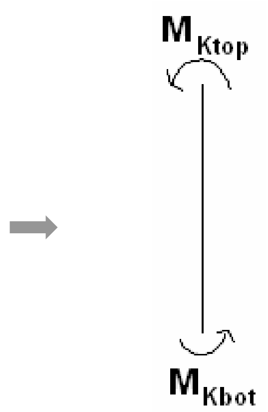


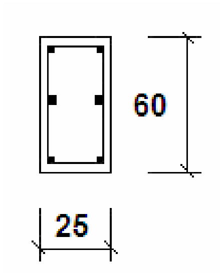
Figure 3.15 : Axial load calculation using graphical procedure

Axial load was computed to be -34 kN (tension) using the graphical procedure which is the intersection point of interaction diagram and lines drawn between gravity and earthquake loading values. According to the rule 7A-3 of the 2007 Turkish Earthquake Code, column axial load was $N_K = N_D + N_{E,3} = 13.8$ kN (compression) which was the limit for the axial load (34 kN) calculated from graphical procedure.

Finally, moment capacity can be calculated using axial load of 13.8 kN and the interaction diagram.







Mid Span transverse reinforcement: Ø8 / 25
Longitudinal reinforcement: 6 Ø14
Support transverse reinforcement : Ø8 / 25

M_{Kbit}	92.4 kNm
M_{Ktop}	92.4 kNm

Figure 3.16 : Column moment capacity

After calculating the moment capacity of members, next step is to compute the shear capacity and shear demand related to flexural strength. Shear capacity for column 3S8 was calculated using the formula given in TS-500 (2000).

$V_r = V_c + V_w = 0.8\ V_{cr} + V_w$ (3.1)

$V_c = 0.8 \times 0.65\ f_{ctm}\, b_w\ d\ (1 + \gamma\ N / A_c)$ (3.2)

= 0.8 x 0.65 x 1.21 x 250 x 585 x (1 + 0.07 x 13.8 x 1000 / (250 x 600))
= 92770 N

$V_w = A_{sw}\ f_{ywm}\ d / s$ = 100.53 x 191 x 585 / 250 = 44930 N

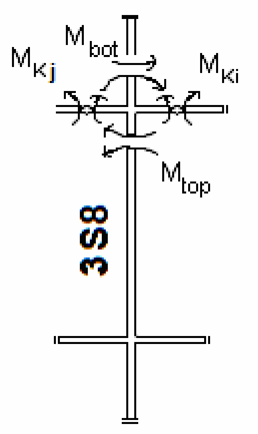
V_r = **137.7 kN**

Remembering that CBCR values are larger than 1 for top and bottom sections of 3S8, top and bottom moments used for the calculation of shear demand are calculated using Equations 3.3 and 3.4.

$$M_{top} = \frac{M_{top}}{M_{top} + M_{bottom}} \sum M_K \qquad (3.3)$$

$$\sum M_K = M_{Ki} + M_{Kj} \qquad (3.4)$$

Where, M_{top} and M_{bot} are the moments induced by the lateral loads and M_{top} represents the moment at the top of column 3S8 and M_{bot} represents the moment at the bottom of column 4S8. M_{Ki} is the moment capacity of the beam joining to the right of the joint at the top of column 3S8 and M_{Kj} is the moment capacity of the beam joining to the left.



M_{top}	244.7 kNm
M_{bot}	-215.1 kNm
M_{Ki}	53.0 kNm
M_{Kj}	89.9 kNm

Figure 3.17 : Calculation of M_{top}

$\Sigma M_K = 53.0 + 89.9 = 144.9$ kNm

$M_{top} = 144.9 \times 244.7 / (244.7 + 215.1) = 77.1$ kNm

Same calculations were done for the bottom end of the column 3S8 and using these moments shear demand were calculated as;

$V_E = (M_{K\ top} + M_{K\ bot}) / l_n$

$V_E = (77.1 + 75.14) / 2.2 =$ **69.2 kN**

$V_E < V_r$ means that column 3S8 is **ductile**.

Similar procedure is applied to the beams and member failure type is defined using the equations for beams. Sample calculation are given for the beam K333.

$V_r = V_c + V_w = 0.8\ V_{cr} + V_w$

$V_c = 0.8x0.65\ f_{ctm}\ b_w\ d = 0.8 \times 0.65 \times 1.21 \times 200 \times 585 = 73764.6$ N,

$V_w = A_{sw}\ f_{ywm}\ d/s = 100 \times 191 \times 585 / 200 = 55867.5$ N,

$V_{ri} = V_{rj} =$ **129.6 kN**

Section shear demand calculation is based on Figure 3.18.

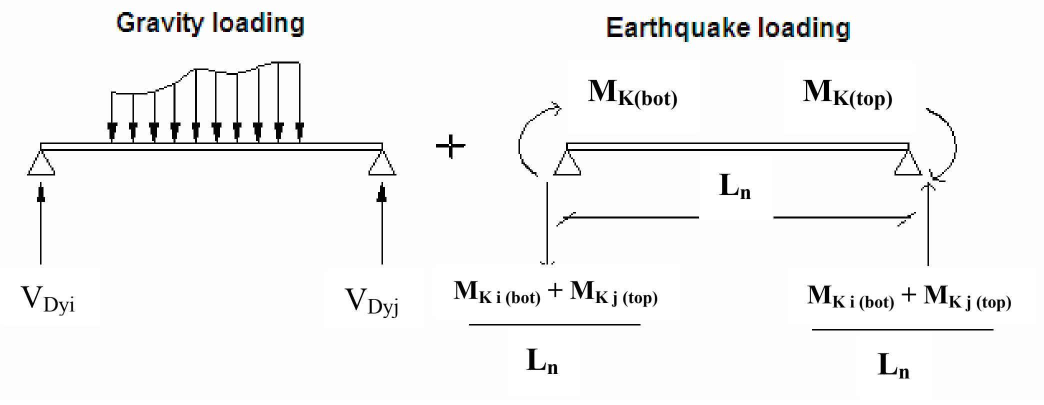


Figure 3.18 : Shear demand calculation of beams

$$V_E = V_{dy} - (M_{Ki,(bot)} + M_{Kj,\,(top)}) / l_n \tag{3.5}$$

V_{Dy} = 13.2 kN, $M_{Ki(bot)}$ =53.0 kNm, $M_{Kj(top)}$ = 89.9 kNm, l_n (clear length) = 1.6 m,

V_{Ei} = **80.68 kN**

Equation 3.5 is $V_E = V_{Dy} + (M_{Ki(bot)} + M_{Kj(top)}) / l_n$ for end j.

V_{Dy} = 2.44 kN, $M_{Ki(bot)}$ = 53.0 kNm, $M_{Kj(top)}$ = 89.9 kNm, l_n =1.6 m

V_{Ej} = **96.33 kN**

For both ends, V_E values are smaller than V_r, thus, beam K333 is evaluated as **ductile**.

3.3.3 Comparison of Demand / Capacity Ratios (r) with Limit Values (r_{Limit})

Parameters that were used for the r_{Limit} definition of column 3S8 from Table 2.2 were:

$$\left.\begin{array}{l} N = 14 \text{ kN} \\ A_c = 1500 \text{ cm}^2 \\ f_{cm} = 12 \text{ MPa} \end{array}\right\} \qquad N_K / (A_c\, f_{cm}) = 0.07$$

$$\left.\begin{array}{l} V = V_E = 69.2 \text{ kN} \\ b_w\, d = 1500 \text{ cm}^2 \\ f_{ctm} = 1.21 \text{ MPa} \end{array}\right\} \qquad V_E / (b_w\, d\, f_{ctm}) = 0.39$$

Column section is not confined.

For Life Safety performance level and the parameters given above, r_{Limit} was found to be **3.5** from Table 2.2. This value was compared with the residual moment capacity of the section for which, M_E at top of the column was 244.7 kNm and 253.6 kNm at the bottom of the column.

$$r_{bottom} = \frac{M_{Ebot}}{M_K - M_D} = \frac{253.6}{92.4 - (1.31)} = 2.8 \qquad \frac{r}{r_{Limit}} = \frac{2.8}{3.5} = 0.79$$

$$r_{top} = \frac{M_{Etop}}{M_K - M_D} = \frac{244.7}{92.4 - (1.54)} = 2.7 \qquad \frac{r}{r_{Limit}} = \frac{2.7}{3.5} = 0.79$$

Both ends of the column 3S8 satisfied the limit of Life Safety thus the member was acceptable according to the Life Safety performance level. If any r / r_{Limit} values is larger than 1 for one end only, that member would not satisfy the performance level considered.

Parameters that were used for the r_{Limit} definition of beam K333 in Table 2.3 were:

End-i

$$\left.\begin{array}{l} \rho = 0.003 \\ \rho' = 0.006 \\ \rho_b = 0.0284 \end{array}\right\} \frac{(\rho - \rho')}{\rho_b} = -0.106$$

$$\left.\begin{array}{l} V = 80.68 \text{ kN} \\ b_w\, d = 1170 \text{ cm}^2 \\ f_{ctm} = 1.21 \text{ MPa} \end{array}\right\} \frac{V_E}{b_w d f_{ctm}} = 0.57$$

End-j

$$\left.\begin{array}{l} \rho = 0.006 \\ \rho' = 0.003 \\ \rho_b = 0.0284 \end{array}\right\} \frac{(\rho - \rho')}{\rho_b} = 0.106$$

$$\left.\begin{array}{l} V = 96.32 \text{ kN} \\ b_w\, d = 1170 \text{ cm}^2 \\ f_{ctm} = 1.21 \text{ MPa} \end{array}\right\} \frac{V_E}{b_w d f_{ctm}} = 0.68$$

Beam section is not confined.

Considering Life Safety performance level and the parameters given above, r_{Limit} was found to be **4** for end-i and by interpolation 3.7 for end-j from Table 2.3. These values were compared with the residual moment capacities of the section for which, M_E at end-i was 274.4 kNm and 253.0 kNm at the end-j.

$$r_i = \frac{M_{Ei}}{M_K - M_D} = \frac{274.4}{53.0 - (-6.93)} = 4.7 \qquad \frac{r}{r_{Limit}} = \frac{4.7}{4} = 1.16$$

$$r_j = \frac{M_{Ej}}{M_K - M_D} = \frac{253.0}{89.9 - (1.68)} = 2.8 \qquad \frac{r}{r_{Limit}} = \frac{2.8}{3.7} = 0.76$$

One end of the beam K333 did not satisfy the limit of Life Safety thus the member was unacceptable according to the Life Safety performance level. If any r / r_{Limit} values is larger than 1 for one end, that member would not satisfy the performance level as well.

r / r_{Limit} values for all members and for all floors are given in Figure 3.19.

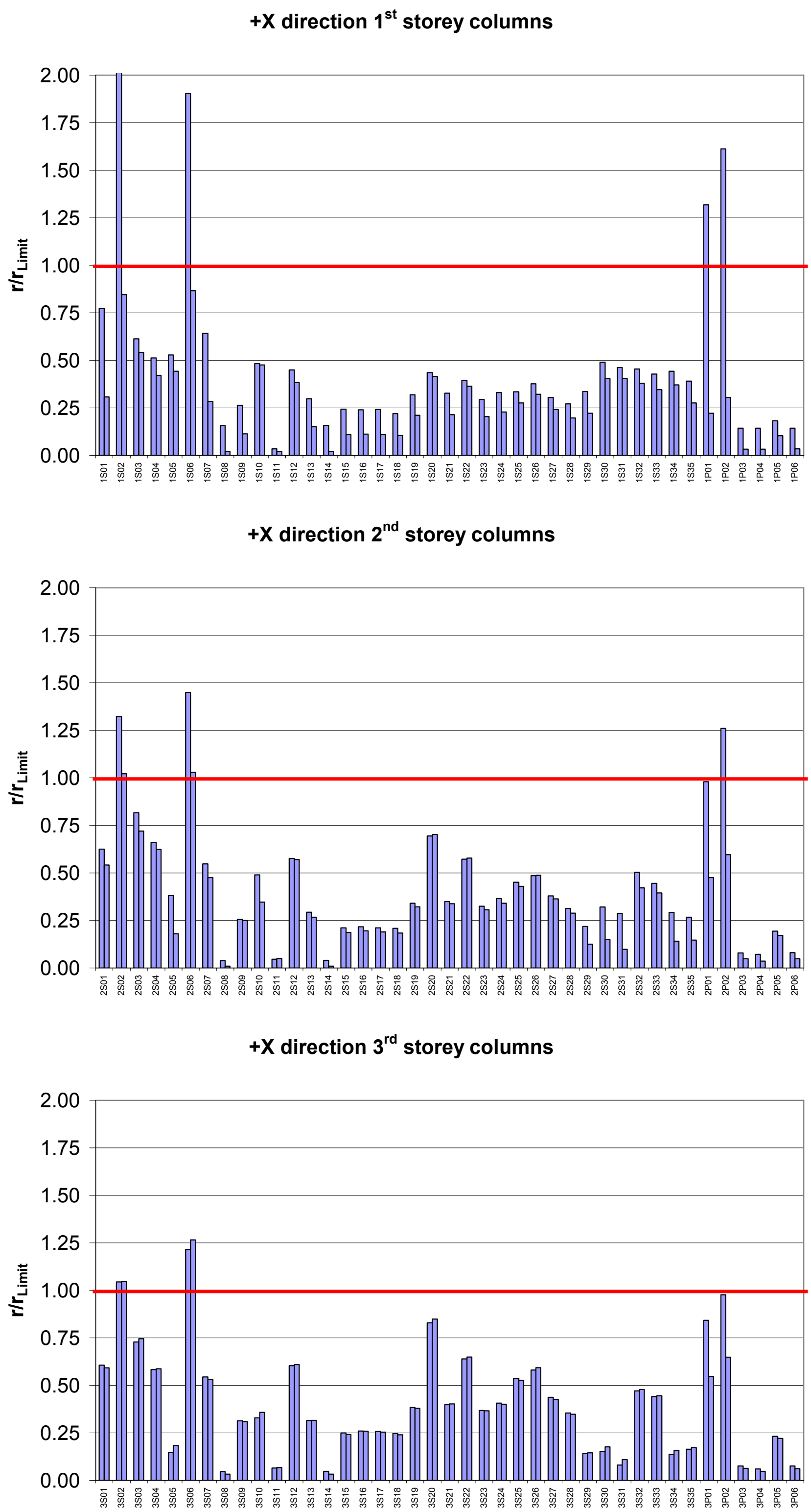
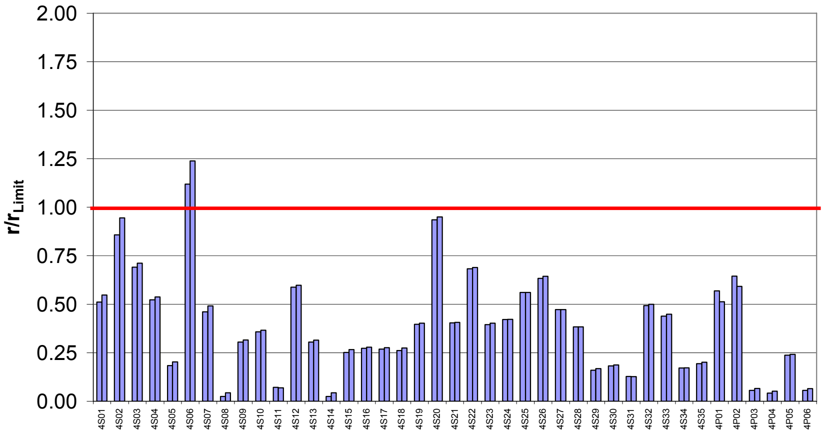
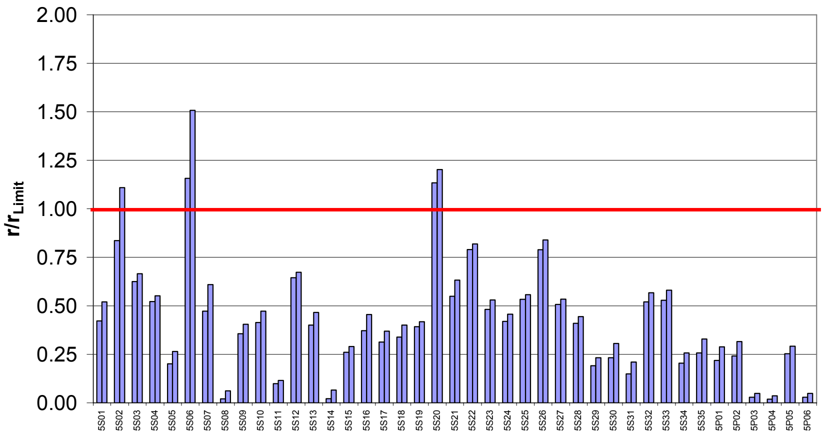


Figure 3.19 : r/r_{Limit} values

+X direction 4th storey columns



+X direction 5th storey columns



+X direction 1st storey beams

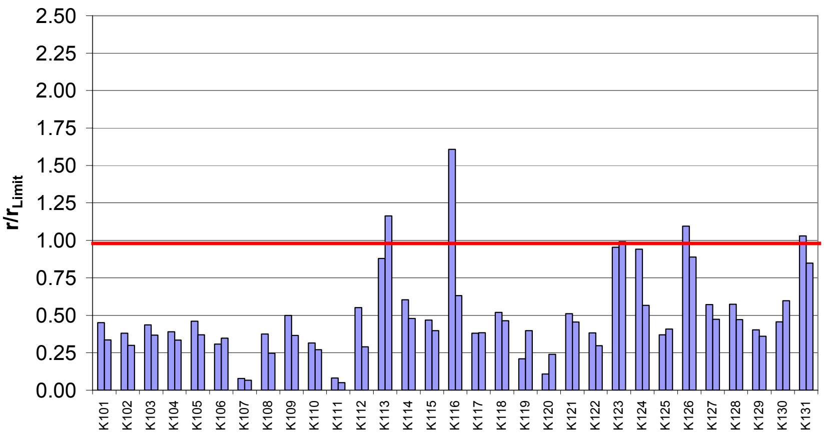
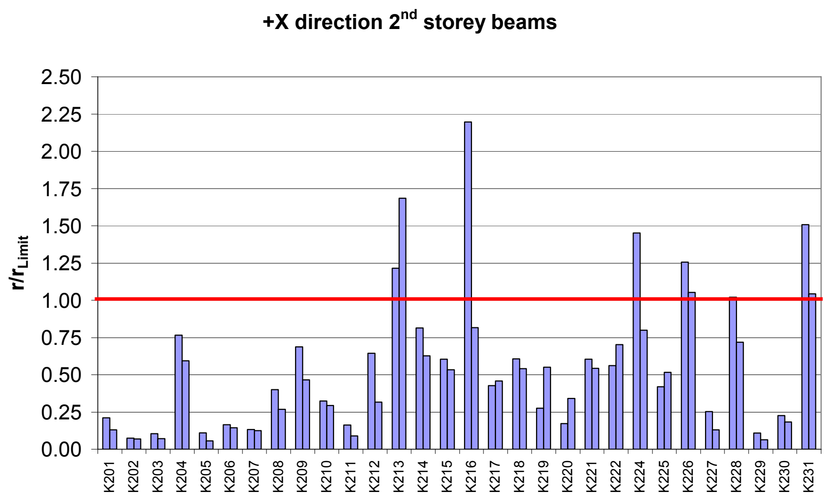
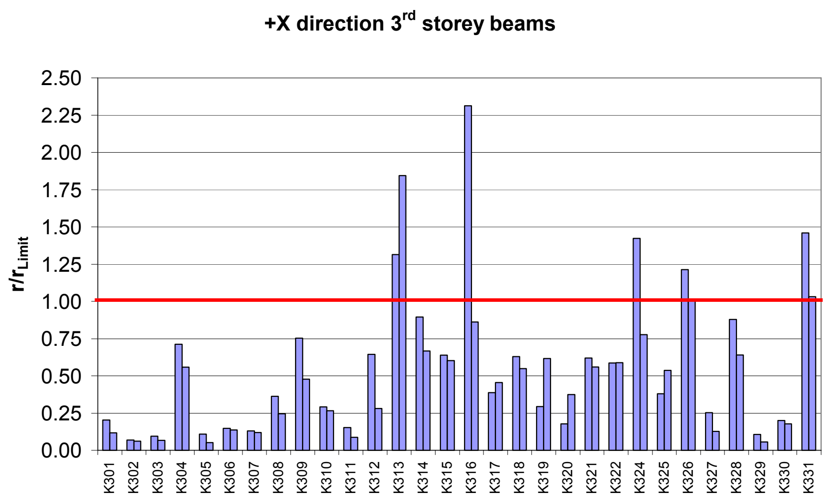


Figure 3.19 : r/r_{Limit} values (continued)





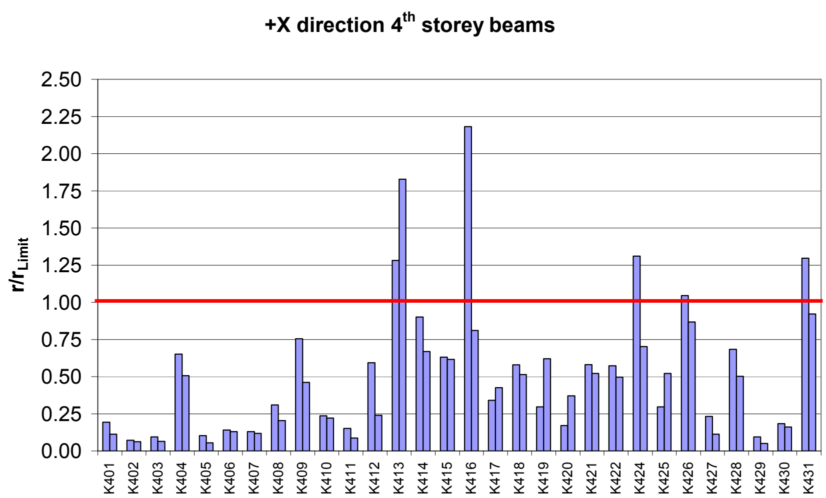
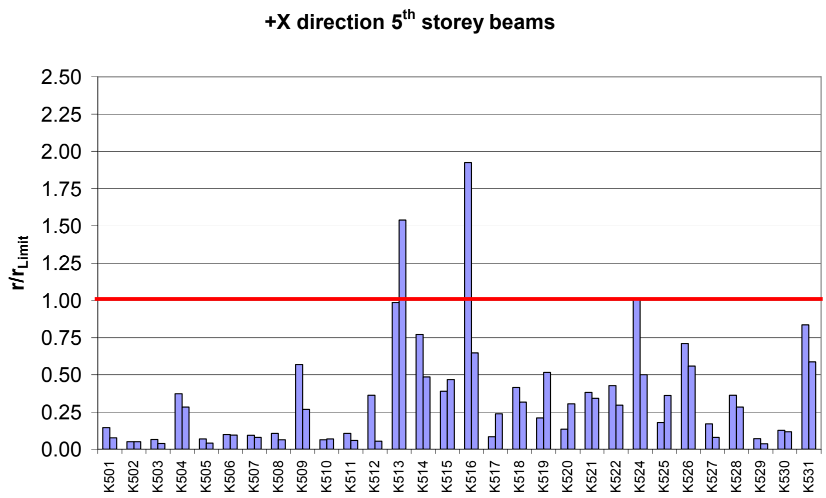
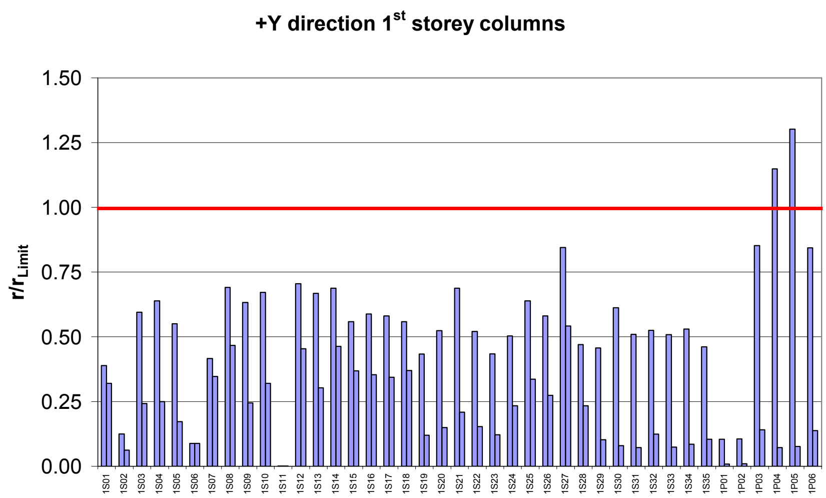


Figure 3.19 : r/r_{Limit} values (continued)





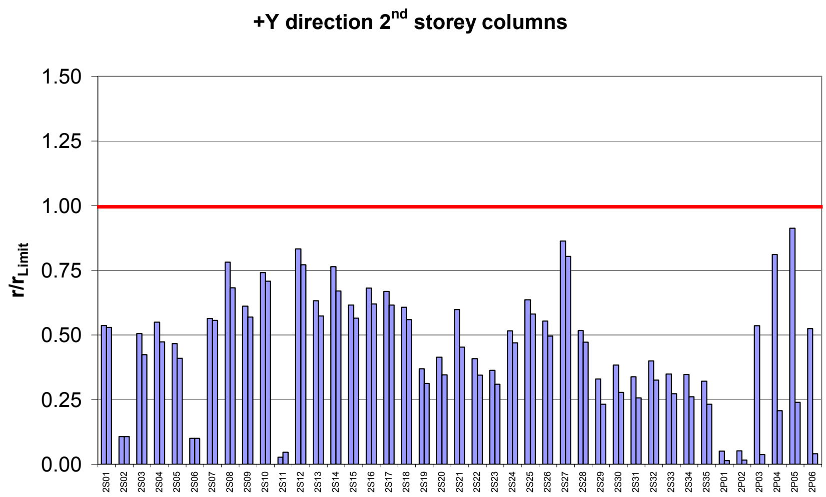


Figure 3.19 : r/r_{Limit} values (continued)

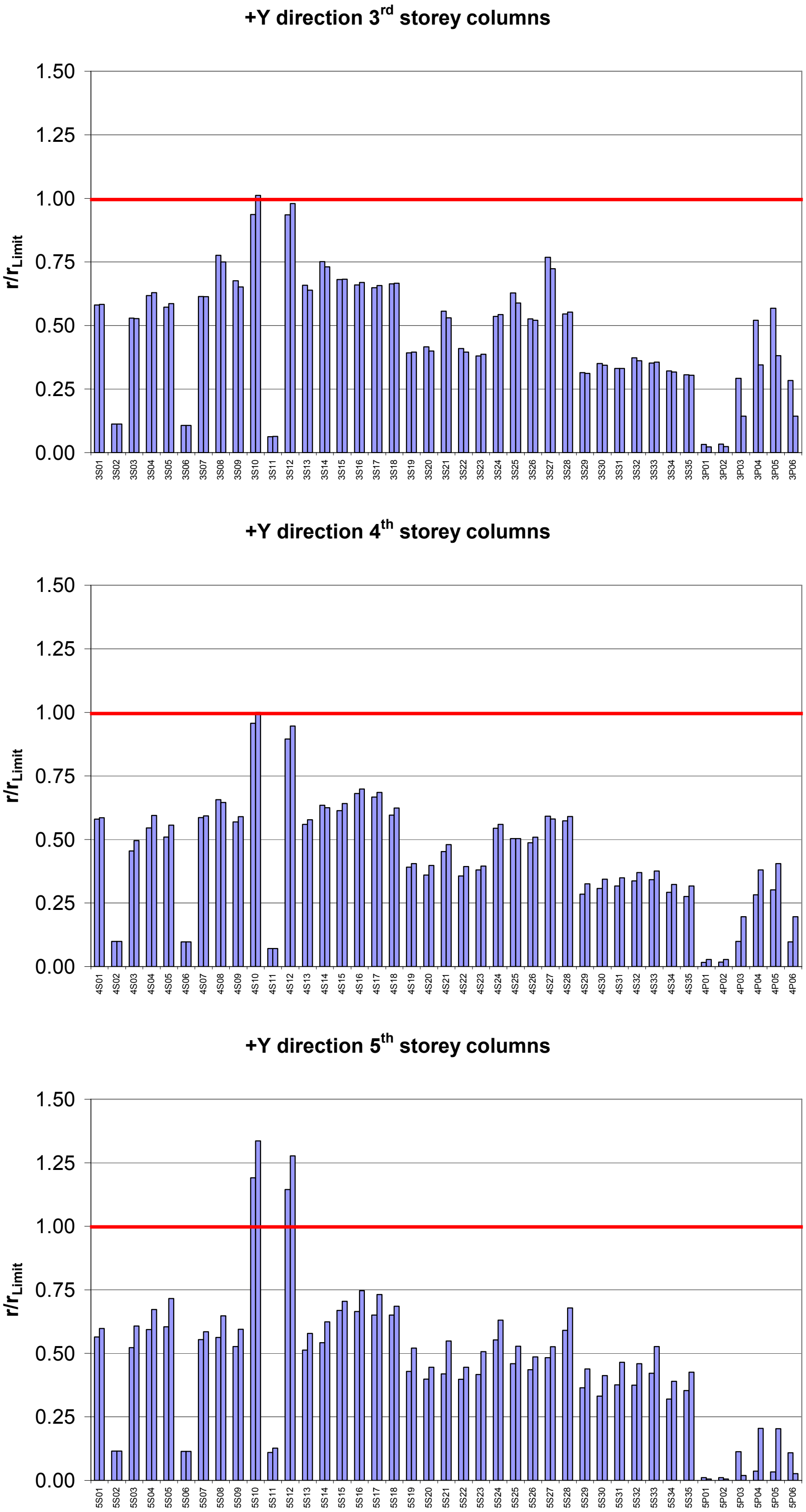
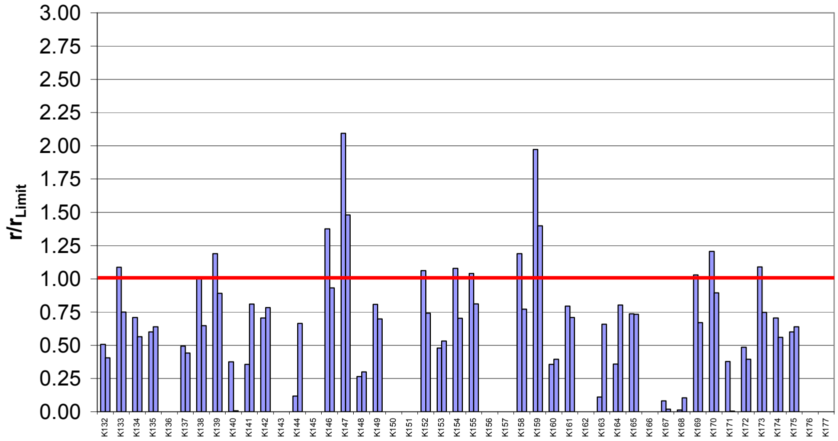
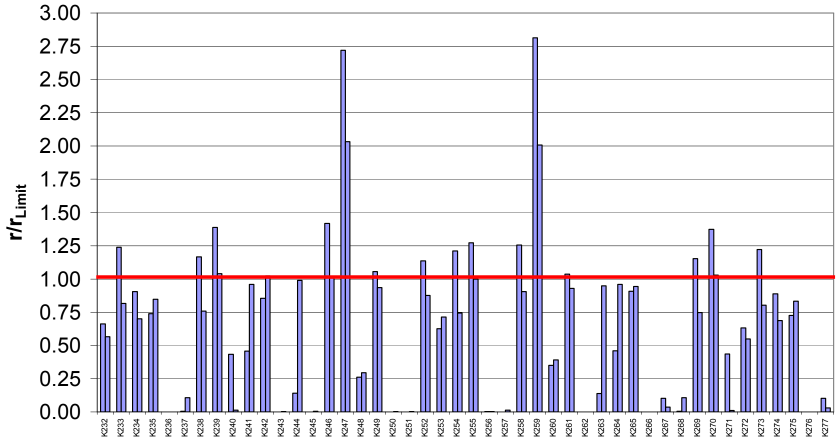


Figure 3.19 : r/r_{Limit} values (continued)

+Y direction 1st storey beams



+Y direction 2nd storey beams



+Y direction 3rd storey beams

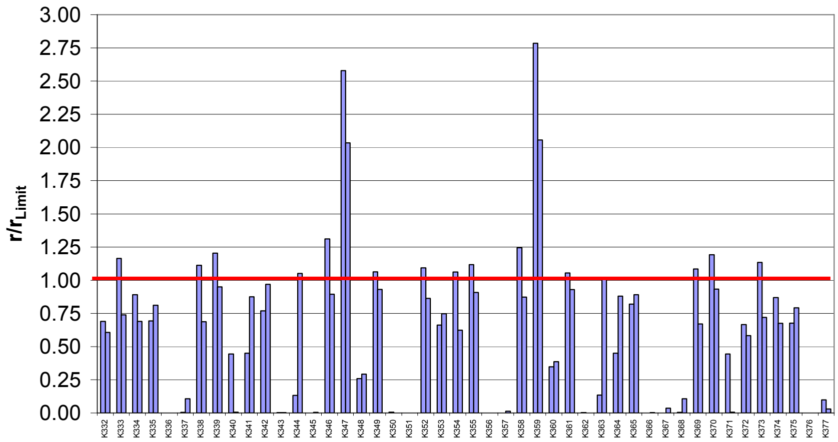
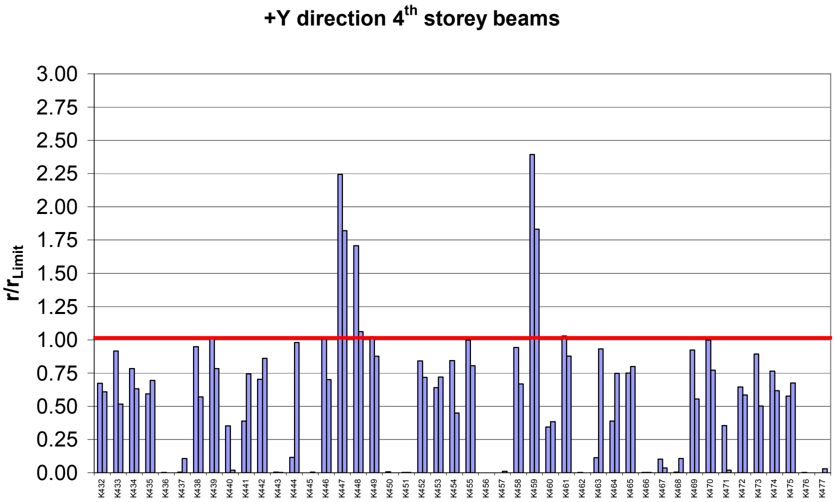


Figure 3.19 : r/r_{Limit} values (continued)



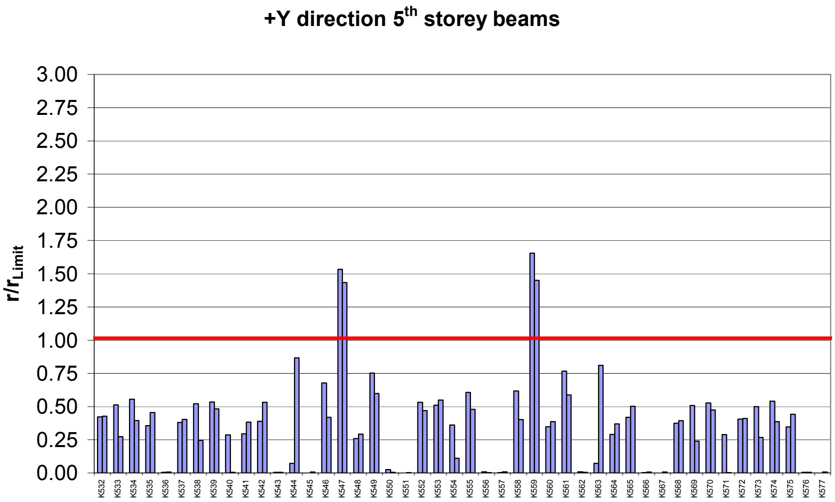


Figure 3.19 : r/r_{Limit} values (continued)

3.3.4 Global performance of the building

In Table 3.9, the ratio of the number of unacceptable beams to all beams in the considered storey and in the considered direction, and the ratio of storey shear taken by the unacceptable members to storey total shear are given. For the values greater than 30% for beams and 20% for columns and walls (40% for top storey), the corresponding storey and the building performance would not be acceptable.

Table 3.9 : Global performance of the members and the ratio of unacceptable members

	+X direction		+Y direction	
St	**Columns and Walls (%)**	**Beams (%)**	**Columns and Walls (%)**	**Beams (%)**
1	**82.4**	10.3	**32.7**	29.4
2	**41.0**	25.0	0.0	**41.2**
3	1.5	25.0	0.0	**47.1**
4	1.7	20.0	0.0	8.8
5	8.3	10.0	11.1	5.9

In the +X direction, the reason for high shear percent in the first two stories is that the shear walls P1 and P2 do not satisfy the life safety performance limits. In addition, columns S02 and S06 are not satisfactory too, which also exhibited damage in actual damage observations.

In the +Y direction, walls 1P4 and 1P5 do not satisfy life safety performance and this results in a high shear percent taken by the unacceptable columns and walls at the first storey. Fifth storey columns 5S10 and 5S12 couple with the walls and are not satisfactory for life safety performance level. Moreover, their section dimensions reduce from 90x25cm to 60x25cm starting from the second storey. Due to the high elastic demands, beams joining to the walls are also not satisfactory.

Calculated interstorey drift results are given in the Table 3.10. Although they are less then 0.03, which is the drift limit for life safety, building global performance was ***not*** satisfactory for life safety performance level.

Table 3.10 : Storey drifts

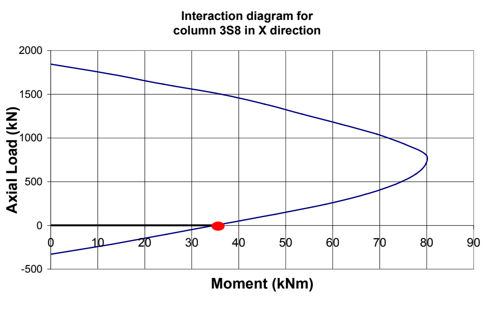
	+X direction			+Y direction		
St	H_i (m)	$(\Delta_i)_{max}$	$(\Delta_i)_{max} / H_i$	H_i (m)	$(\Delta_i)_{max}$	$(\Delta_i)_{max} / H_i$
1	2.8	0.012211	0.004361	2.8	0.006170	0.002204
2	2.8	0.019584	0.006994	2.8	0.010575	0.003777
3	2.8	0.021745	0.007766	2.8	0.012066	0.004309
4	2.8	0.020884	0.007459	2.8	0.011752	0.004197
5	2.8	0.018103	0.006465	2.8	0.010391	0.003711

3.4 Nonlinear Analysis Procedure

Three-dimensional model, storey masses and inertia, storey structural plans and rigid end zones are the same with the linear elastic model.

3.4.1 Modeling and Analysis

Moment-curvature relation was obtained and converted to moment-plastic rotation relation as shown in Figure 3.20. Here φ_p represents plastic curvature and using plastic curvature, plastic rotation is obtained.



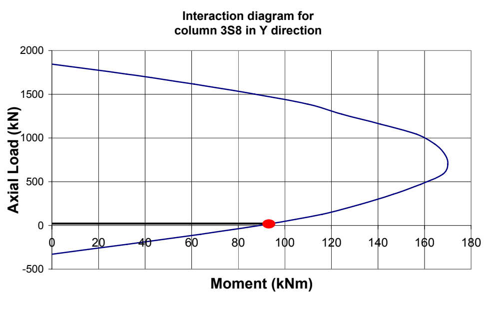


Figure 3.20 : Curvature to plastic rotation transformation

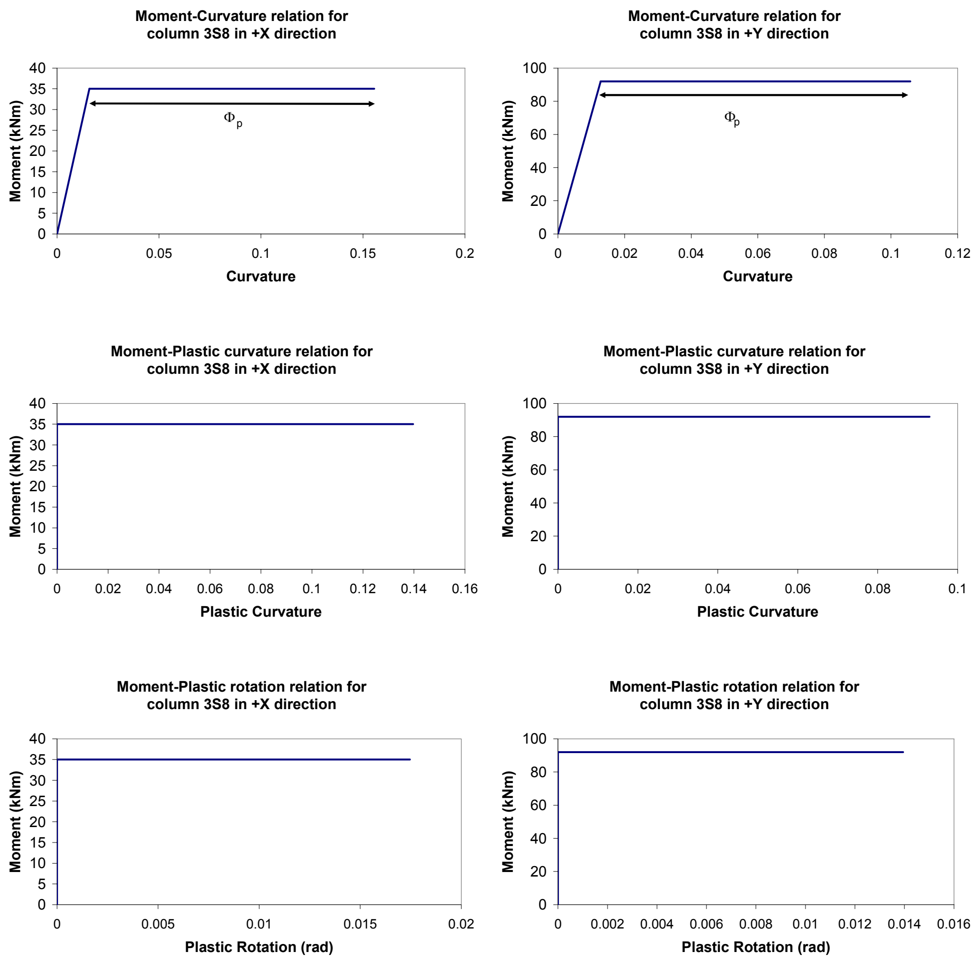


Figure 3.20 : Curvature to plastic rotation transformation (continued)

Same transformation was done for positive and negative moments of all beam and all column ends. In addition, interaction surface was calculated for column members and assigned to the model.

Before starting the assessment process, building model must be controlled for the appropriateness of the equivalent lateral load procedure. Building has;

Total height of 14m which is less than 25m.

Total number of floors of 5 which is less than 8.

Maximum torsional irregularity coefficient as shown in Table 3.11 (η_{bi}) was 1.1, which is less than 1.4.

Mass participation ratios were 76% in the Y direction whereas 41% in the X direction which is smaller than 70%, the limit for single mode analysis. Therefore, multi-mode pushover procedure is needed to be applied for +X direction. Sum of modal mass participation ratios of first five modes is larger than 90% hence, first five modes were used for multi-mode pushover. On the other hand, equivalent lateral load procedure can be applied in the +Y direction. Pushover curves of third mode, which was used for the +Y direction analysis, and first and second modes, which were used for the modal combination of +X direction, are shown in Figure 3.21 and Figure 3.22.

Table 3.11 : Torsional irregularity check

	+X				+Y			
Storey	**(Δ_i)max**	**(Δ_i)min**	**(Δ_i)mean**	**η_b**	**(Δ_i)max**	**(Δ_i)min**	**(Δ_i)mean**	**η_b**
1	0.0134	0.0051	0.0093	1.4477	0.0131	0.0125	0.0128	1.0223
2	0.0162	0.0044	0.0103	1.5711	0.0149	0.0145	0.0147	1.0127
3	0.0162	0.0044	0.0103	1.5769	0.0148	0.0146	0.0147	1.0065
4	0.0156	0.0048	0.0102	1.5308	0.0142	0.0142	0.0142	1.0008
5	0.0146	0.0052	0.0099	1.4786	0.0134	0.0132	0.0133	1.0048

Performing nonlinear static analysis, capacity curve (pushover curve) for the first three modes were obtained and shown in Figure 3.21 to Figure 3.22.

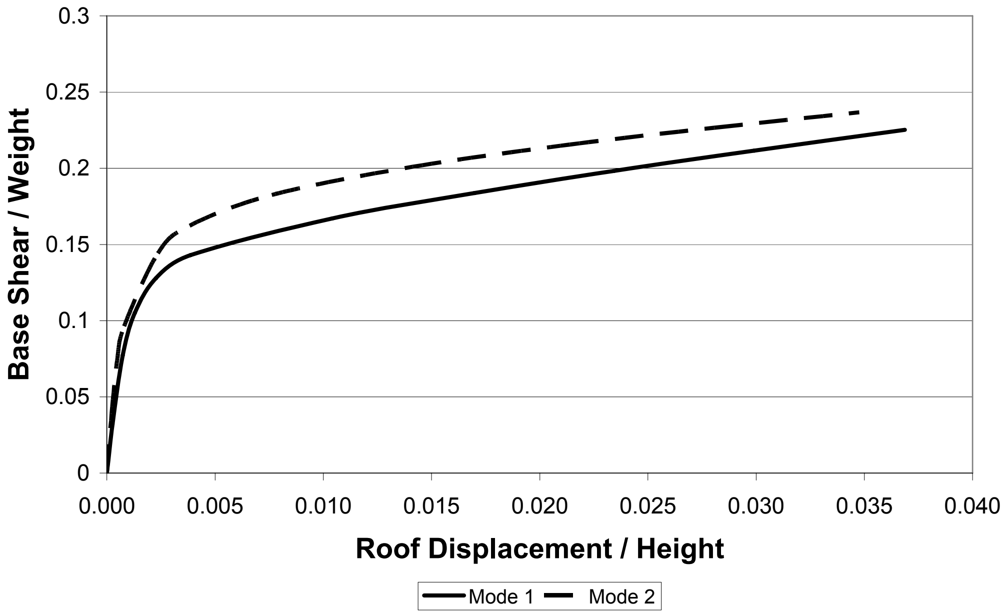


Figure 3.21 : Capacity curves of mode 1 and mode 2 in the +X direction

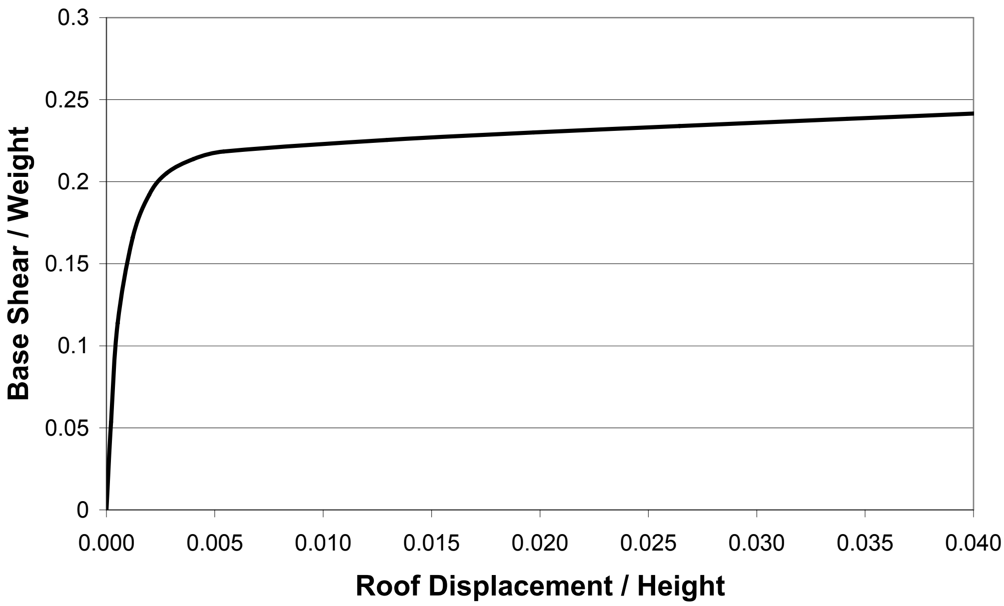


Figure 3.22 : Capacity curve of mode 3 in the +Y direction

3.4.2 Calculation of Target Displacement

Target displacement calculations for the first three modes of the building were performed under Düzce earthquake time history by the inelastic analysis of a Single Degree Of Freedom (SDOF) system. North-South and East-West components in the Düzce earthquake were applied to the +X and +Y directions respectively. Ground accelerations used in the analysis are illustrated in Figure 3.23 and Figure 3.24.

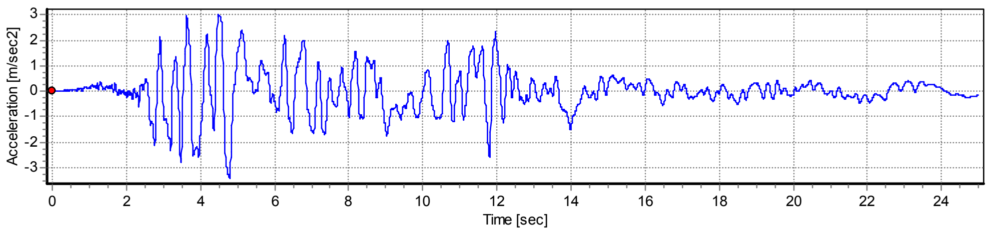


Figure 3.23 : North-South component of the Düzce earthquake

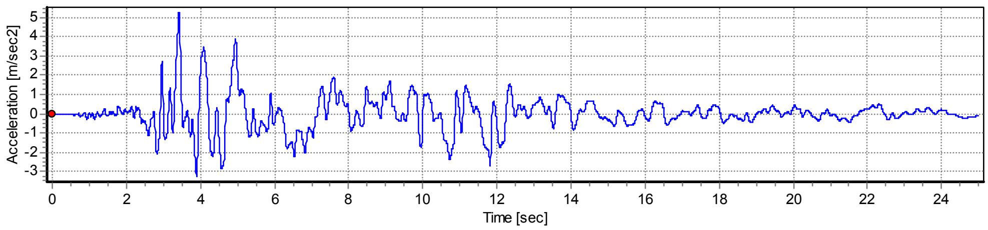


Figure 3.24 : East-West component of the Düzce earthquake

Calculated target displacements were 0.058m for mode 1 (X), 0.05m for mode 2 (X) and 0.07m for mode 3 (Y).

Mass modal participation ratio of Mode 3 is high enough for a single mode analysis in the +Y direction but mode superposition was applied for the +X direction. Modal components were combined with the CQC rule.

3.4.3 Member capacities

Member shear capacity formulations are the same with linear elastic procedure with only one difference. Axial load needed to calculate shear capacity of column sections were obtained from the pushover analysis. Moreover, shear demands were obtained from pushover analysis for both beam and column sections. By the comparison of shear demands with shear capacities, member failure type is defined as ductile or brittle.

If the member is brittle then demand to capacity ratio is shear demand over shear capacity. On the other hand, if the member is ductile, then concrete and steel strains must be calculated to compare them with the strain limits of the corresponding performance level.

Total curvature can be calculated from plastic rotations, and strain values for concrete and steel are found using a section analysis. A sample calculation is given below for unconfined column 1S8.

$\theta_p = 0.004193$ rad

$$\phi_p = \frac{\theta_p}{L_p} = \frac{0.004193}{0.6 * 0.5} = 0.01398 \text{ 1/m}$$

$$\phi_t = \phi_y + \phi_p = 0.00357 + 0.01398 = 0.017547$$

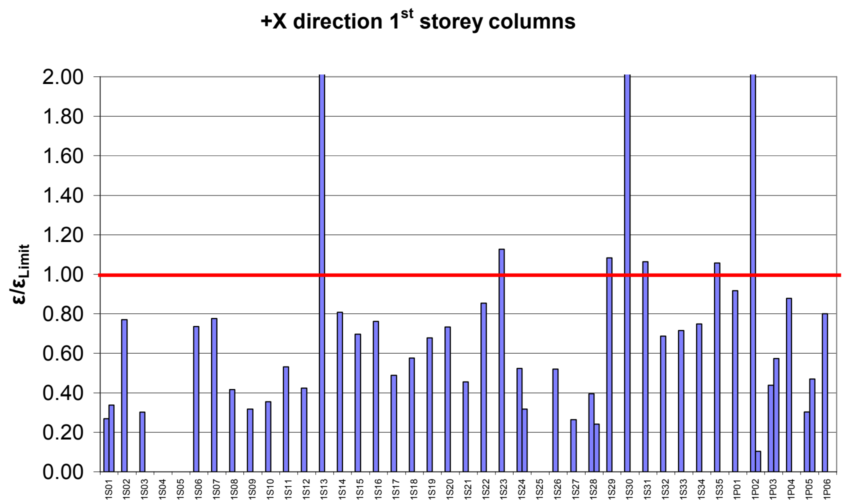
Strain values for total curvature were found as;

$\varepsilon_c = 0.00179$ $\varepsilon_{c(LS)} = 0.0035$ → $\varepsilon_c / \varepsilon_{cg(LS)} = 0.51$

$\varepsilon_s = 0.008606$ $\varepsilon_{s(LS)} = 0.040$ → $\varepsilon_s / \varepsilon_{s(LS)} = 0.22$

Strain to strain limit ratio for the section is the larger of concrete and steel ratios, which was 0.51. Upper end of the column 1S8 was not yielded so no calculation is needed for that end. Strain over strain limit values were smaller than 1 for both ends of the column thus 1S8 was evaluated as acceptable for performance level of Life Safety.

Same calculation and evaluation process was done for all members and acceptability of the members were defined. $\varepsilon / \varepsilon_{Limit}$ values for all members and for all stories are given in Figure 3.25 - Figure 3.27.



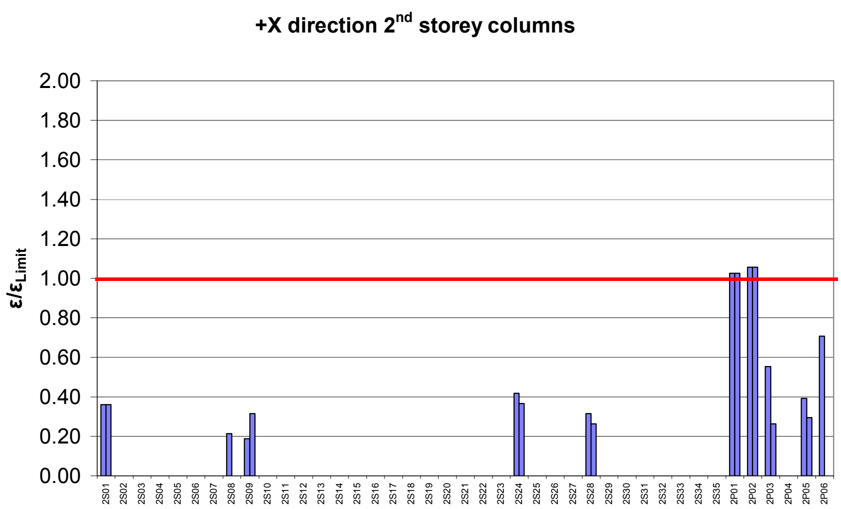


Figure 3.25 : $\varepsilon / \varepsilon_{Limit}$ values

All $\varepsilon / \varepsilon_{Limit}$ values of columns and walls in the +X direction at stories 3, 4 and 5 are smaller than 1, so they are not shown here.

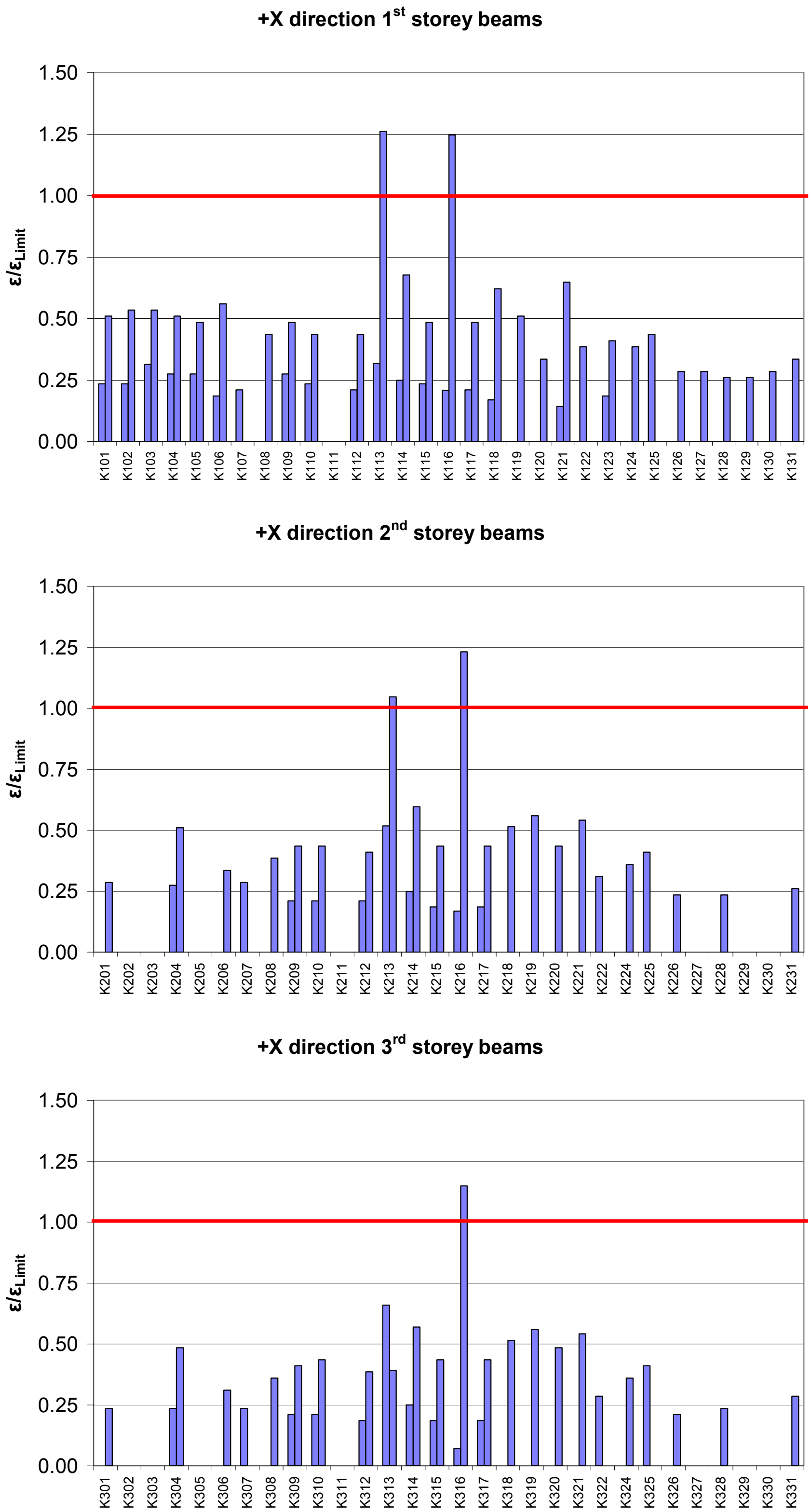
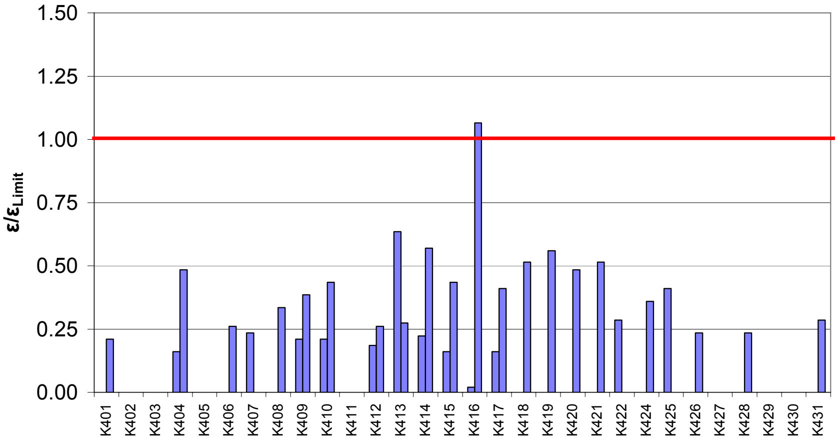
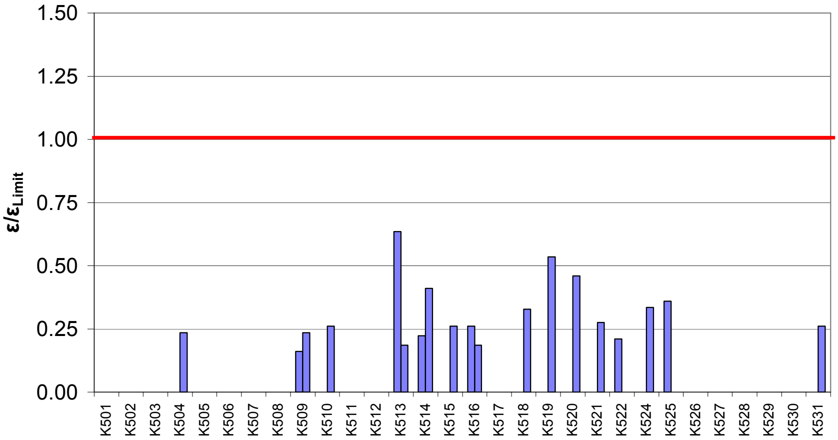


Figure 3.26 : $\varepsilon / \varepsilon_{Limit}$ values

+X direction 4th storey beams



+X direction 5th storey beams



+Y direction 1st storey columns

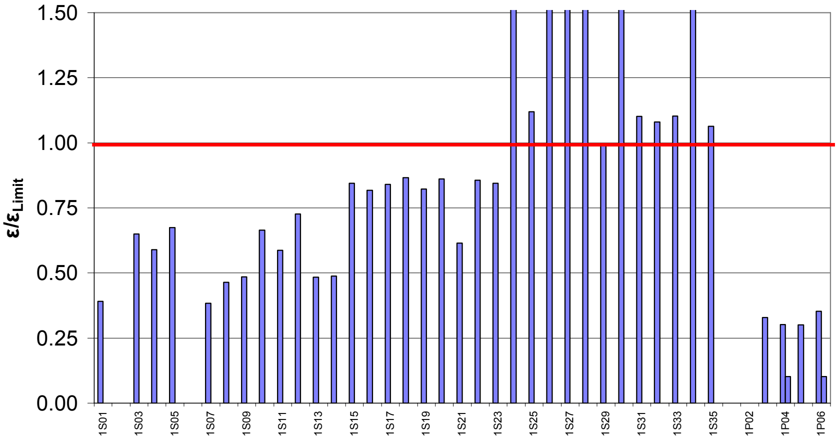


Figure 3.26 : $\varepsilon / \varepsilon_{Limit}$ values (continued)

All $\varepsilon / \varepsilon_{Limit}$ values of columns and walls in the +Y direction at stories 2, 3, 4 and 5 are smaller than 1, so they are not shown here.

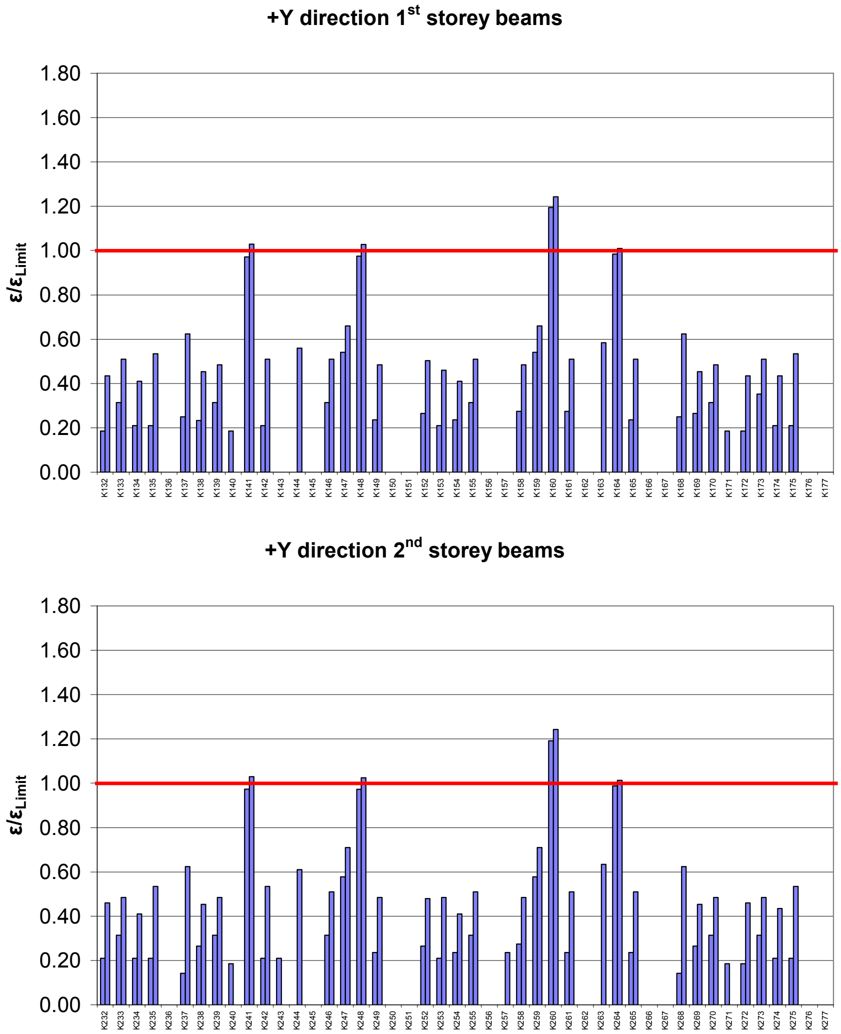
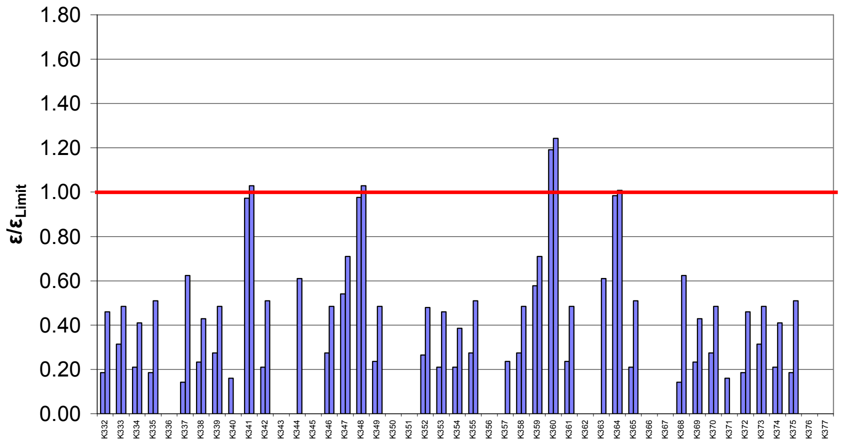
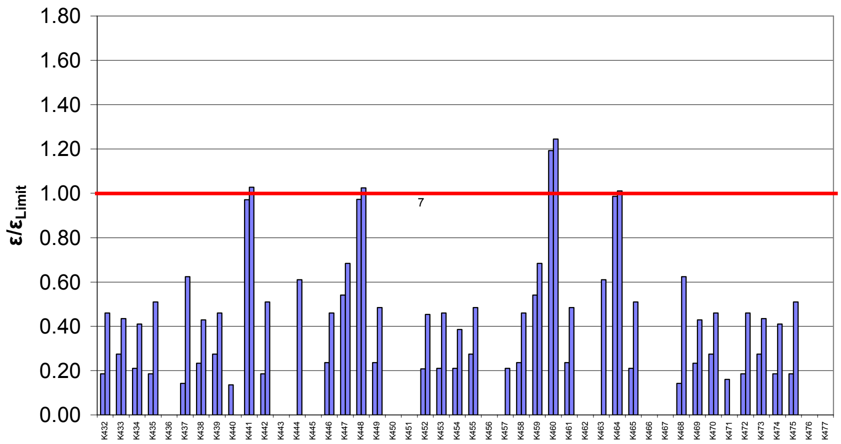


Figure 3.27 : $\varepsilon / \varepsilon_{Limit}$ values

+Y direction 3rd storey beams



+Y direction 4th storey beams



+Y direction 5th storey beams

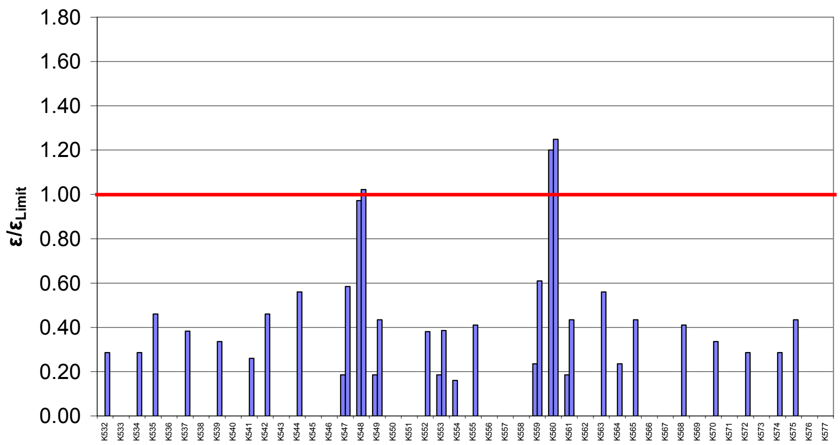


Figure 3.27 : $\varepsilon / \varepsilon_{Limit}$ values (continued)

3.4.4 Global Performance of the Building

Global performance evaluation of the building for all stories and for two orthogonal directions are given in Table 3.12.

Table 3.12 : Global performance of the members and the ratios of unacceptable members

	+X direction		+Y direction	
St	**Columns and Walls (%)**	**Beams (%)**	**Columns and Walls (%)**	**Beams (%)**
1	**23.3**	6.9	**30.0**	11.8
2	**67.9**	10.0	0.0	11.8
3	0.0	5.0	0.0	11.8
4	0.0	5.0	0.0	11.8
5	0.0	0.0	0.0	5.9

High column shear percents in both directions are due to the yielding walls which are dominant in shear resistance.

Calculated interstorey drift results were given in the Table 3.13. Although they are within the limits of life safety, building global performance was ***not*** satisfactory for life safety performance level.

Table 3.13 : Storey drifts

	+X direction			+Y direction		
St	**H_i (m)**	**$(\Delta_i)_{max}$**	**$(\Delta_i)_{max} / H_i$**	**H_i (m)**	**$(\Delta_i)_{max}$**	**$(\Delta_i)_{max} / H_i$**
1	2.8	0.013419	0.004793	2.8	0.013065	0.004666
2	2.8	0.016162	0.005772	2.8	0.014870	0.005311
3	2.8	0.016249	0.005803	2.8	0.014781	0.005279
4	2.8	0.015643	0.005587	2.8	0.014188	0.005067
5	2.8	0.014640	0.005229	2.8	0.013366	0.004774

CHAPTER IV

CASE STUDY 2 : BUILDING IN CASE STUDY 1 RETROFITTED WITH INTERIOR SHEAR WALLS

In this case study, the five-storey existing residential building in Case Study 1 which was retrofitted with interior shear walls and new columns is assessed using both linear elastic and nonlinear procedures described in the 2007 Turkish Earthquake Code.

4.1 Description of the Retrofit Option

Structural properties of the building were defined in the previous chapter. Shear walls were added to the system in both orthogonal directions. Two new columns were also constructed. Structural properties and associated parameters for the 2007 Turkish Earthquake Code are tabulated in Table 4.1 and location of the shear walls are illustrated in Figure 4.2. Three dimensional view of the retrofitted building is shown in Figure 4.1.

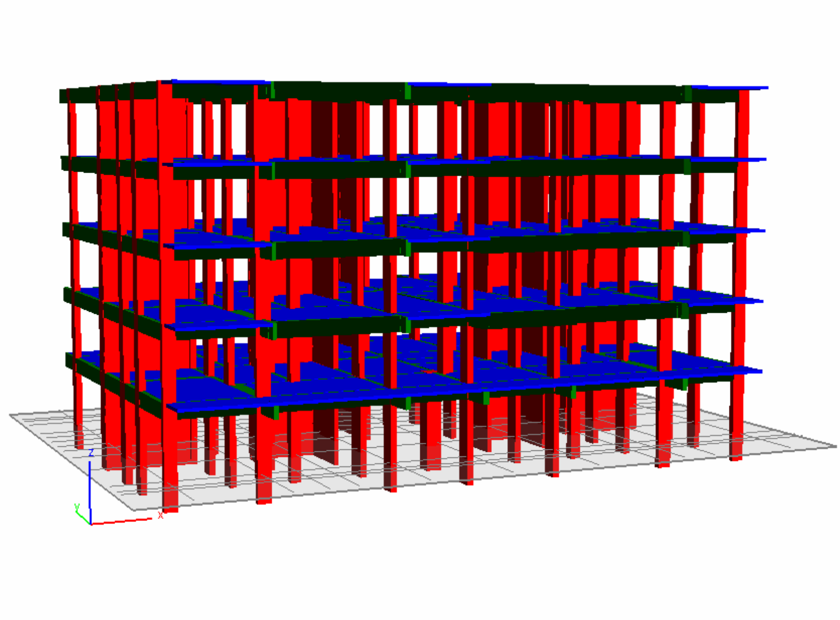


Figure 4.1 : 3D model of the retrofitted building

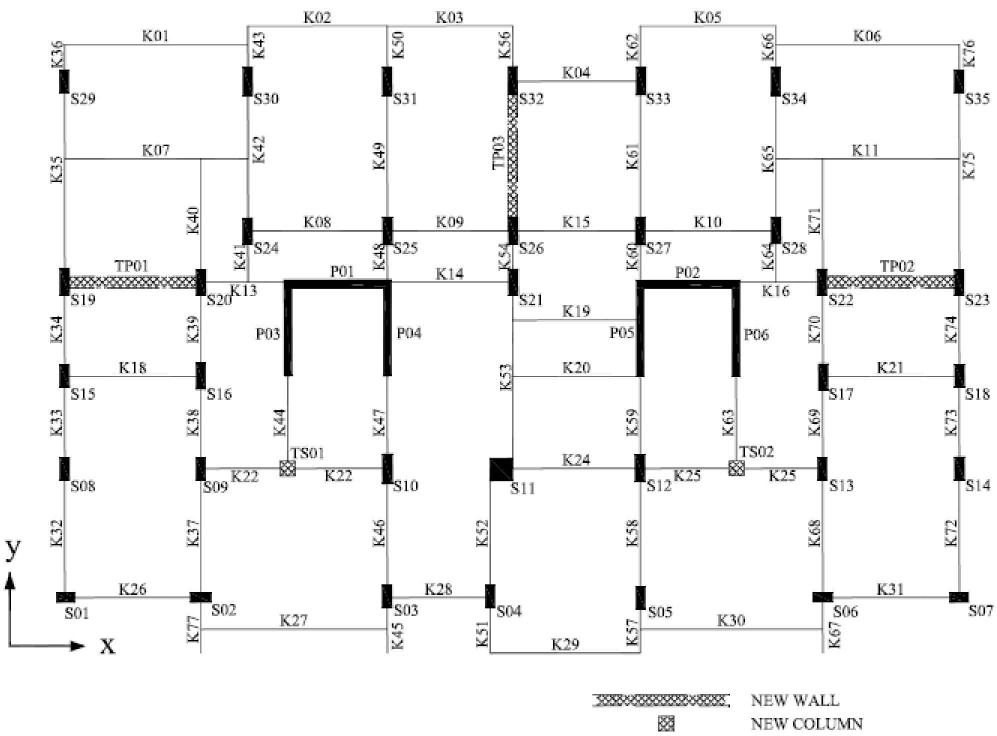


Figure 4.2 : Typical floor plan of retrofitted system

Table 4.1 : Structural system properties and Code parameters

Building Properties	
Project available?	Yes
Knowledge level	Extensive
Knowledge level factor	1
Reinforcement establishment factor	1
Existing concrete strength	12 MPa (E_c = 25000 MPa)
Retrofitting concrete strength	25 MPa (E_c = 30250 MPa)
Existing reinforcement strength	220 MPa
Retrofitting reinforcement strength	420 MPa

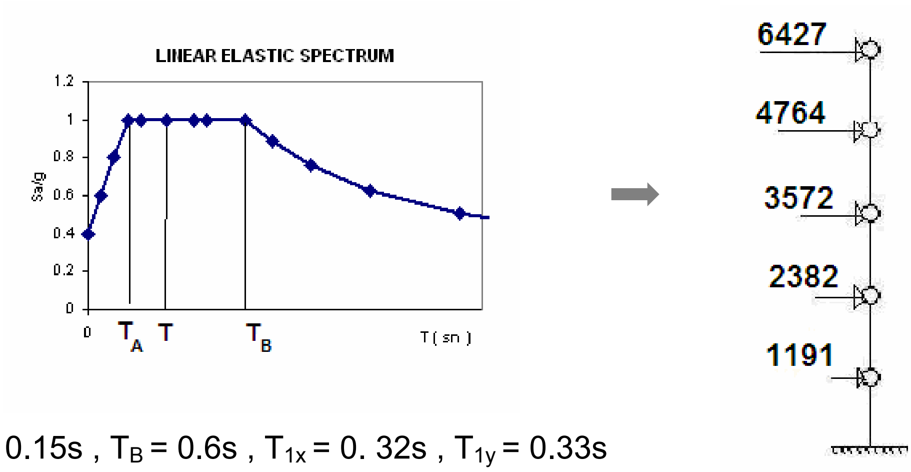
2007 Turkish Earthquake Code Parameters	
Seismic Zone	1
Seismic Zone Factor (A_o)	0.4
Building Importance Factor	1
Soil Class	Z3
Live Load Participation Factor	0.3
Target Performance Level	Life Safety

Section dimensions were given in Chapter 3. The dimensions for shear walls are 370x30cm for walls in X direction, 350x25cm for the wall in Y direction. New columns are square in shape and dimensions are 40x40cm. New walls are named as TP and new columns are labeled as TS. Modal properties of the retrofitted building are given in Table 4.2.

First mode periods are illustrated with the Code spectrum in Figure 4.3

Table 4.2 : Modal properties of the retrofitted building

MODE	PERIOD	INDIVIDUAL MODE (PERCENT)		CUMULATIVE SUM (PERCENT)	
		UX	UY	UX	UY
1	0.459936	3.5139	0.0002	3.5139	0.0002
2	0.329194	0.3932	75.0998	3.9072	75.0999
3	0.321182	69.9770	0.4278	73.8842	75.5278
4	0.140683	0.5478	0.0000	74.4320	75.5278
5	0.089059	0.0005	16.6836	74.4325	92.2114



$T_A = 0.15s$, $T_B = 0.6s$, $T_{1x} = 0.32s$, $T_{1y} = 0.33s$

Figure 4.3 : Code spectrum and lateral load distribution

Equivalent lateral load distribution according to the Code spectrum are shown in Table 4.3.

Table 4.3 : Equivalent static lateral load distribution

St	Weight (W_i) (kN)	Height (m)	H_i (m)	W_i H_i (kNm)	F_i (kN)
1	4345.83	2.8	2.8	12168.32	1190.921
2	4345.83	2.8	5.6	24336.65	2381.842
3	4345.83	2.8	8.4	36504.97	3572.763
4	4345.83	2.8	11.2	48673.3	4763.683
5	4188.87	2.8	14	58644.18	6427.153

Torsional irregularity check has been done before starting the assessment procedure and retrofitted system was found suitable for the equivalent static lateral load method.

Table 4.4 : Torsional irregularity check

	+X				+Y			
Storey	**(Δ_i)max**	**(Δ_i)min**	**(Δ_i)mean**	**η_b**	**(Δ_i)max**	**(Δ_i)min**	**(Δ_i)mean**	**η_b**
1	0.0065	0.0041	0.0053	1.2225	0.0057	0.0056	0.0056	1.0099
2	0.0113	0.0075	0.0094	1.2018	0.0097	0.0096	0.0097	1.0048
3	0.0133	0.0094	0.0113	1.1712	0.0112	0.0112	0.0112	1.0004
4	0.0132	0.0101	0.0117	1.1365	0.0110	0.0110	0.0110	1.0008
5	0.0121	0.0100	0.0110	1.0948	0.0099	0.0099	0.0099	1.0012

4.2 Linear Elastic Procedure

Linear elastic procedure calculations were represented in detail in Chapter 3, here only the r/r_{Limit} values and global performance of the structure will be introduced.

4.2.1 Comparison of Demand / Capacity Ratios (r) with Limit Values (r_{Limit})

Limit values were obtained from Table 2.2 for columns, Table 2.3 for beams and Table 2.4 for shear walls respectively with the calculated parameters. Normalized values of r / r_{Limit} are illustrated in Figure 4.4.

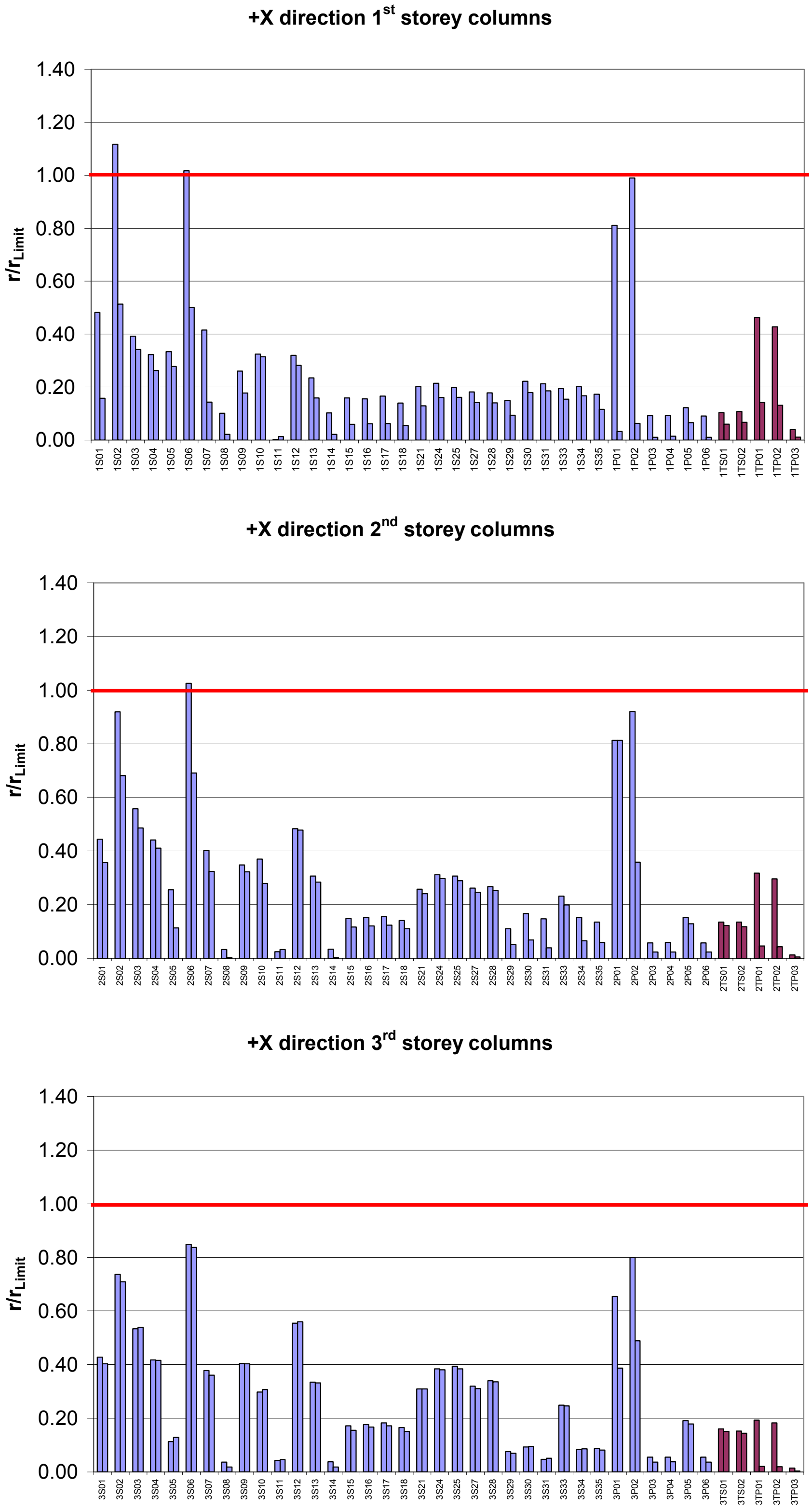
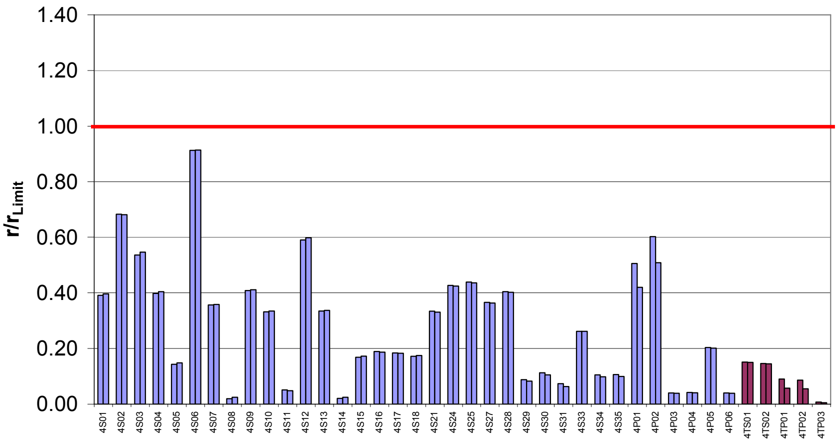
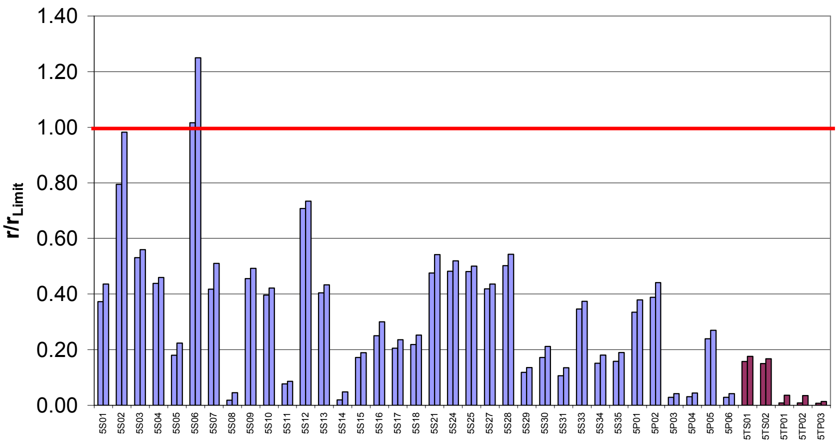


Figure 4.4 : r/r_{Limit} values

+X direction 4th storey columns



+X direction 5th storey columns



+X direction 1st storey beams

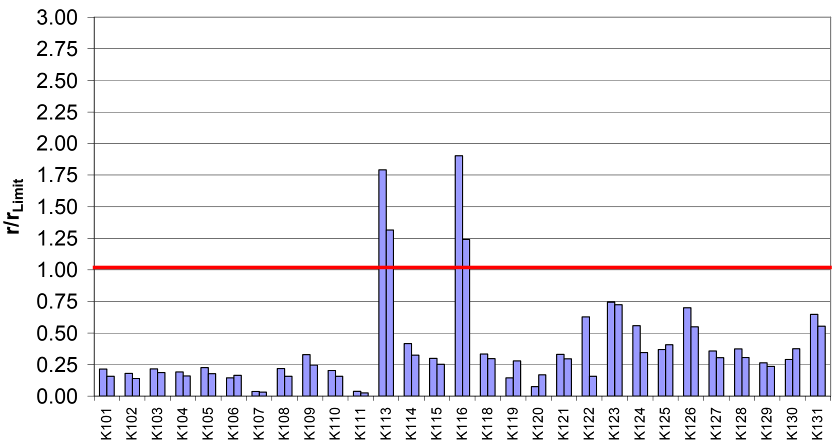
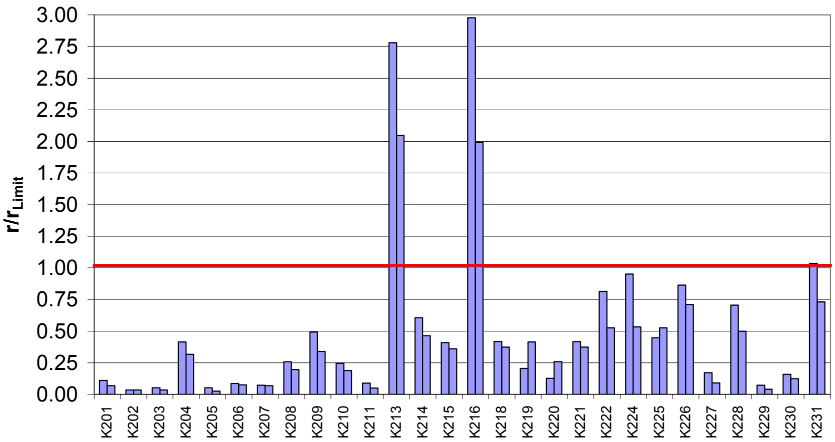
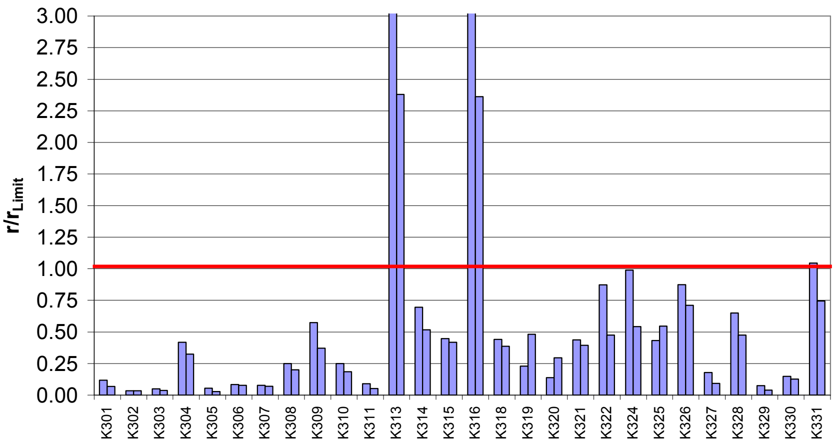


Figure 4.4 : r/r$_{Limit}$ values (continued)

+X direction 2nd storey beams



+X direction 3rd storey beams



+X direction 4th storey beams

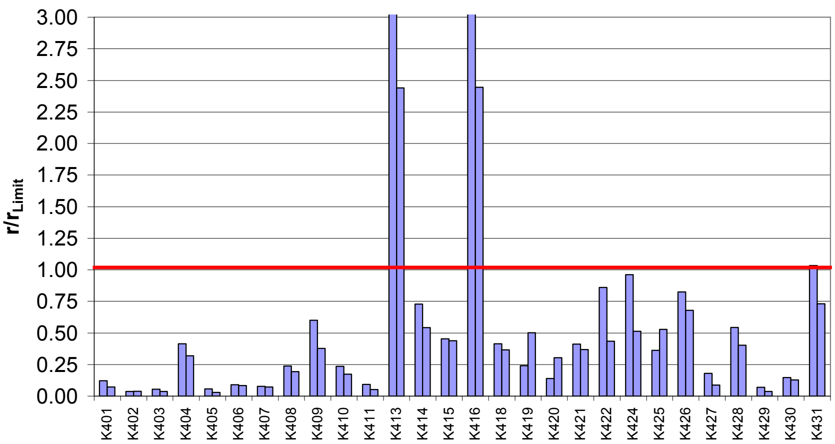
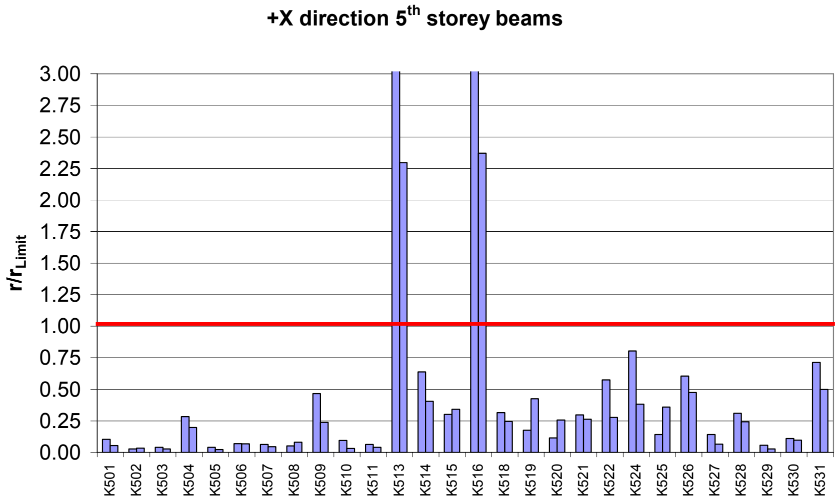
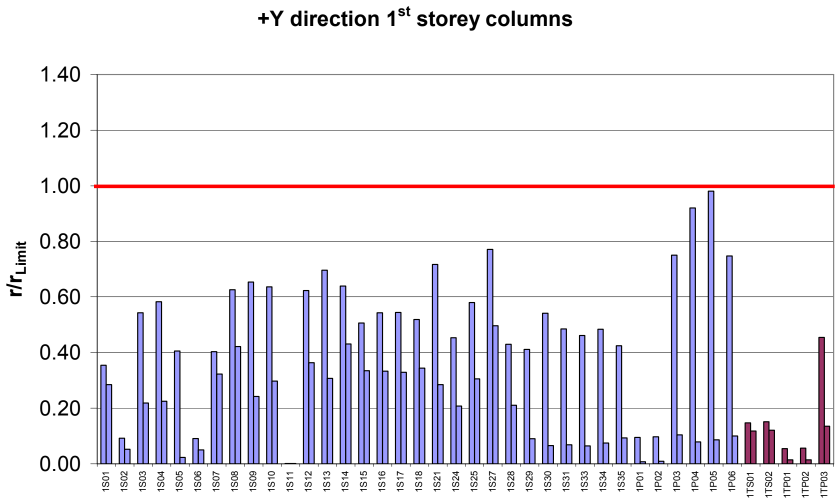


Figure 4.4 : r/r_{Limit} values (continued)





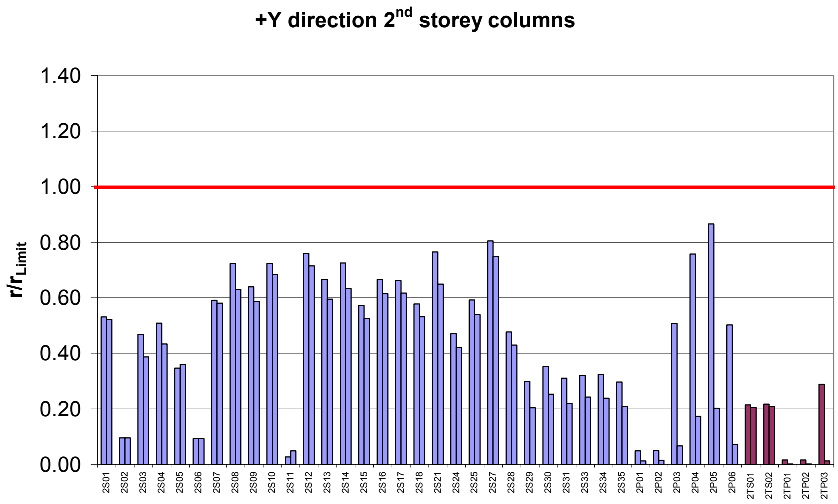
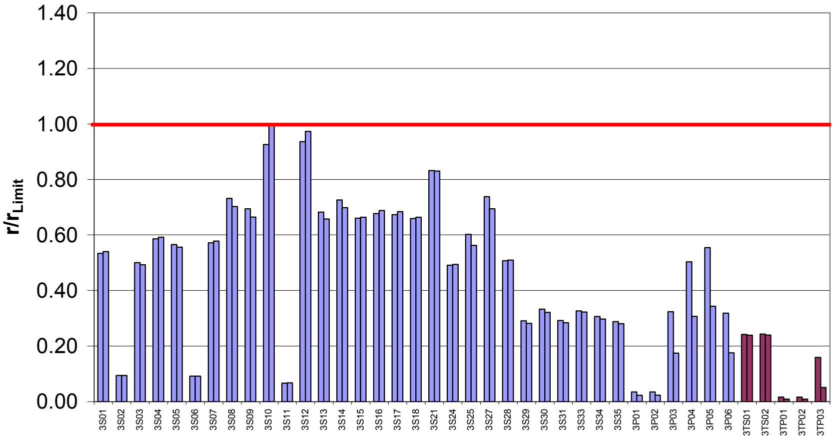
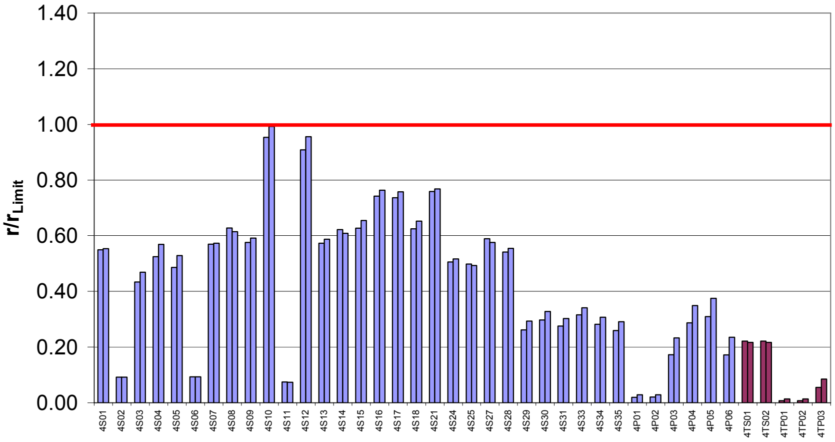


Figure 4.4 : r/r_{Limit} values (continued)

+Y direction 3rd storey columns



+Y direction 4th storey columns



+Y direction 5th storey columns

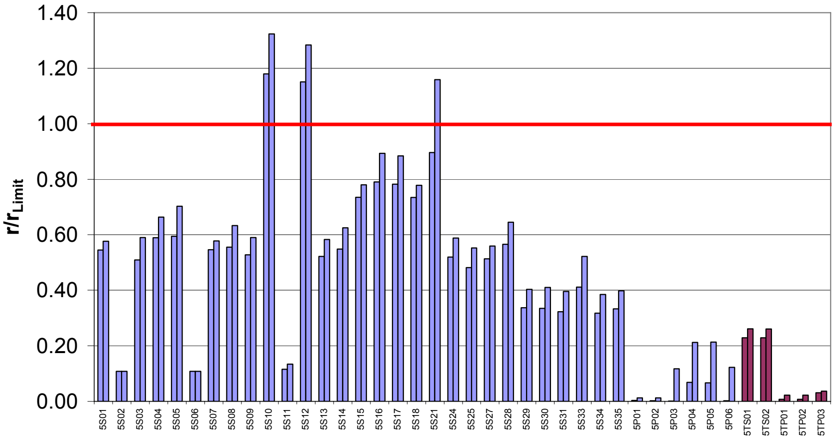
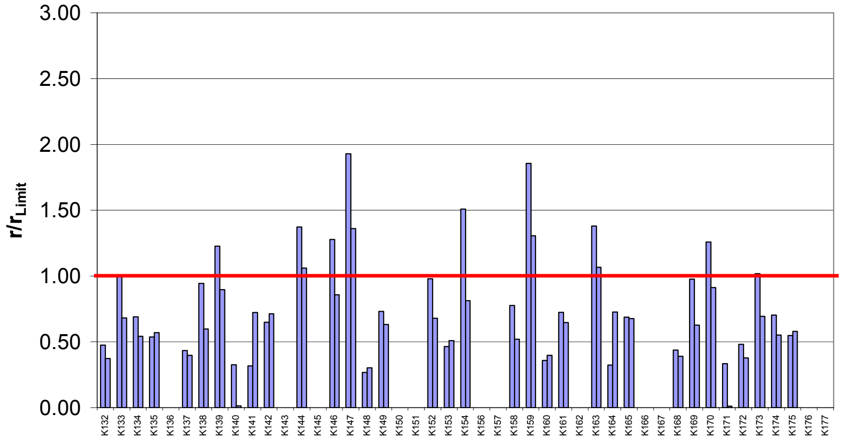
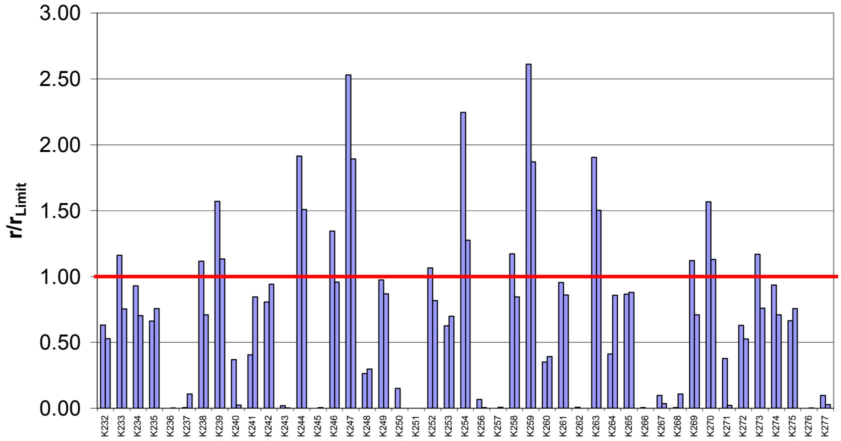


Figure 4.4 : r/r_{Limit} values (continued)

+Y direction 1st storey beams



+Y direction 2nd storey beams



+Y direction 3rd storey beams

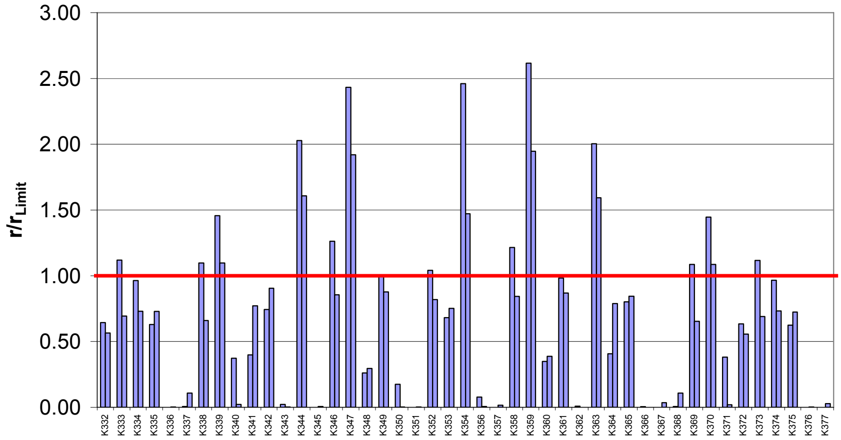
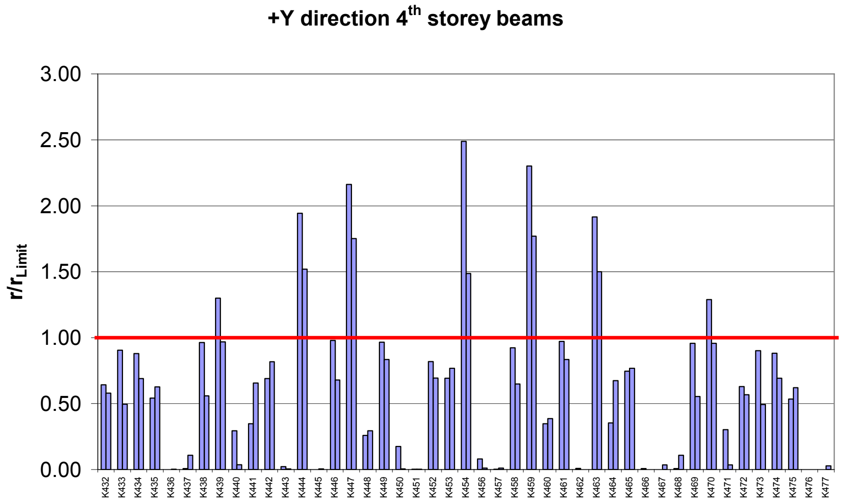


Figure 4.4 : r/r_{Limit} values (continued)



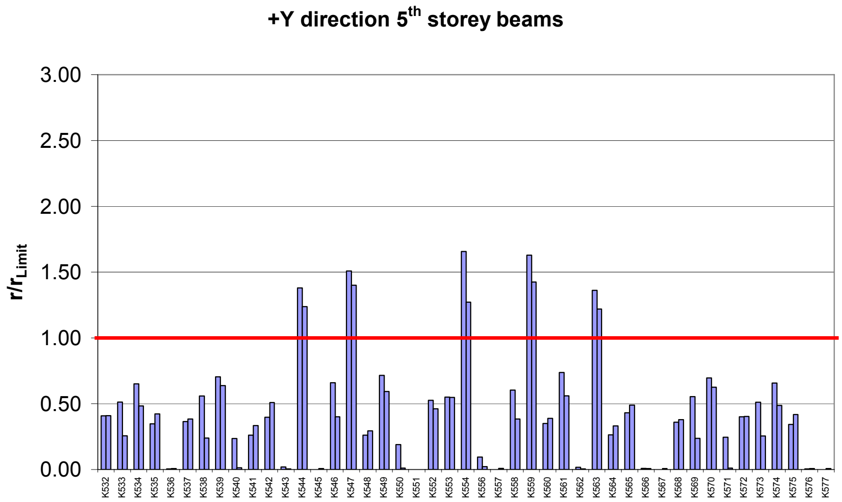


Figure 4.4 : r/r_{Limit} values (continued)

4.2.2 Global Performance of the Building

Before retrofitting, the existing shear walls were taking 79% and 61% of the storey shear in the +X and +Y directions respectively. These ratios are reduced to 29% and 49% after retrofitting.

Table 4.5 : Global performance of the members and the ratios of unacceptable members

	+X direction		+Y direction	
St	**Columns and Walls (%)**	**Beams (%)**	**Columns and Walls (%)**	**Beams (%)**
1	0.5	7.4	0.0	24.2
2	0.0	11.1	0.0	**42.4**
3	0.0	11.1	0.0	**39.4**
4	0.0	11.1	0.0	21.2
5	1.7	11.1	14.7	15.2

Global performance evaluation of the building was found to be ***not*** satisfactory for the Life Safety performance level because of the beams in Y direction. However, if the beams that have demand to capacity ratios smaller than 1.17 have been evaluated as acceptable (K233, K238, K252, K258, K269, K273, K333, K338, K369, K373), then beam ratios reduce to 24.2, 27.3, 27.3, 21.2 and 15.2 for the five stories respectively, which are within the limits of life safety. Accordingly building performance level becomes Life Safety.

Calculated interstorey drifts are presented in Table 4.6 for which the limit is 0.03.

Table 4.6 : Storey drifts

	+X direction			+Y direction		
St	**H_i (m)**	**$(\Delta_i)_{max}$**	**$(\Delta_i)_{max} / H_i$**	**H_i (m)**	**$(\Delta_i)_{max}$**	**$(\Delta_i)_{max} / H_i$**
1	2.8	0.006520	0.002328	2.8	0.005668	0.002024
2	2.8	0.011300	0.004036	2.8	0.009731	0.003476
3	2.8	0.013251	0.004733	2.8	0.011174	0.003991
4	2.8	0.013248	0.004732	2.8	0.011038	0.003942
5	2.8	0.012052	0.004304	2.8	0.009932	0.003547

4.3 Nonlinear Static Procedure

The building was also assessed by the non-linear procedure. The procedure was explained in the previous chapter in detail. Therefore, for this case study, only the results are summarized.

4.3.1 Target Displacement in the +X and +Y Directions

Target displacement calculation is performed by drawing the Code and capacity spectrum on the same graph. For both orthogonal directions, natural vibration periods are less than the corner period (T_B=0.6 sec) of the Code spectrum. Hence, an iterative procedure is applied as described in Chapter 2 and the target displacements are found as 0.0534m in the +X direction and 0.0584m in the +Y direction respectively (Figure 4.5 and Figure 4.6).

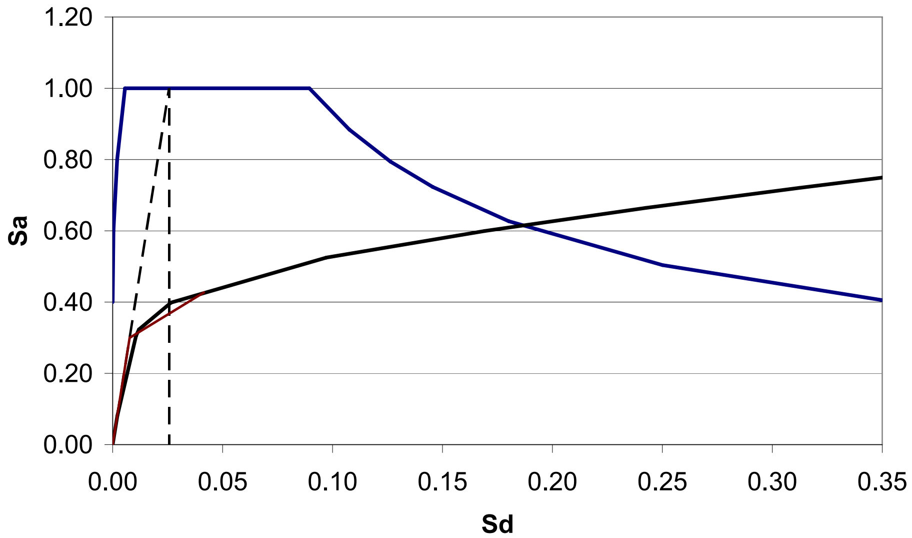


Figure 4.5 : Target displacement for +X direction

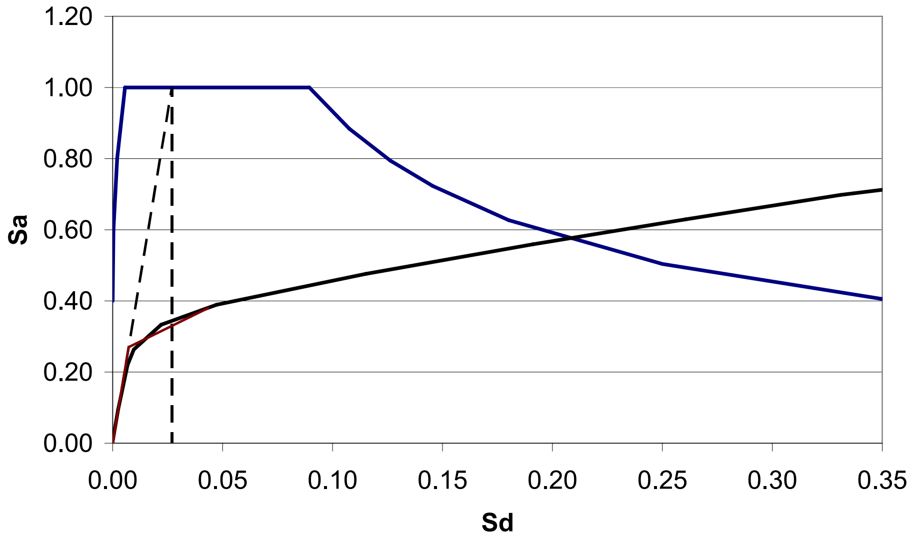
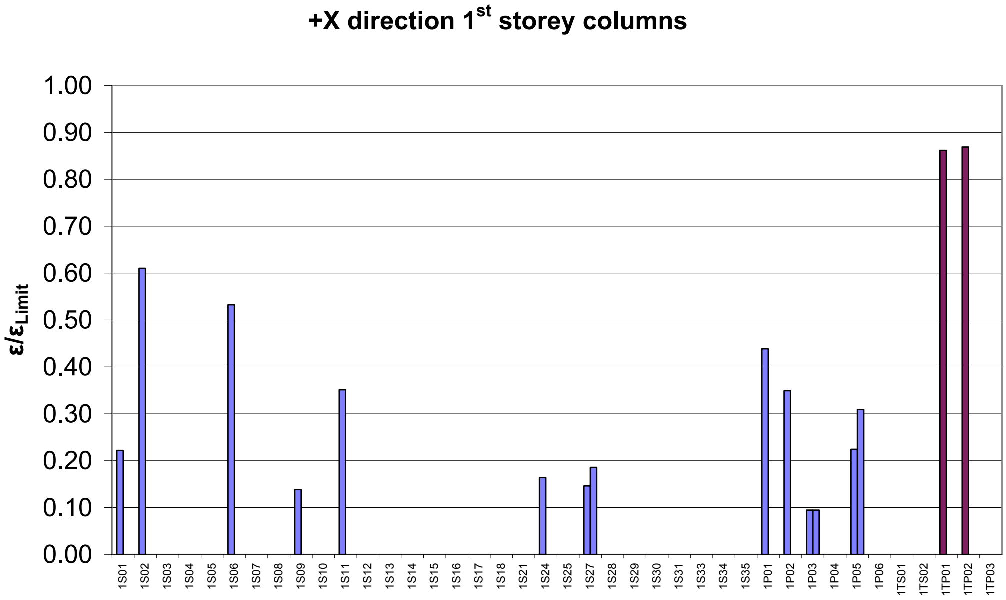


Figure 4.6 : Target displacement for +Y direction

4.3.2 Comparison of Strains (ε) with Strain Limits (ε_{Limit})

The ratio of ε / ε_{Limit} values are calculated for all member ends and presented in bar chart form for beams and vertical members separately at each storey.



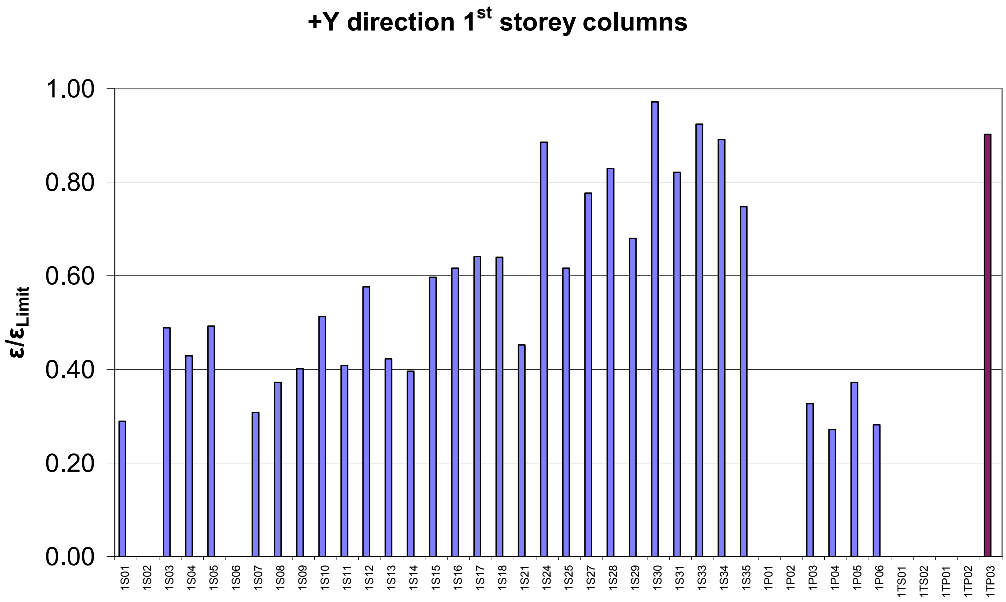


Figure 4.7 : ε / $\varepsilon_{\text{Limit}}$ values

All ε / $\varepsilon_{\text{Limit}}$ values of the columns and walls at stories 2, 3, 4 and 5 were smaller than 1 in both directions, so they are not shown here.

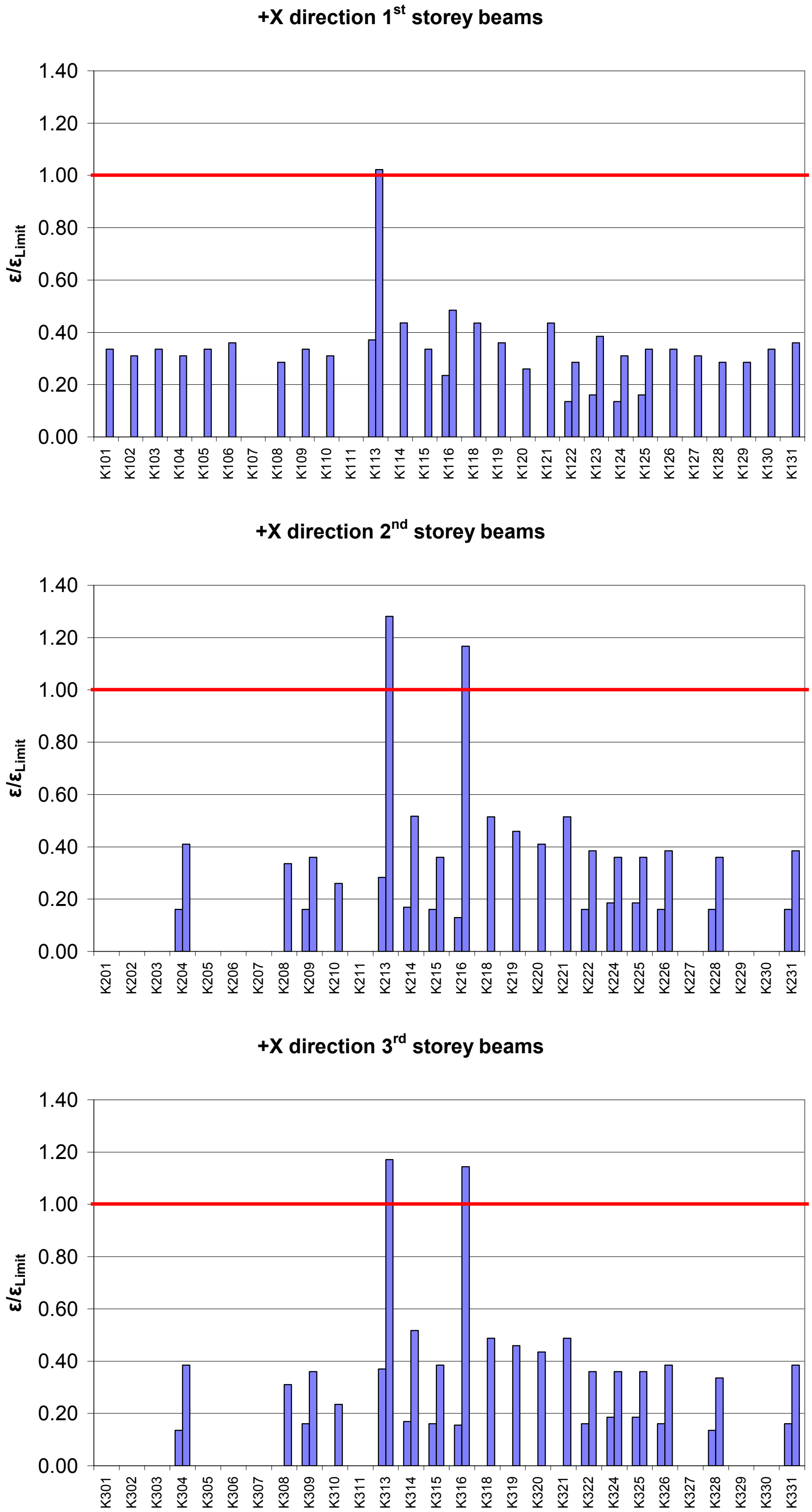
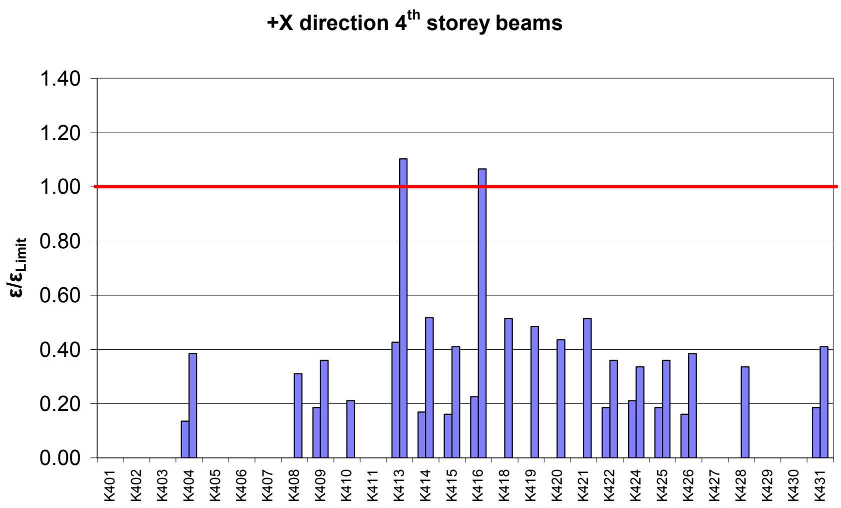
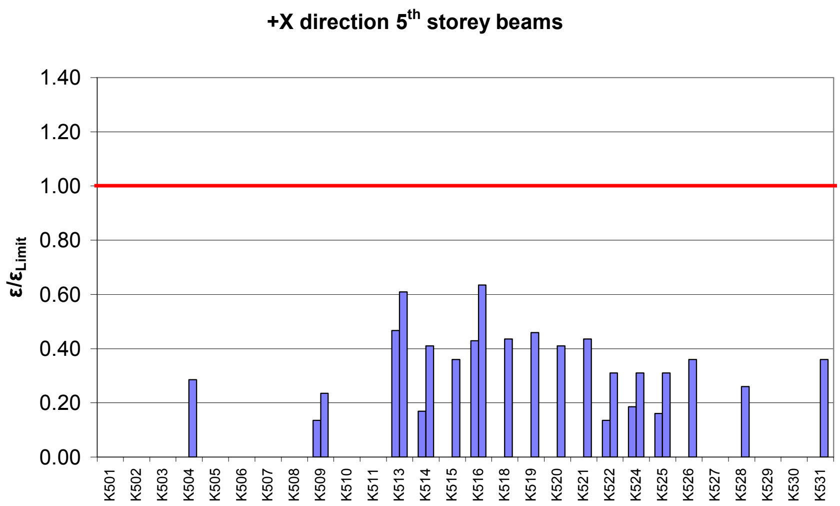


Figure 4.8 : ε / ε $_{\text{Limit}}$ values





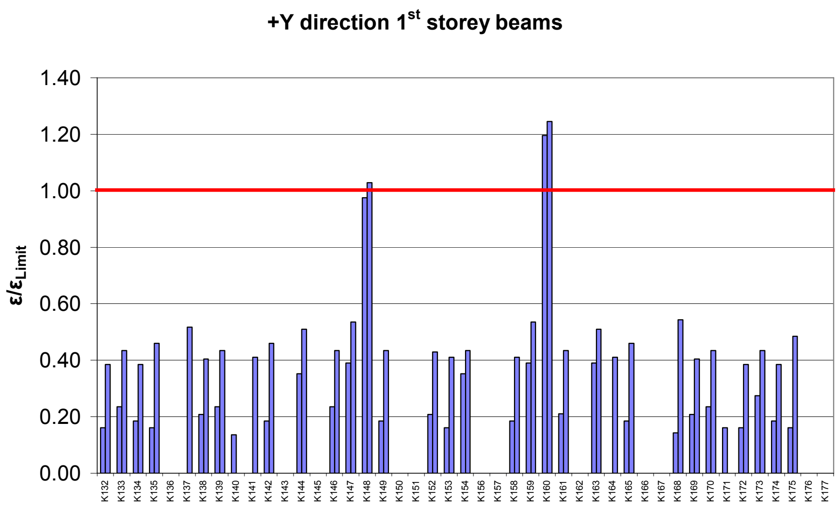
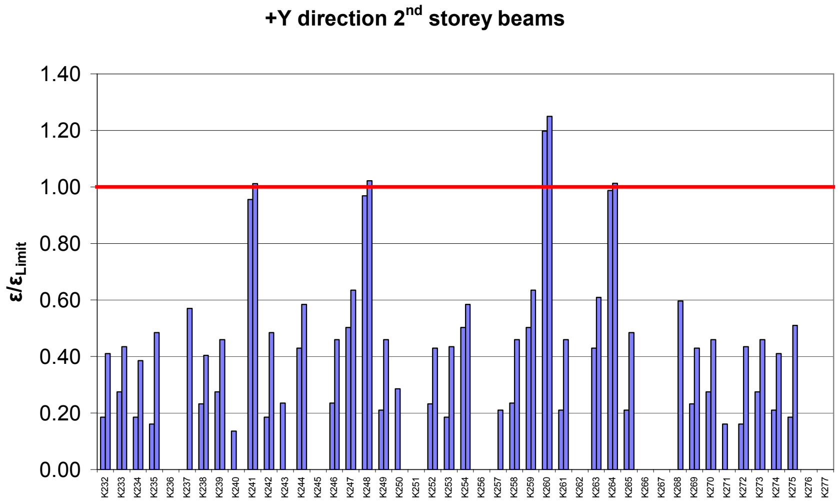
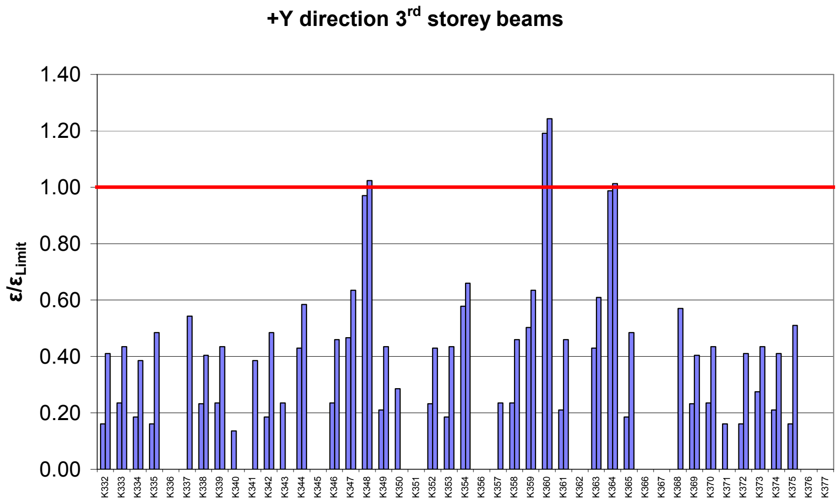


Figure 4.8 : ε / ε $_{Limit}$ values (continued)





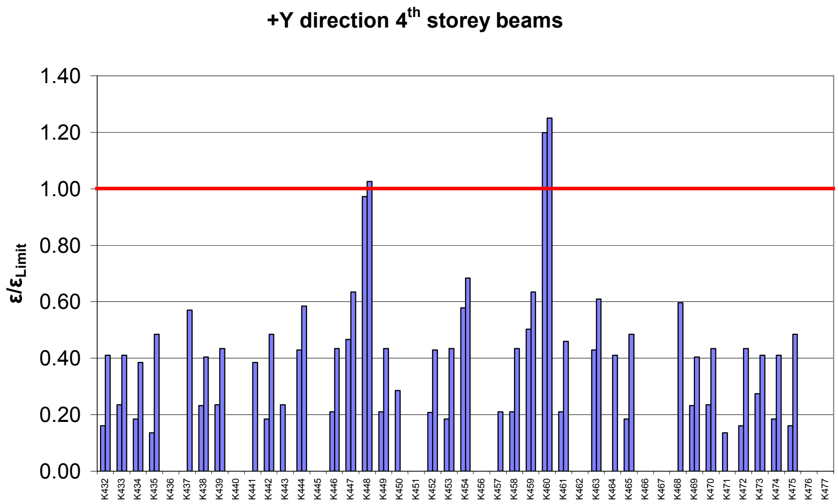


Figure 4.8 : ε / ε $_{Limit}$ values (continued)

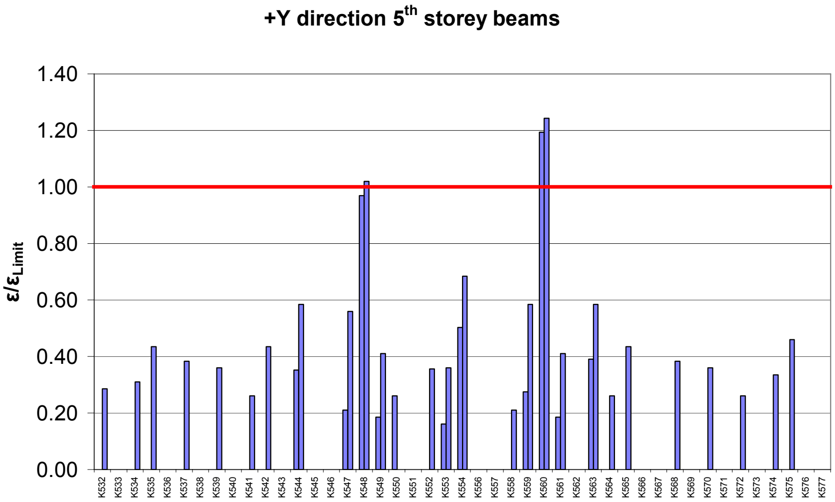


Figure 4.8 : ε / ε $_{Limit}$ values (continued)

4.3.4 Global Performance of the Building

Global performance evaluation of the building for all stories for two orthogonal directions are given in Table 4.7.

Table 4.7 : Global performance of the members

	+X direction		+Y direction	
St	Columns and Walls (%)	Beams (%)	Columns and Walls (%)	Beams (%)
1	0.0	3.7	0.0	6.1
2	0.0	11.1	0.0	12.1
3	0.0	11.1	0.0	9.1
4	0.0	11.1	0.0	6.1
5	0.0	0.0	0.0	6.1

Beams that suffered damage according to the nonlinear assessment procedure are the beams joining to the walls in both directions.

Since the case study was a residential building, the target performance level was selected as “Life Safety” and the calculated interstorey drifts should be less than 0.03. The results are given in Table 4.8 below.

Table 4.8 : Storey drifts

	+X direction			**+Y direction**		
St	**H_i (m)**	**$(\Delta_i)_{max}$**	**$(\Delta_i)_{max}$ / H_i**	**H_i (m)**	**$(\Delta_i)_{max}$**	**$(\Delta_i)_{max}$ / H_i**
1	2.8	0.007691	0.002747	2.8	0.009511	0.003397
2	2.8	0.012484	0.004458	2.8	0.012686	0.004531
3	2.8	0.013111	0.004683	2.8	0.013106	0.004681
4	2.8	0.013239	0.004728	2.8	0.012991	0.004640
5	2.8	0.013009	0.004646	2.8	0.012402	0.004429

Global performance evaluation of the building was found to be satisfactory for the Life Safety performance level.

CHAPTER V

CASE STUDY 3: ASSESSMENT OF AN EXISTING 6-STOREY RESIDENTIAL BUILDING IN BAKIRKÖY, İSTANBUL

In this case study, a six-storey existing residential building was assessed using both linear elastic and nonlinear procedures described in the 2007 Turkish Earthquake code.

5.1 Structural Properties of the Building

Case study building is composed of six stories with varying properties. First floor is 2.7m in height whereas upper floors are 2.8m in height. Floor areas are 132.6m^2 for the first two floors and 150.6m^2 for the others. This is because of the overhangs about 1.5m long, starting from the second floor. Structural system is composed of four and three moment resisting frames without shear wall, in X and Y direction respectively. Building photo, typical floor plan and 3D model can be found in Figure 5.1 through Figure 5.3. Frame system and member dimensions are not uniform through the height of the building. Typical section dimensions for frame E are given in Table 5.1. Example calculations for procedures will be given for the column 3S8 and beam K313 located on frame E.

Figure 5.1 : Building photo

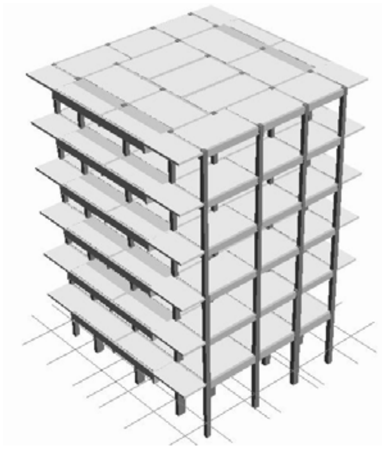


Figure 5.2 : 3D model of the building

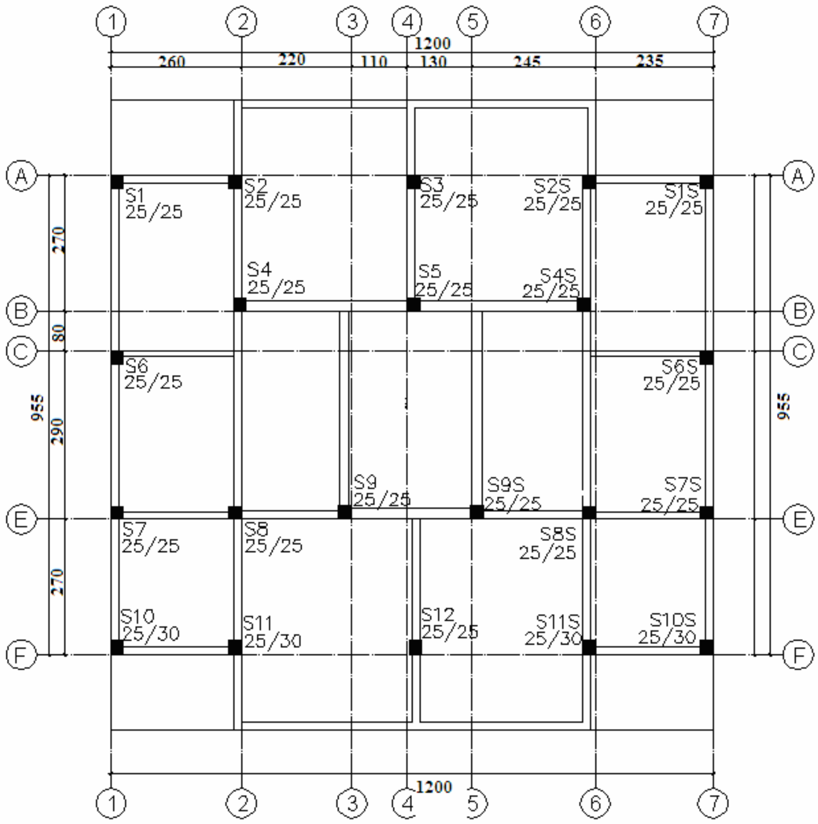


Figure 5.3 : Typical floor plan (3^{rd} storey)

All beams are 15x50cm at the first storey of frame E and all columns are 25x25cm at 5th and 6th stories of the frame E. Other section dimensions are given in Table 5.1.

Table 5.1 : Column and beam dimensions for frame E

Column	by(cm)	bx(cm)
1S7	40	30
1S7S	40	30
1S8	40	30
1S8S	40	30
1S9	25	55
1S9S	25	55
2S7	40	25
2S7S	40	25
2S8	40	25
2S8S	40	25
2S9	25	50
2S9S	25	50

Column	by(cm)	bx(cm)
3S7	30	25
3S7S	30	25
3S8	30	25
3S8S	30	25
3S9	25	30
3S9S	25	30
4S7	25	25
4S7S	30	25
4S8	25	25
4S8S	25	25
4S9	25	25
4S9S	25	25

Beam	b(cm)	h(cm)
K213	15	50
K214	20	50
K215	15	50
K216	15	40
K217	15	40
K313	15	50
K314	20	50
K315	15	50
K316	15	40
K317	15	40

Beam	b(cm)	h(cm)
K413	15	50
K414	20	50
K415	15	50
K416	15	30
K417	15	30
K513	15	50
K514	20	50
K515	15	50
K516	15	30
K517	15	30
K613	15	50
K614	20	50
K615	15	50
K616	15	30
K617	15	30

Existing building properties and associated parameters for the 2007 Turkish Earthquake Code are tabulated in Table 5.2.

Table 5.2 : Existing properties and code parameters of the building

Existing Building Properties	
Project available?	Yes
Knowledge level	Extensive
Knowledge level factor	1
Reinforcement establishment factor	1
Existing concrete strength	14 MPa (E_c = 26150 MPa)
Existing reinforcement strength	300 MPa
2007 Turkish Earthquake Code Parameters	
Seismic Zone	1
Seismic Zone Factor (A_o)	0.4
Building Importance Factor	1
Soil Class	Z3
Live Load Participation Factor	0.3
Target Performance Level	Life Safety

5.2 Assessment of the Building by Linear Elastic Procedure

5.2.1 Modeling and Analysis

Linear assessment procedure starts with correct modeling of the building for which, cracked section stiffness were used for all members. Uncracked section stiffness of beams are reduced by 40% in order to obtain cracked section stiffness, whereas for columns and shear walls, section modifiers are calculated with respect to axial load level of 1G+nQ load combination. Rigid end zone description for sample column and beam are shown in Figure 5.4 for frame E. Modal properties were found as given in

Table 5.3 by performing a eigenvalue analysis. Equivalent static loads were calculated and tabulated in Table 5.4.

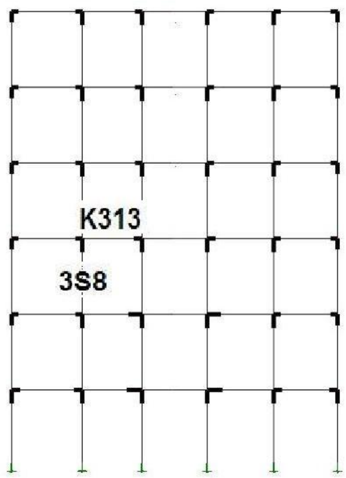


Figure 5.4 : Frame E with rigid end zones

Table 5.3 : Modal properties of the building

MODE	PERIOD	INDIVIDUAL MODE (PERCENT)			CUMULATIVE SUM (PERCENT)		
		UX	UY	UZ	UX	UY	UZ
1 (X)	1.144	69.710	0.008	0.000	69.710	0.008	0.000
2 (Y)	1.011	0.859	71.659	0.000	70.569	71.668	0.000
3	0.999	8.203	3.916	0.000	78.772	75.584	0.000
4	0.407	8.125	0.001	0.000	86.897	75.585	0.000
5	0.366	0.047	10.590	0.002	86.944	86.175	0.002
6	0.359	1.286	0.264	0.000	88.230	86.439	0.002

Using these modal properties equivalent static loads were calculated as follows:

$R_a(T_1) = 1$, $I = 1$, $A_o = 0.4$ and $W = 8501.1$ kN

For $T_x = 1.14$s and $T_y = 1.01$s, S(T) values were calculated as 1.49g and 1.65g respectively. From these values $A(T_1)$ are 0.59g and 0.66g.

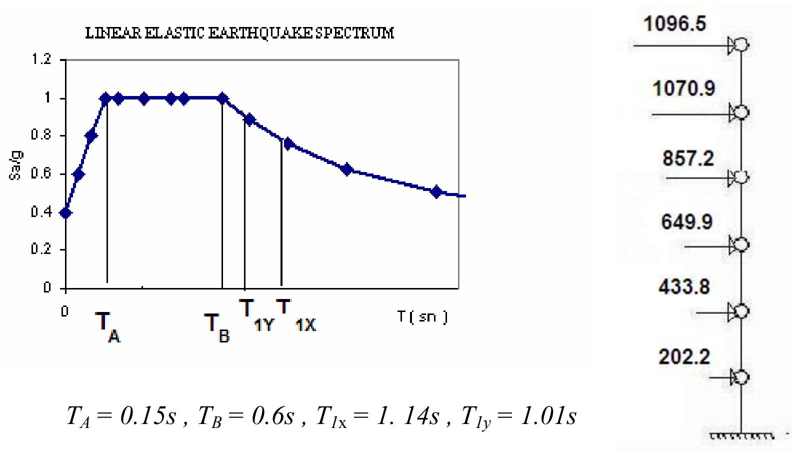
$\lambda = 0.85$, $V_t = 4310.5$ kN (X direction) and $V_t = 4760.8$ kN (Y direction)

$\Delta F_N = 0.0075 \; N \; V_t = 0.0075 \times 6 \times 4310.5 = 194.0$ kN (X direction)

$\Delta F_N = 0.0075 \; N \; V_t = 0.0075 \times 6 \times 4760.8 = 214.2$ kN (Y direction)

Table 5.4 : Equivalent static lateral load distribution

St	Storey Mass (t)	Storey Weight (W_i)(kN)	Storey Height (m)	H_i (m)	$W_i H_i$ (kNm)	F_{iX} (kN)	F_{iY} (kN)
1	147.39	1445.9	2.7	2.7	3903.9	202.2	223.4
2	155.20	1522.5	2.8	5.5	8373.8	433.8	479.1
3	154.08	1511.5	2.8	8.3	12545.7	649.9	717.8
4	151.96	1490.7	2.8	11.1	16547.1	857.2	946.7
5	151.60	1487.2	2.8	13.9	20672.0	1070.9	1182.8
6	106.34	1043.2	2.8	16.7	17421.4	1096.5	1211.0



$T_A = 0.15s$, $T_B = 0.6s$, $T_{1x} = 1.14s$, $T_{1y} = 1.01s$

Figure 5.5 : Earthquake design spectrum and lateral load distribution

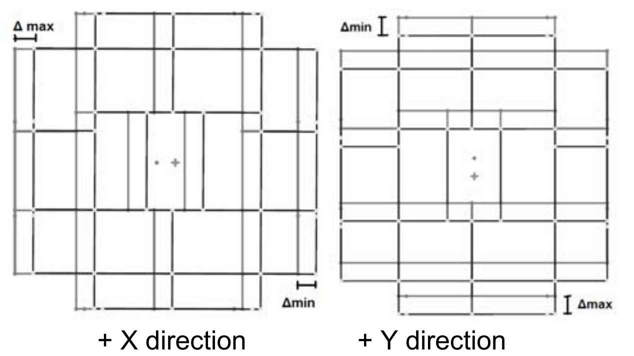
Before starting the assessment process, building model must be controlled for the appropriateness of the equivalent static load procedure. Building has;

Total height of 16.7m which is less than 25m.

Total number of floors is 6 which is less than 8.

Maximum torsional irregularity coefficient (η_{bi}) as shown in Figure 5.6 was 1.13, which is less than 1.4.

These three checks showed that building satisfies the requirements of equivalent static load procedure, hence this procedure is used.



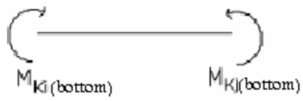
	+X	+Y
$(\Delta_i)_{max}$	0.05218	0.03822
$(\Delta_i)_{min}$	0.03991	0.03756
$(\Delta_i)_{mean}$	0.04604	0.03789
η_b	1.13323	1.00868

Figure 5.6 : Torsional irregularity check

5.2.2 Calculation of Member Capacities

Beam moment capacities are calculated using simple section analysis procedure. Example beam K313 (15cmx50cm) has ϕ6/20cm transverse reinforcement with no confined region. Longitudinal reinforcement area and calculated moment capacities are given in Table 5.5.

Table 5.5 : End moment capacity of beam K313

	Moment capacity at top		Moment capacity at bottom	
	i	***j***	***i***	***j***
A_s (cm^2)	2.18	2.18	1.46	1.46
M_K (kNm)	30.38	30.38	20.78	20.78
				

Column moment capacity calculation will be done by using the procedure described in Section 2.1.2.1 and a sample calculation will be given for the column 3S8.

Axial load will be transferred from the beams joining to the columns 3S8, 4S8, 5S8 and 6S8 and N_E notation will be used for these axial loads. N_D is the axial load of the column under 1G+nQ loading.

Capacity Control procedure starts with the calculation of shear transmitted from beams. M_K represents the moment capacity of the beam and M_D is the moment induced by vertical loading which was reduced to 85%. Third storey beams K316 and K313 joins to the column 3S8 and shear transmitted to the column is $V_{E,3}$.

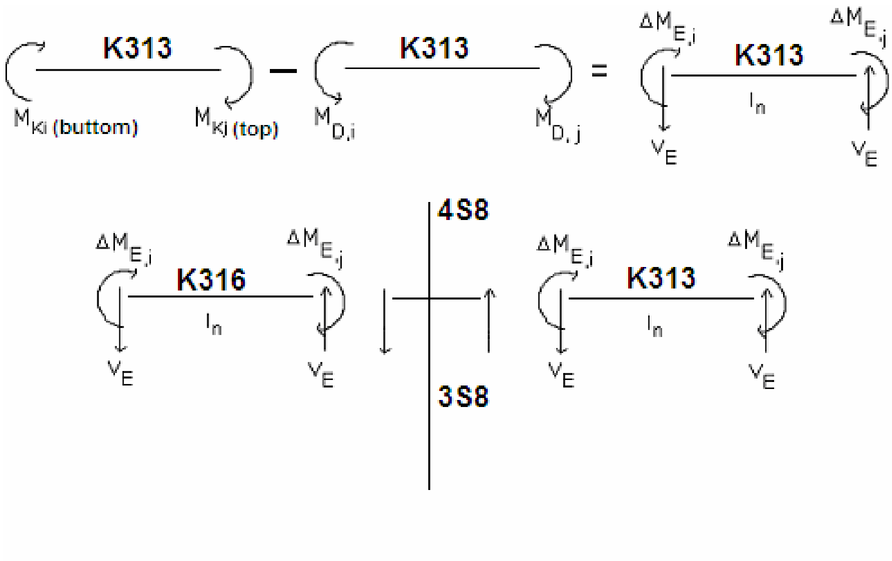


Figure 5.7 : Shear force transmitted from K313 and K316 to the columns 3S8

Beam K313

$M_{K,i}$(bottom)	: 20.8 kNm	$M_{K,j}$(top)	: -30.4 kNm
$M_{D,i}$(bottom)	: -4.73 kNm	$M_{D,j}$(top)	: -2.61 kNm
$\Delta M_{E,i} = M_{Ki}(bot) - M_{D,i}$		$\Delta M_{E,j} = M_{Kj}(top) - M_{D,j}$	
$= 20.8 - (-4.73)$	: 25.53 kNm	$= -30.4 - (-2.61)$	: -27.79 kNm

$$V_{E,3,2} = (\Delta M_{E,i} - \Delta M_{E,j}) / l_n$$

$$= (25.53 - (-27.79)) / 1.9 = 28.1 \text{ kN}$$

Same calculations were applied to the beam K316 and $V_{E,3,1}$ was found to be 16.06 kN in opposite direction. Total shear transferred to the column 3S8 at storey 3 was;

$$V_{E,3} = V_{E,3,1} + V_{E,3,2} = 16.06 + (-28.1) = -10.04 \text{ kN (tension)}$$

Similarly;

$V_{E,4}$ = -15.8 kN , $V_{E,5}$ = -14.5 kN, $V_{E,6}$ =-16.0 kN

$N_{E,3}$ is the total shear force transmitted to the column 3S8 from the beams joining to the S8 columns above 3S8.

$N_{E,3} = V_{E,3} + V_{E,4} + V_{E,5} + V_{E,6} = -10.04 - 15.8 - 14.5 - 16.0 = -56.34$ kN

$N_D = 259.0$ kN

$N_D + N_{E,3} = 259.0 - 56.34 = 202.86$ kN (compression)

Moment capacity of the column 3S8 was calculated using the axial load of 202.86 kN

$M_K = 36.85$ kNm

Calculating the moment capacity of 4S8 as 29.28 kNm column to beam capacity ratio (CBCR) was found as follows;

$$\text{CBCR} = \frac{(M_{Kbot} + M_{Ktop})}{(M_{Ki(bot)} + M_{Kj(top)})} = \frac{(29.28 + 36.85)}{(20.78 + 19.32)} = 1.65$$

CBCR values are used to identify the hinge mechanism and if it is smaller than 1, moment capacities of beams joining to that joint are reduced by CBCR. Other values of CBCR of 4S8, 5S8 and 6S8 were 1.59, 1.36, and 0.61 respectively. On the adjacent column axis 9 CBCR values were 1.12, 0.83, 0.69, 0.3 for columns 3S9, 4S9, 5S9 and 6S9. Other adjacent column axis 7 had all CBCR values of larger than 1.

Beam capacities were modified for the CBCR values smaller than 1.

Table 5.6 : Beam moment modification with CBCR

Beam	**Moment Capacity (bottom / top)**	**CBCR**	**Modified Moment Capacity**	**Shear**
K613	20.78 (end-i) 30.4 (end-j)	0.61 0.3	12.68 9.12	12.49
K616	10.6 (end-i) 13.7 (end-j)	 0.613	10.6 8.4	9.04
K513	20.8 (end-i) 30.4 (end-j)	 0.694	20.8 21.1	21.9
K413	20.8 (end-i) 30.4 (end-j)	 0.836	20.8 25.6	24.1

Using modified beam capacities and thus shear forces, shear transferred from beams to the columns were recalculated.

$V_{E,6} = 9.04 – 12.49 =$ **-3.45 kN**.

$V_{E,5} = V_{E,5,1} + V_{E,5,2} = 12.2 + (-21.9) =$ **-9.7 kN**

$V_{E,4} = V_{E,4,1} + V_{E,4,2} = 12.0 + (-24.1) =$ **-12.1 kN**

Axial loads were;

6th storey 6S8 column $N_{E,6} = V_{E,6} = -3.45 =$ **-3.45 kN**

5th storey 5S8 column $N_{E,5} = V_{E,5} + V_{E,6} = -9.7 - 3.45 =$ **-13.15 kN**

4th storey 4S8 column $N_{E,4} = V_{E,4} + V_{E,5} + V_{E,6} = -12\ 1 - 9.7 - 3.45 =$ **-25.5 kN**

3rd storey 3S8 column $N_{E,1} = V_{E,3} + V_{E,4} + V_{E,5} + V_{E,6} = -10.94\ \ -12\ 1\ -9.7 - 3.45 =$ **-36.44 kN** which was -56.34 kN before modification.

$N_K = N_D + N_{E,3} = 259.0 - 36.44 = 222.56$ kN

These axial loads constitute limit values for the axial loads calculated from graphical procedure.

Required parameters for the calculation of axial load by graphical procedure for column 3S8 were given below.

N_D : 259.0 kN Axial load due to gravity loading.

$M_{D\text{-}bot}$: 0.1 kNm Bottom moment due to gravity loading.

$M_{D\text{-}top}$: -0.15 kNm Top moment due to gravity loading.

N_E : -215.6 kN Axial load of earthquake loading with R=1.

$M_{E\text{-}bot}$: -267.8 kNm Bottom moment of earthquake loading with R=1.

$M_{E\text{-}top}$: 278.7 kNm Top moment of earthquake loading with R=1.

$N_{K\text{-}bot}$,$M_{K\text{-}bot}$: Moment and axial load capacities calculated from interaction diagram.

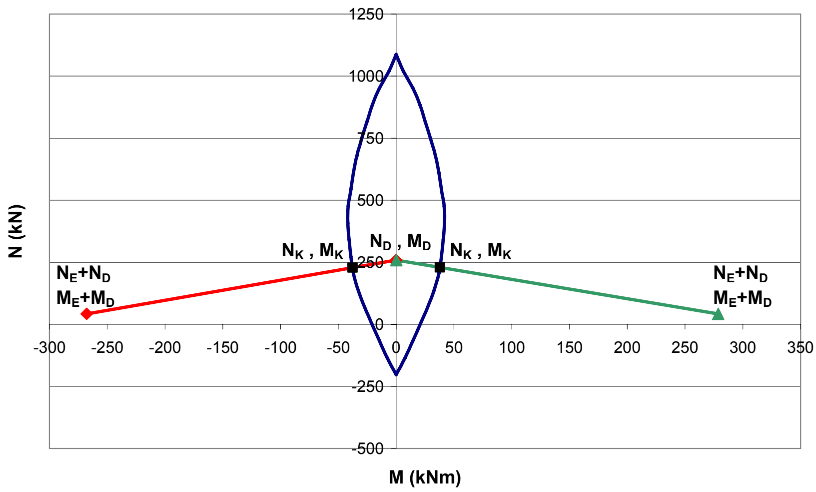
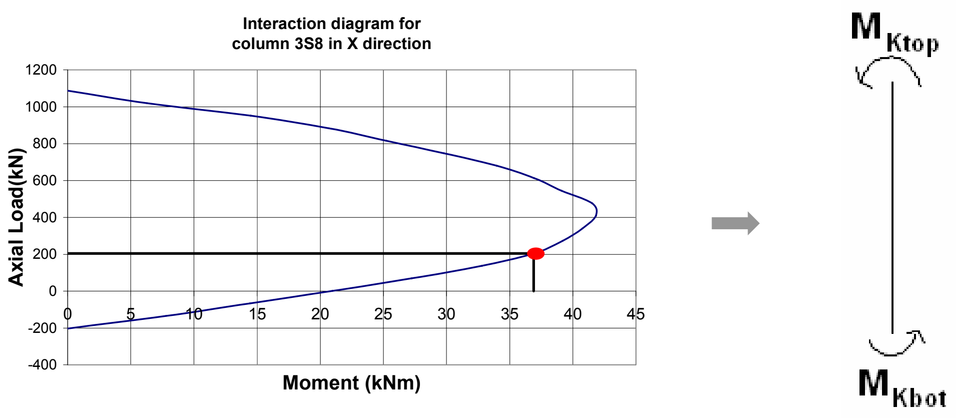
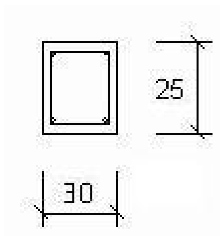


Figure 5.8 : Axial load calculation using graphical procedure

Axial load was computed to be 228.7 kN using graphical procedure. According to the rule 7A-3 of the 2007 Turkish Earthquake Code, column axial load was 222.6 kN which was smaller than 228.7 kN.

Finally, moment capacity can be calculated using axial load of 222.7 kN and the interaction diagram.





Mid Span transverse reinforcement: Ø6 / 20
Longitudinal reinforcement: 4 Ø14
Support transverse reinforcement : Ø6 / 20

$M_{Kbıt}$	37.6 kNm
M_{Ktop}	37.6 kNm

Figure 5.9 : Column moment capacity

After calculating moment capacity of members, next step is to compute shear capacity and shear demand related to flexural strength. Shear capacity for column 3S8 was calculated using the formula given in TS-500 (2000).

$$V_r = V_c + V_w = 0.8\ V_{cr} + V_w \quad (5.1)$$

$$V_c = 0.8 \text{ x } 0.65\ f_{ctm}\, b_w\ d\ (1 + \gamma\ N / A_c) \quad (5.2)$$

$$= 0.8 \text{ x } 0.65 \text{ x } 1.31 \text{ x } 300 \text{ x } 235 \text{ x } (1 + 0.07 \text{ x } 222.6 \text{ x } 1000 / (250 \text{ x } 300))$$

$$= 58007 \text{ N}$$

$$V_w = A_{sw}\ f_{ywm}\ d / s = 56.54 \text{ x } 400 \text{ x } 235 / 200 = 26574 \text{ N}$$

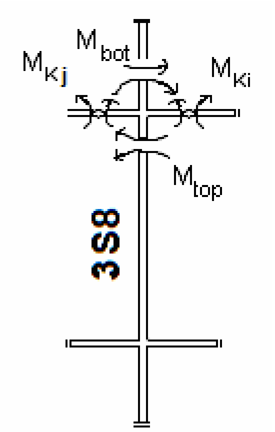
V_r = **84.6 kN**

Remembering that CBCR are larger than 1 for top and bottom sections of 3S8, top and bottom moments used for the calculation of shear demand are calculated using Equations 5.3 and 5.4.

$$M_{top} = \frac{M_{top}}{M_{top} + M_{bottom}} \sum M_K \tag{5.3}$$

$$\sum M_K = M_{Ki} + M_{Kj} \tag{5.4}$$

Where, M_{top} and M_{bot} are the moments induced by the lateral loads and M_{top} represents the moment at the top of column 3S8 and M_{bot} represents the moment at the bottom of column 4S8. M_{Ki} is the moment capacity of the beam joining to the right of the joint at the top of column 3S8 and M_{Kj} is the moment capacity of the beam joining to the left.



M_{top}	278.23 kNm
M_{bot}	-213.29 kNm
M_{Ki}	20.78 kNm
M_{Kj}	19.32 kNm

Figure 5.10 : Calculation of M_{top}

$\Sigma M_K = 20.78 + 19.32 = 40.1$ kNm

$M_{top} = 40.1 \times 278.73 / (213.29 + 278.73) = 22.7$ kNm

Same calculations were done for the bottom end of the column 3S8 and using these moments shear demand were calculated as;

$V_E = (M_{K\ top} + M_{K\ bot}) / l_n$

$V_E = (22.7 + 19.4) / 2.3 = \mathbf{18.3\ kN}$

$V_E < V_r$ means that column 3S8 is **ductile**.

Similar procedure is applied to the beams and member failure type is defined using the equations for beams. Sample calculation was given for the beam K313.

$V_r = V_c + V_w = 0.8\ V_{cr} + V_w$

$V_c = 0.8x0.65\ f_{ctm}\ b_w\ d = 0.8 \times 0.65 \times 1.31 \times 150 \times 485 = 49557.3$ N,

$V_w = A_{sw}\ f_{ywm}\ d/s = 56.54 \times 400 \times 485 / 200 = 54844$ N,

$V_{ri} = V_{rj} = \mathbf{104.4\ kN}$

Section shear demand calculation is based on Figure 5.11.

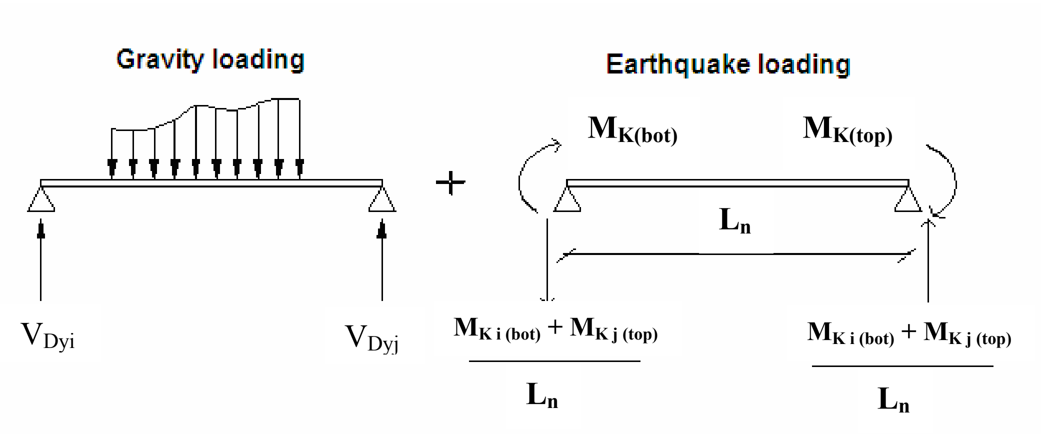


Figure 5.11 : Shear demand calculation of beams

$V_E = V_{Dy} – (M_{Ki,(bot)} + M_{Kj,\,(top)}) / l_n$ (5.5)

V_{Dy} = 11.99 kN, $M_{Ki(bot)}$ =20.78 kNm, $M_{Kj(top)}$ = 30.38 kNm, l_n (clear length) = 1.9 m,

V_{Ei} = **15.14 kN**

Equation 5.5 is $V_E = V_{Dy} + (M_{Ki}(bot) + M_{Kj}(top)) / l_n$ for end j.

V_{Dy} = 13.67 kN, $M_{Ki(bot)}$ = 20.78 kNm, $M_{Kj(top)}$ = 30.38 kNm, l_n =1.9 m

V_{Ej} = **40.80 kN**

For both ends, V_E values were smaller than V_r, thus, beam K313 was evaluated as **ductile**.

5.2.3 Comparison of Demand / Capacity Ratios (r) with Limit Values (r_{Limit})

Parameters that were used for the r_{Limit} definition of column 3S8 from Table 2.2 were:

$$\left.\begin{array}{l} N = 222.6\ kN \\ A_c = 750\ cm^2 \\ f_{cm} = 14\ MPa \end{array}\right\} \quad N_K / (A_c\, f_{cm}) = 0.21$$

$$\left.\begin{array}{l} V = V_E = 18.3\ kN \\ b_w\, d = 705\ cm^2 \\ f_{ctm} = 1.31\ MPa \end{array}\right\} \quad V_E / (b_w\, d\, f_{ctm}) = 0.19$$

Column section is not confined.

For Life Safety performance level and the parameters given above, by linear interpolation r_{Limit} was found to be **2.9** from Table 2.2. This value was compared with the residual moment capacity of the section for which, M_E at top of the column was 278.73 kNm and 267.76 kNm at the bottom of the column.

$$r_{bottom} = \frac{M_{Ebot}}{M_K - M_D} = \frac{267.76}{37.6-(-0.1)} = 7.1 \qquad \frac{r}{r_{Limit}} = \frac{7.1}{2.9} = 2.42$$

$$r_{top} = \frac{M_{Etop}}{M_K - M_D} = \frac{278.73}{37.6-(-0.15)} = 7.38 \qquad \frac{r}{r_{Limit}} = \frac{7.38}{2.9} = 2.53$$

Both ends of the column 3S8 did not satisfy the limit of Life Safety thus the member was unacceptable according to the Life Safety performance level. If r / r_{Limit} values is larger than 1 for one end only, that member would not satisfy the performance level considered.

Parameters that were used for the r_{Limit} definition of beam K313 from Table 2.3 were:

End-i

$$\left.\begin{array}{l}\rho = 0.002\\ \rho' = 0.003\\ \rho_b = 0.01851\end{array}\right\} \frac{(\rho-\rho')}{\rho_b} = -0.054$$

End-j

$$\left.\begin{array}{l}\rho = 0.003\\ \rho' = 0.002\\ \rho_b = 0.01851\end{array}\right\} \frac{(\rho-\rho')}{\rho_b} = 0.054$$

$$\left.\begin{array}{l}V = 13.14 \text{ kN}\\ b_w d = 727.5 \text{ cm}^2\\ f_{ctm} = 1.31 \text{ MPa}\end{array}\right\} \frac{V_E}{b_w d f_{ctm}} = 0.16$$

$$\left.\begin{array}{l}V = 40.80 \text{ kN}\\ b_w d = 727.5 \text{ cm}^2\\ f_{ctm} = 1.31 \text{ MPa}\end{array}\right\} \frac{V_E}{b_w d f_{ctm}} = 0.43$$

Beam section is not confined.

Considering Life Safety performance level and the parameters given above, r_{Limit} was found to be **4** for both ends from Table 2.3 These values were compared with the residual moment capacities of the section for which, M_E at end-i was 327.57 kNm and 328.7 kNm at the end-j.

$$r_i = \frac{M_{Ei}}{M_K - M_D} = \frac{327.57}{20.8 - (-3.77)} = 13.34 \qquad \frac{r}{r_{Limit}} = \frac{13.34}{4} = 3.34$$

$$r_j = \frac{M_{Ej}}{M_K - M_D} = \frac{328.7}{30.4 - (2.39)} = 12.16 \qquad \frac{r}{r_{Limit}} = \frac{12.16}{4} = 3.04$$

Both ends of the beam K313 did not satisfy the limit of Life Safety thus the member was unacceptable according to the Life Safety performance criteria. If r / r_{Limit} values is larger than 1 for one end only, that member would not satisfy the performance level as well.

r / r_{Limit} values for all members and for all floors are given in Figure 5.12.

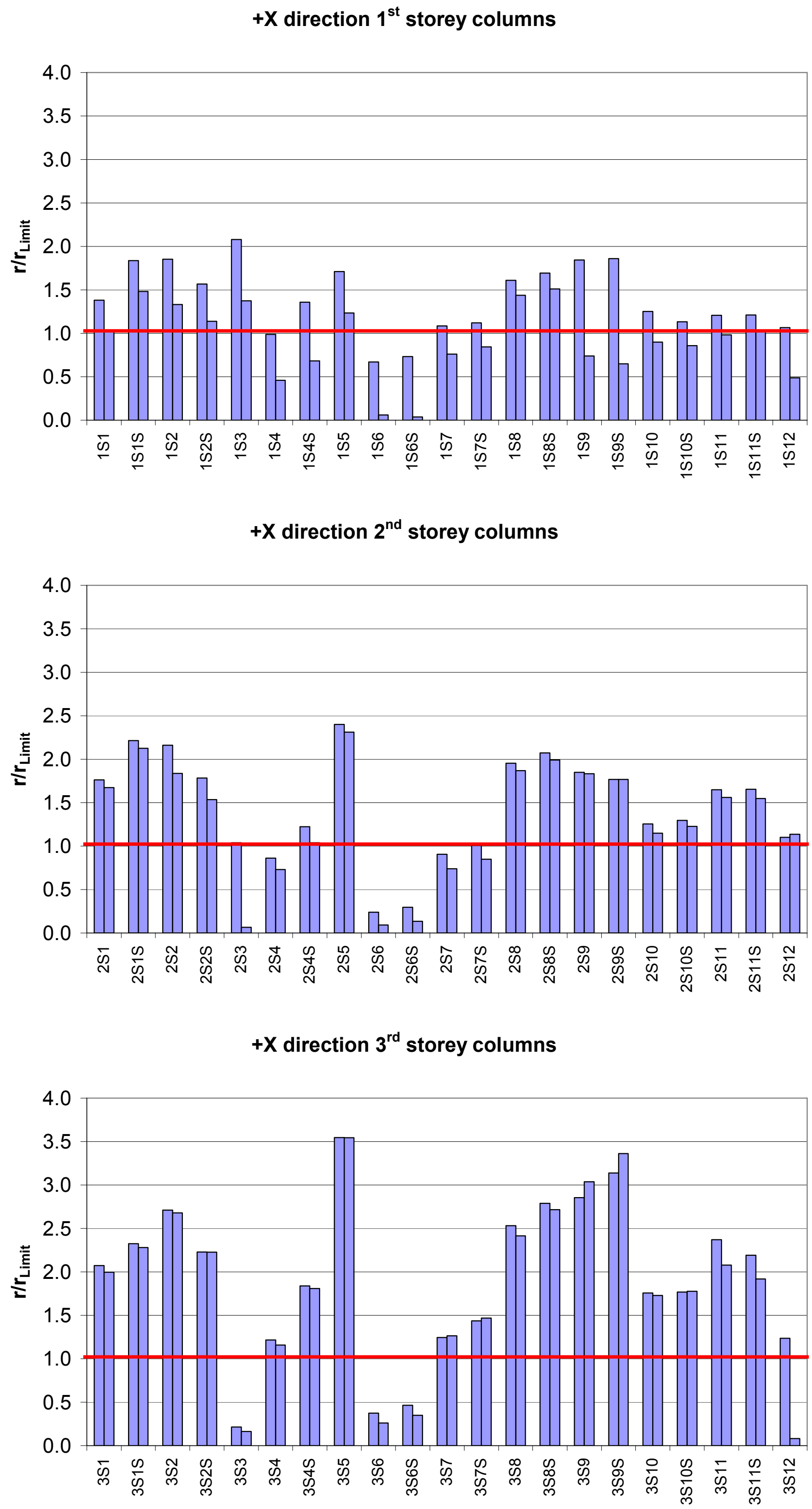
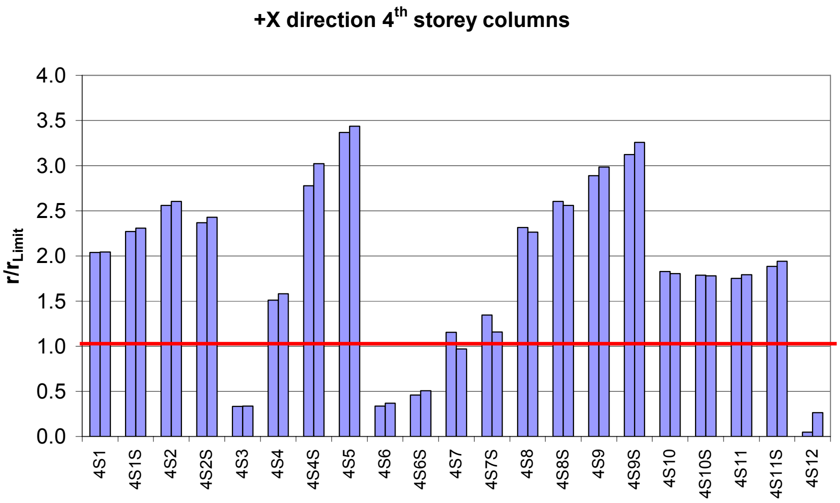
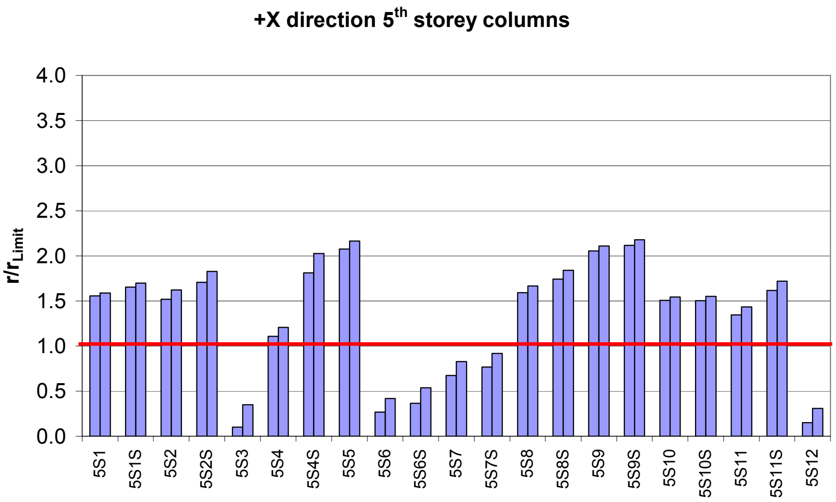


Figure 5.12 : r/r_{Limit} values





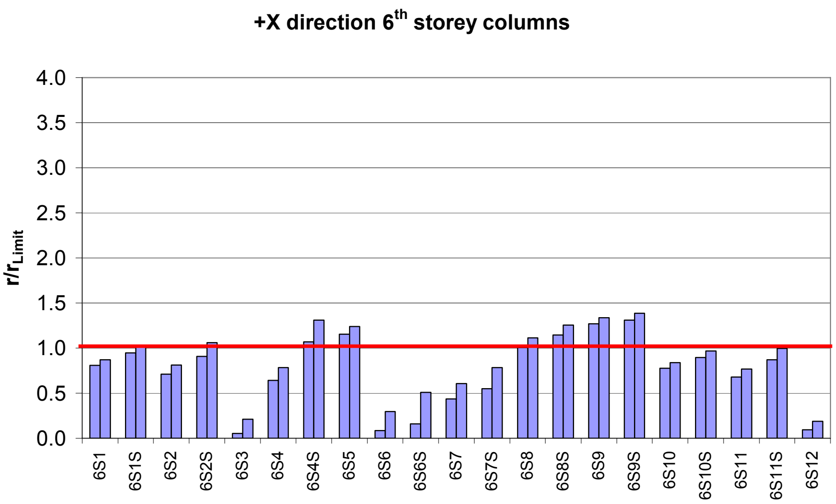
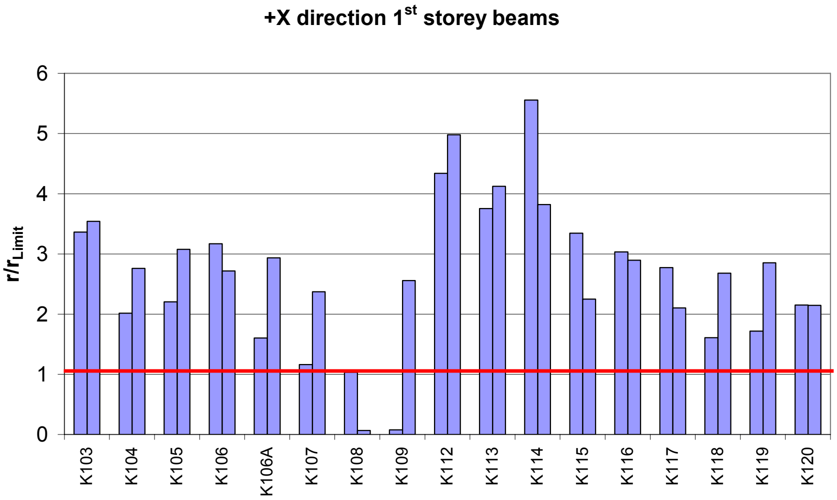
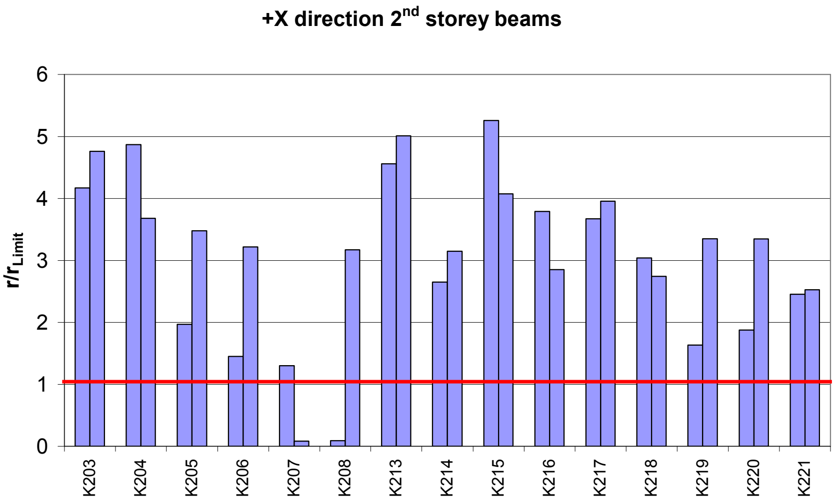


Figure 5.12 : r/r_{Limit} values (continued)





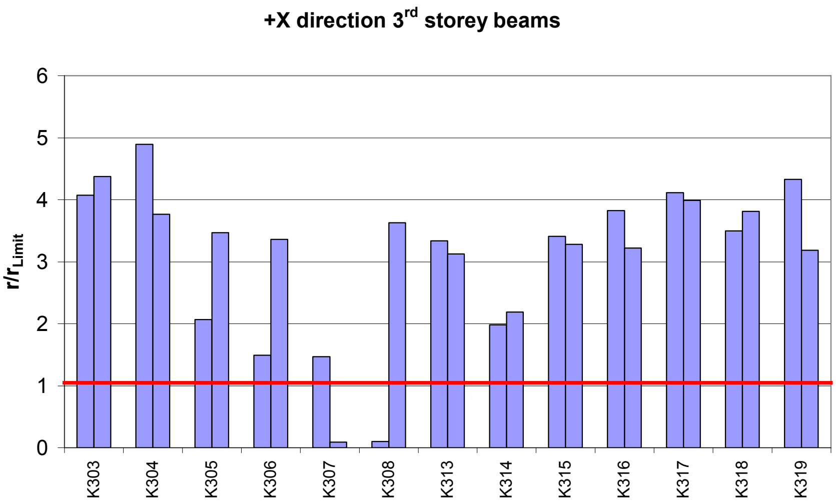
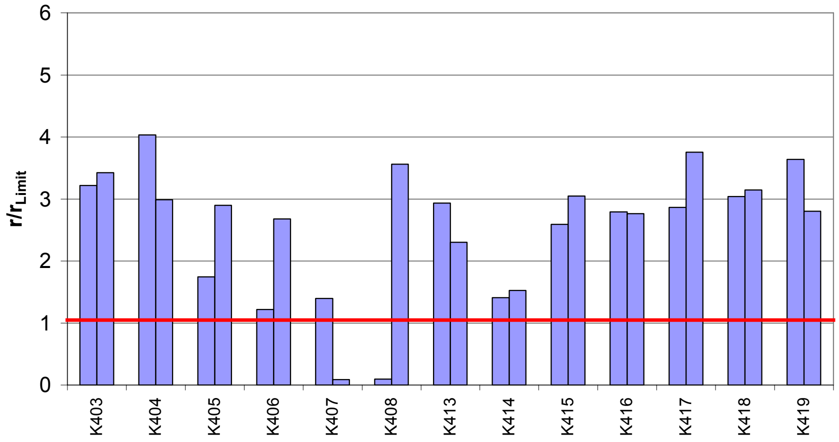
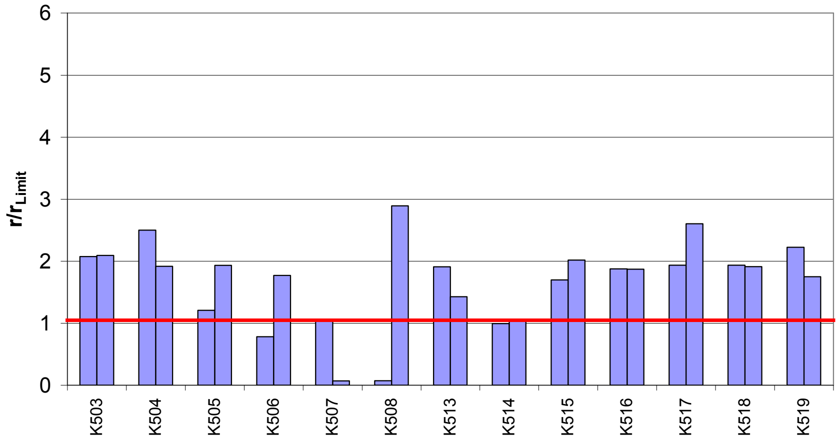


Figure 5.12 : r/r_{Limit} values (continued)

+X direction 4th storey beams



+X direction 5th storey beams



+X direction 6th storey beams

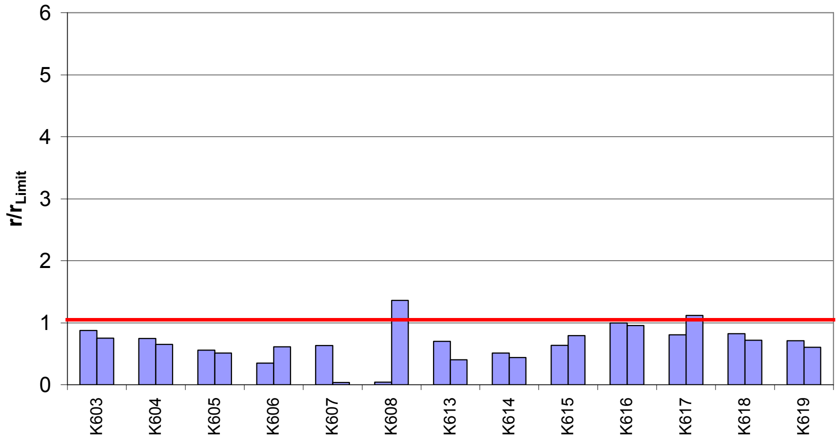
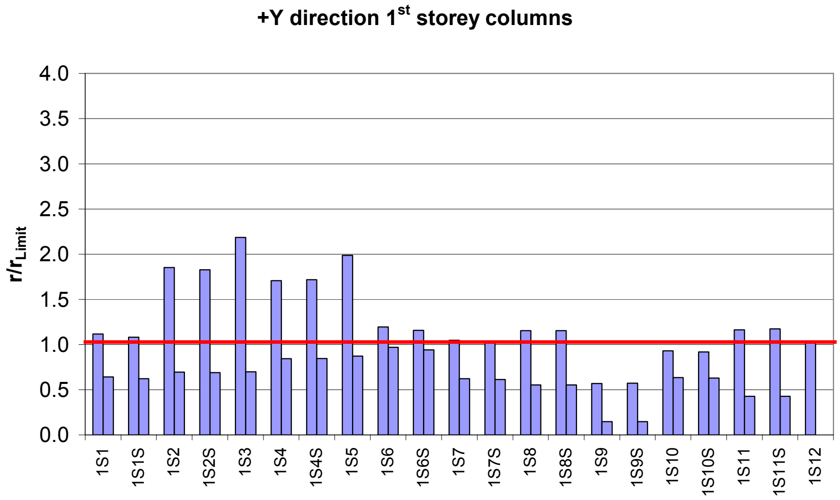
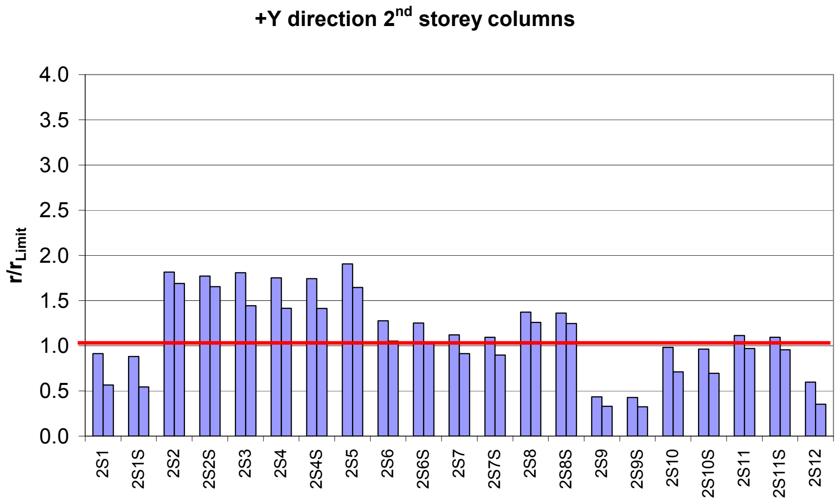


Figure 5.12 : r/r_{Limit} values (continued)





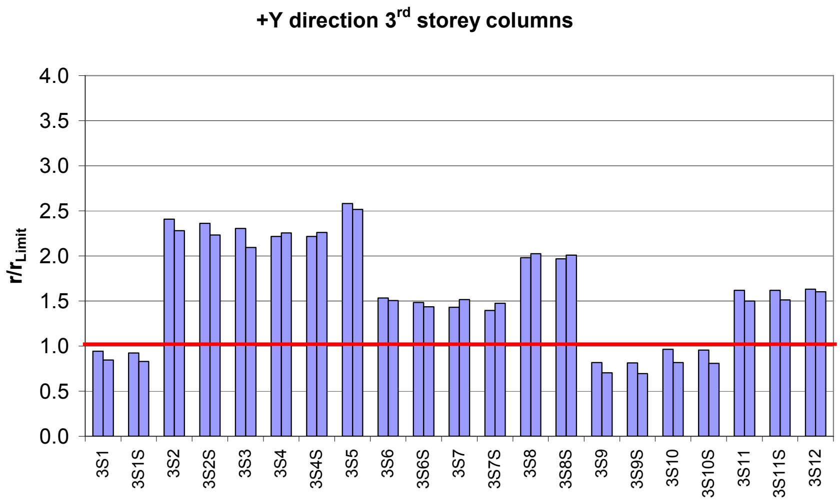
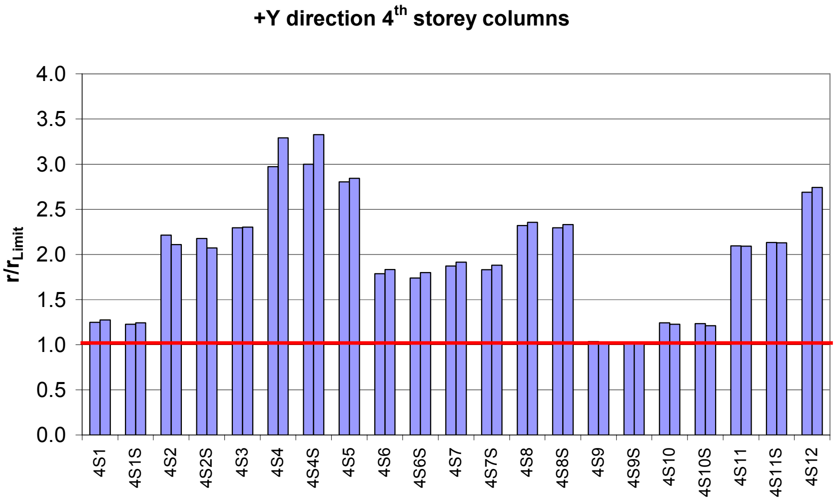
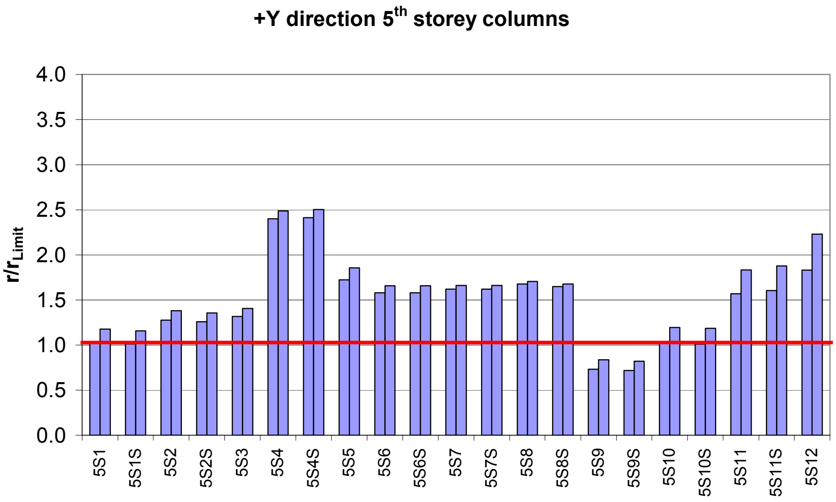


Figure 5.12 : r/r_{Limit} values (continued)





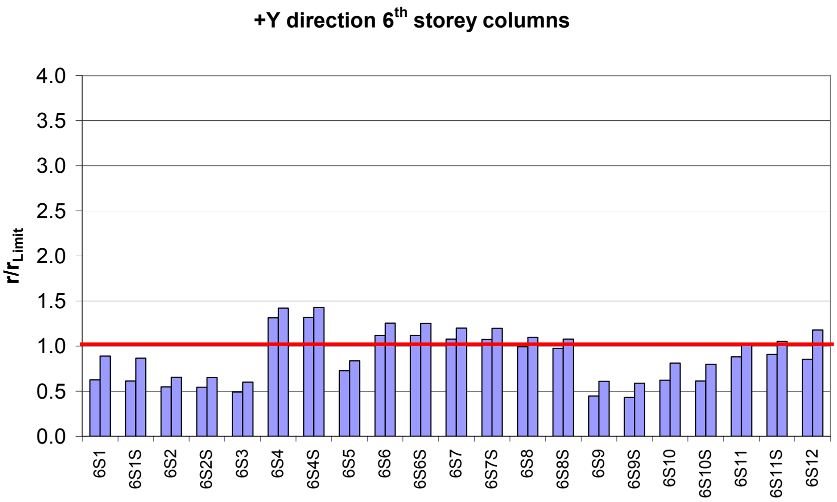
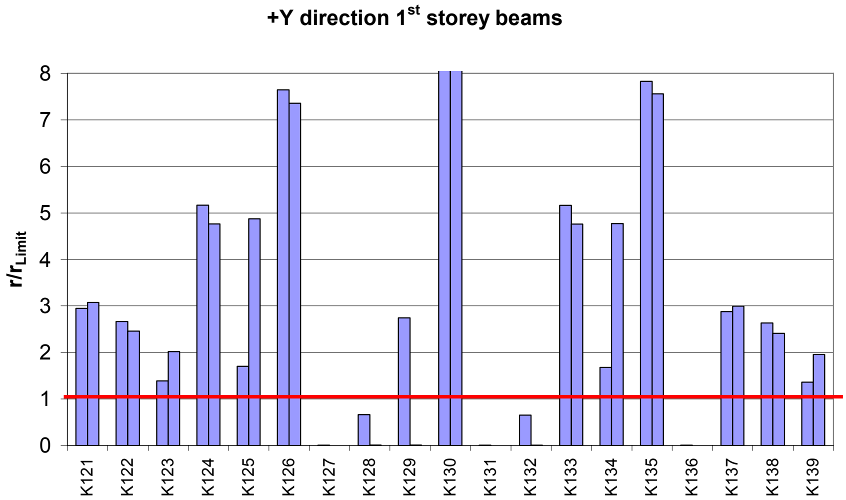
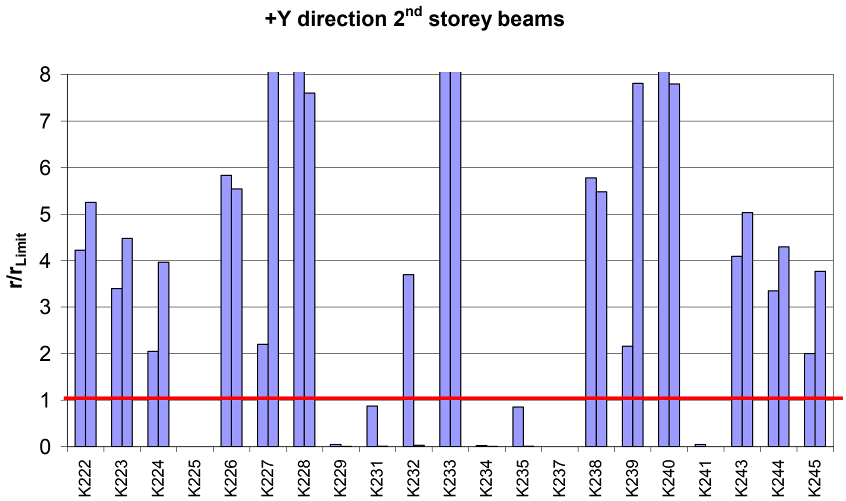


Figure 5.12 : r/r_{Limit} values (continued)





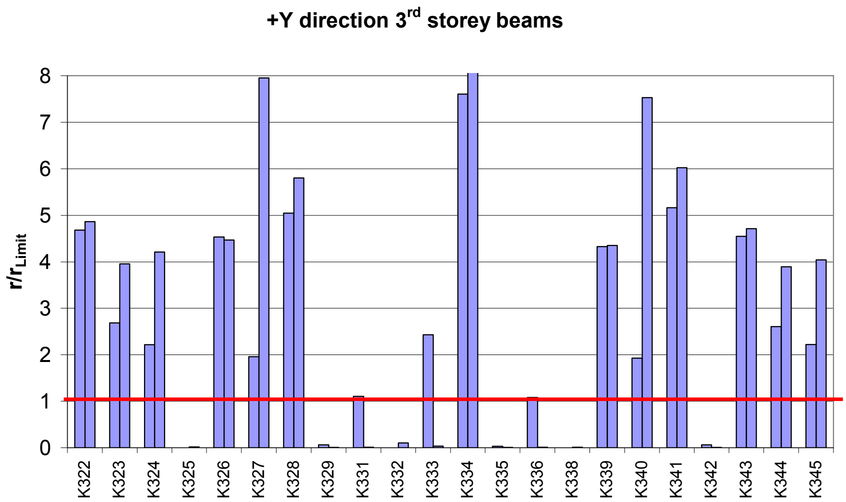
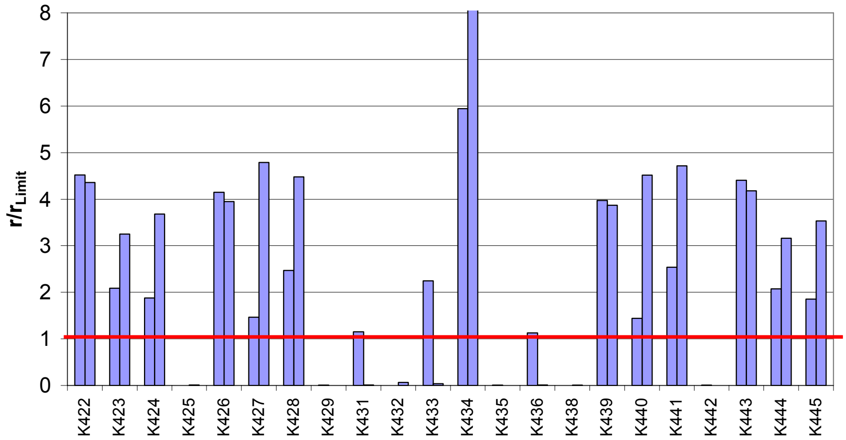
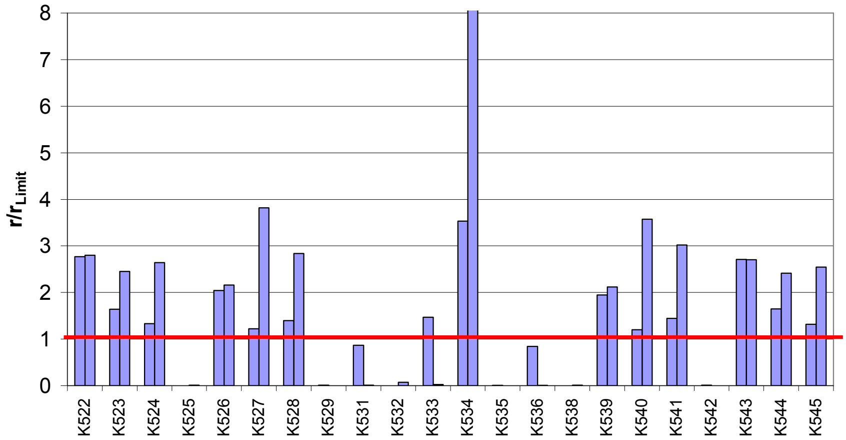


Figure 5.12 : r/r_{Limit} values (continued)

+Y direction 4th storey beams



+Y direction 5th storey beams



+Y direction 6th storey beams

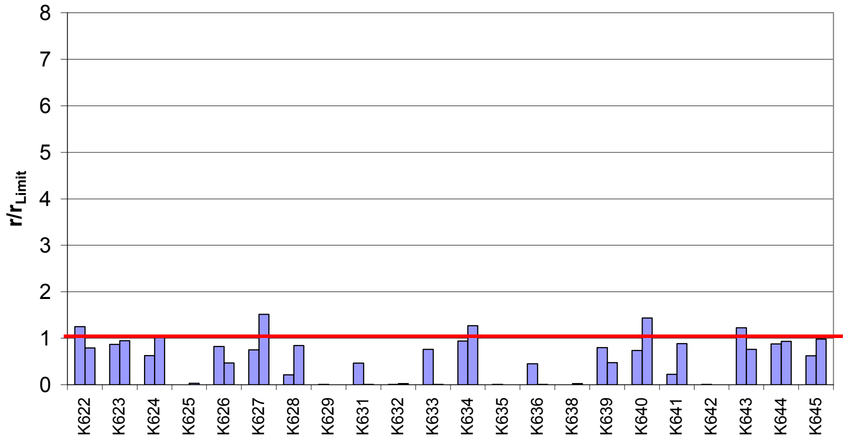


Figure 5.12 : r/r_{Limit} values (continued)

5.2.4 Global performance of the building

Global performance evaluation of the building for all stories and for two orthogonal directions are given in Table 5.7. Calculated interstorey drifts are given in Table 5.8.

Table 5.7 : Global performance of the members and the ratios of unacceptable members

	+X direction		+Y direction	
St	Columns (%)	Beams (%)	Columns (%)	Beams (%)
1	**94.7**	**94.1**	**79.6**	**87.5**
2	**88.7**	**100.0**	**85.6**	**87.5**
3	**97.7**	**100.0**	**85.6**	**100.0**
4	**96.6**	**100.0**	**93.0**	**100.0**
5	**90.4**	**84.6**	**92.4**	**87.5**
6	**49.2**	15.4	**57.0**	**31.3**

Table 5.8 : Storey drifts

	+X direction			+Y direction		
St	H_i (m)	$(\Delta_i)_{max}$	$(\Delta_i)_{max} / H_i$	H_i (m)	$(\Delta_i)_{max}$	$(\Delta_i)_{max} / H_i$
1	2.7	0.028712	0.010634	2.7	0.020511	0.007597
2	2.8	0.052176	0.018634	2.8	0.038220	0.013650
3	2.8	0.064994	0.023212	2.8	0.052133	0.018619
4	2.8	0.067697	0.024177	2.8	0.063578	0.022707
5	2.8	0.057935	0.020691	2.8	0.055894	0.019962
6	2.8	0.036204	0.012930	2.8	0.033523	0.011972

Global performance evaluation of the building was found to be ***not*** satisfactory for the Life Safety performance level.

5.3 Nonlinear Static Procedure

Three-dimensional model, storey masses and inertias, storey structural plans and rigid end zones are the same with the linear elastic model.

5.3.1 Modeling and Analysis

Moment-curvature relation was obtained and transferred to moment-plastic rotation relation as shown in Figure 5.13. Here φ_p represents plastic curvature and using plastic curvature, plastic rotation is obtained.

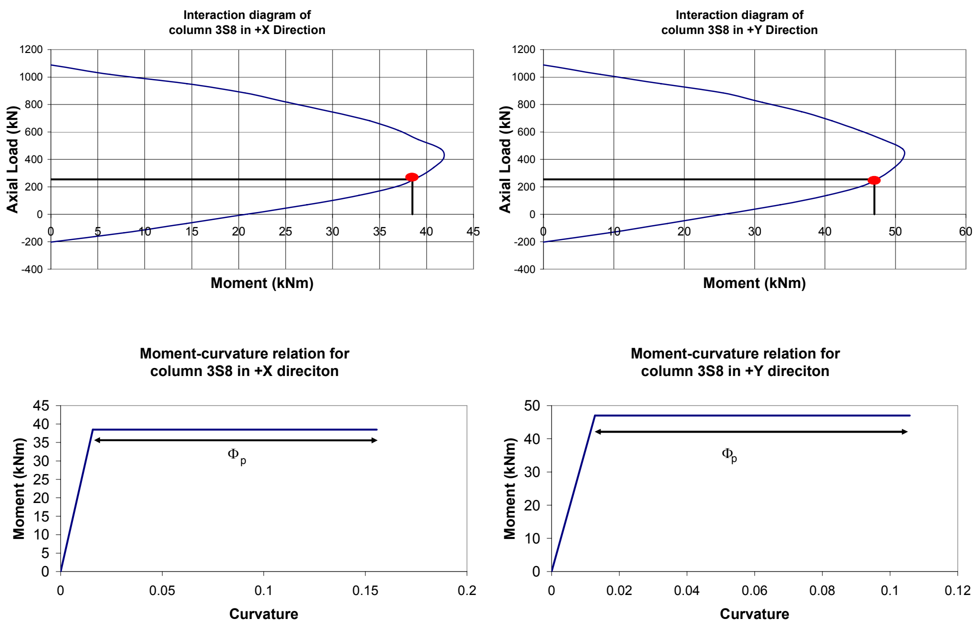


Figure 5.13 : Curvature to plastic rotation tranformation

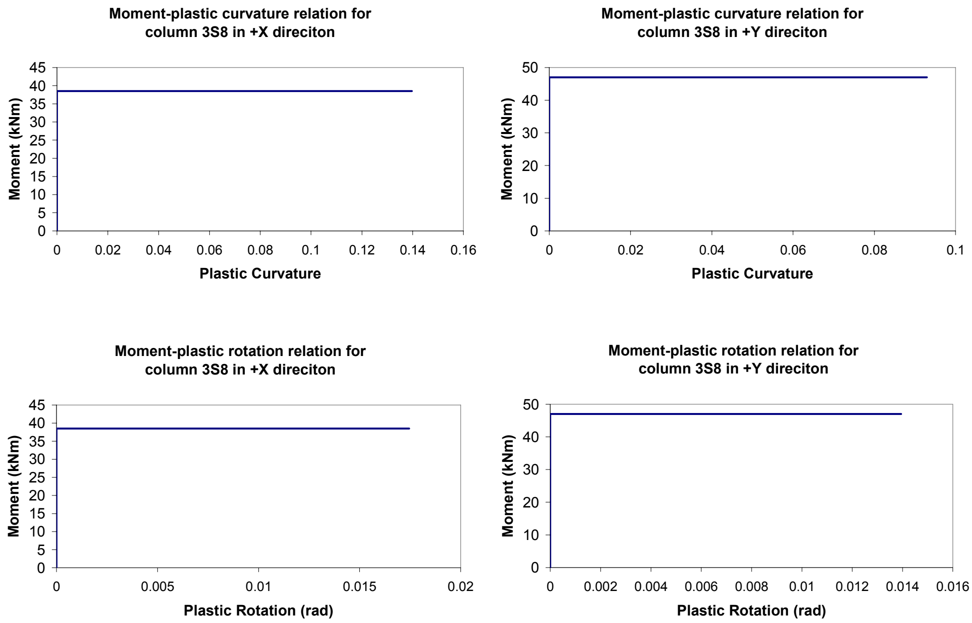


Figure 5.13 : Curvature to plastic rotation tranformation (continued)

Same transformation was done for positive and negative moments of all beams and for all columns. In addition, interaction surface was calculated for column members and assigned to the model.

Performing the nonlinear static analysis, capacity curve (pushover curve) for two orthogonal directions were obtained and shown in Figure 5.14 and Figure 5.15.

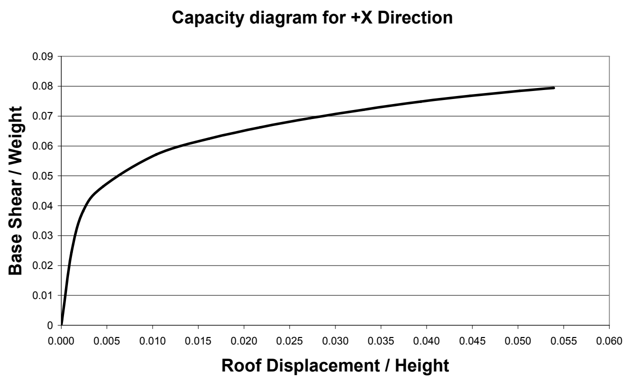


Figure 5.14 : Capacity curve in +X direction

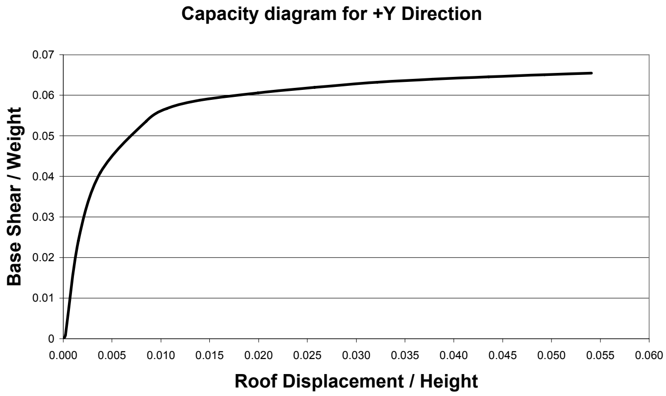
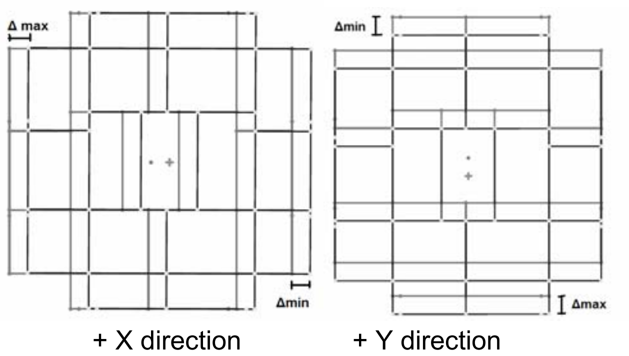


Figure 5.15 : Capacity curve in +Y direction



	+X	**+Y**
$(\Delta_i)_{max}$	0.011658	0.010746
$(\Delta_i)_{min}$	0.006338	0.007369
$(\Delta_i)_{mean}$	0.008983	0.009057
η_b	1.297764	1.186442

Figure 5.16 : Torsional irregularity check

Before starting the assessment process, building model must be controlled for the appropriateness of the equivalent lateral load procedure. Building has;

Total height of 16.7m which is less than 25m.

Total number of floors of 6 which is less than 8.

Maximum torsional irregularity coefficient as shown in Figure 5.16 (η_{bi}) was 1.3, which is less than 1.4.

Mass participation ratios were 70% and 72% for X and Y directions which are larger than 70%.

These four checks showed that building satisfies the requirements of equivalent lateral load procedure hence this procedure will be used.

5.3.2 Calculation of Target Displacement in +X and +Y directions

Target displacement of the building calculation was performed using the capacity curves and the procedure defined in the 2007 Turkish Earthquake Code. Natural vibration periods for both orthogonal directions of the building were larger than the corner period of the Code spectrum (T_B), thus equal displacement rule is valid and nonlinear target displacement is equal to the linear elastic displacement.

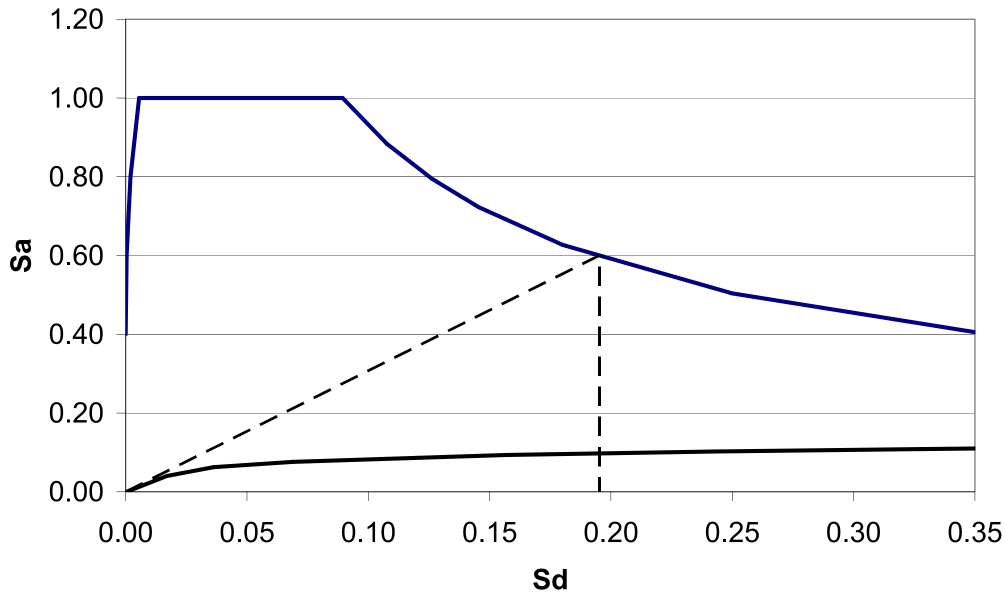


Figure 5.17 : Target displacement for +X direction

$T_x > T_B \rightarrow S_{di1} = S_{de1} = 0.195$

$u_{xN1} = \Phi_{xN1}\, \Gamma_{x1}\, S_{di1} = 0.04821 \times 23.03 \times 0.195$

$u_{xN1} = 0.217$ m

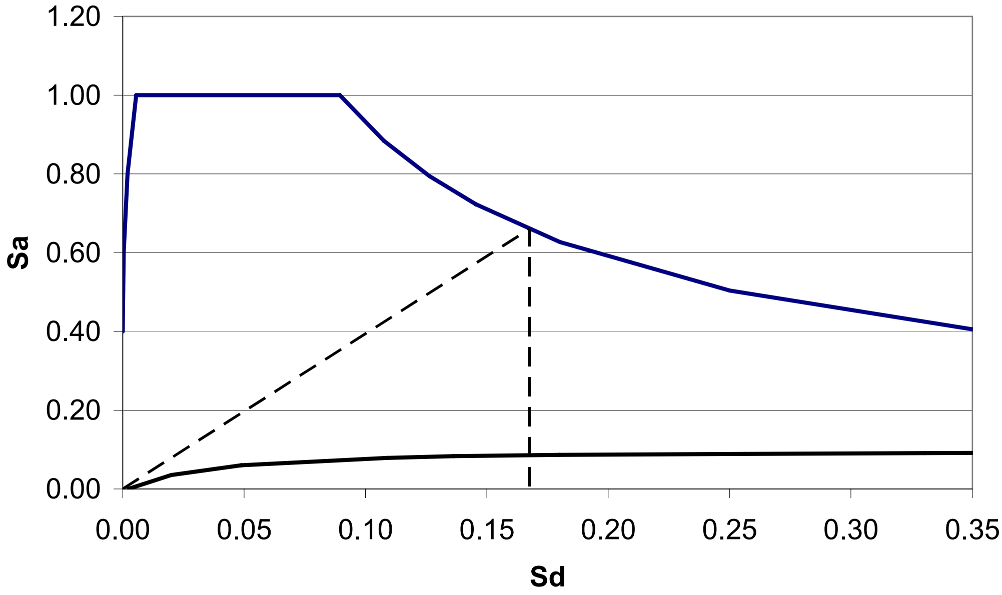


Figure 5.18 : Target displacement +Y direction

$T_y > T_B \rightarrow S_{di1} = S_{de1} = 0.167$

$u_{yN1} = \Phi_{yN1}\, \Gamma_{y1}\, S_{di1} = 0.05373 \times 24.21 \times 0.167$

$u_{yN1} = 0.217\ m$

5.3.3 Member capacities

Member shear capacity formulations are the same with linear elastic procedure with only one difference. Axial load needed to calculate shear capacity of column sections were obtained from the pushover analysis. Moreover, shear demands were obtained from pushover analysis for both beam and column sections. By comparison of shear demands with shear capacities, member failure type is classified as ductile or brittle.

If the member is brittle, then demand capacity ratio is shear demand over shear capacity. On the other hand, if the member is ductile, then concrete and steel strains must be found to compare them with strain limits of corresponding performance level.

Total curvature can be calculated from plastic rotation and strain values for concrete and steel are found using a sectional analysis. A sample calculation is given below for column 1S8.

$\theta_p = 0.002154$ rad

$$\phi_p = \frac{\theta_p}{L_p} = \frac{0.002154}{0.2 * 0.3} = 0.01436 \text{ 1/m}$$

$$\phi_t = \phi_y + \phi_p = 0.01269 + 0.01436 = 0.02705$$

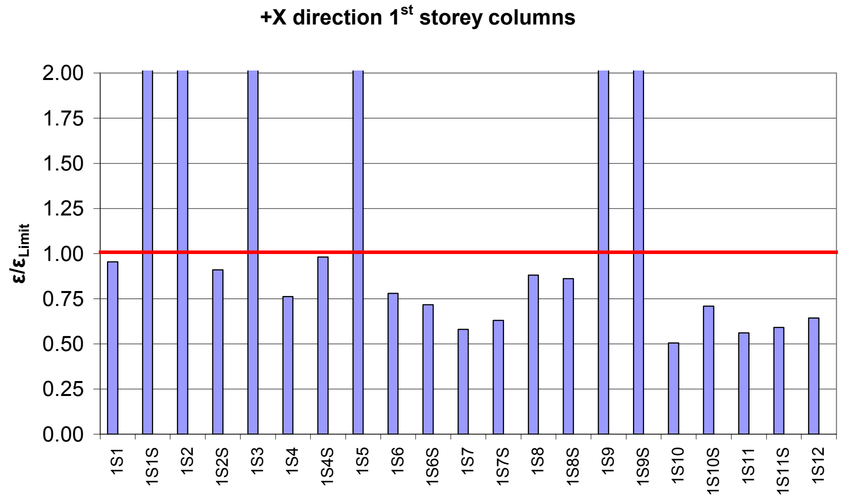
Strain values for total curvature were found as;

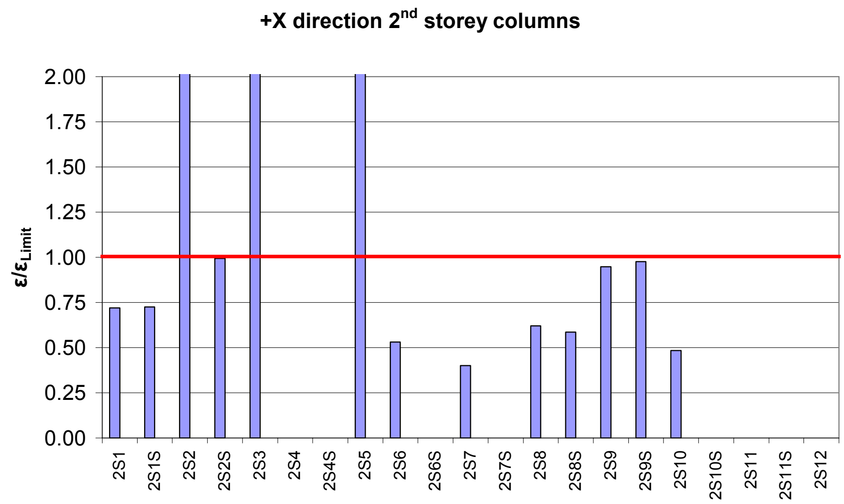
$\varepsilon_c = 0.0032$ $\quad\quad$ $\varepsilon_{c(LS)} = 0.0035$ $\rightarrow$ $\varepsilon_c / \varepsilon_{cg(LS)} = 0.91$

$\varepsilon_s = 0.003907$ $\quad\quad$ $\varepsilon_{s(LS)} = 0.040$ $\rightarrow$ $\varepsilon_s / \varepsilon_{s(LS)} = 0.098$

Strain to strain limit ratio for the section is the larger of concrete and steel ratios, which was 0.91. Upper end of the column 1S8 was not yielded so no calculation is needed for that end. Strain over strain limit vales were smaller than 1 for both ends of the column thus 1S8 was evaluated as acceptable for performance level of Life Safety.

Same calculation and evaluation process was done for all members and acceptability of the members were defined. ε / $\varepsilon_{\text{Limit}}$ values for all members and for all floors are given in Figure 5.19.





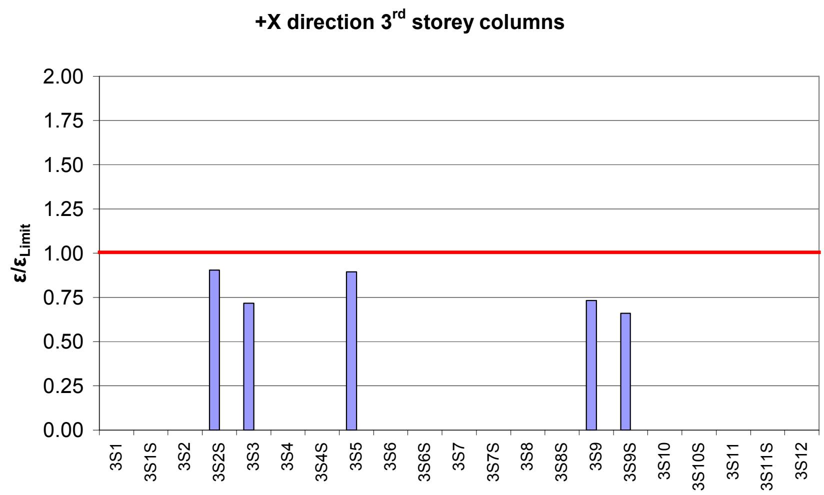
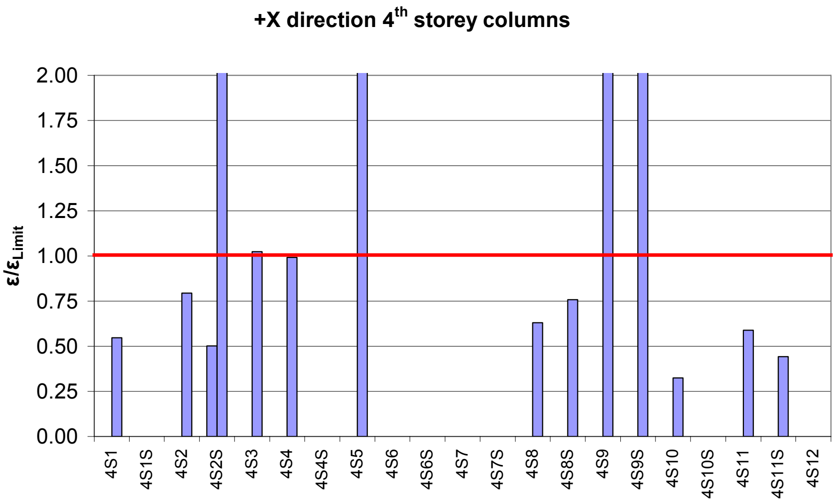
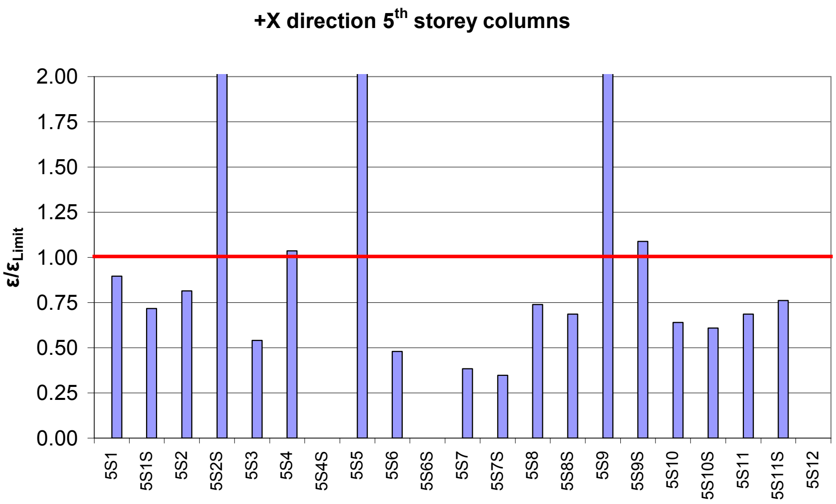


Figure 5.19 : ε / $\varepsilon_{\text{Limit}}$ values





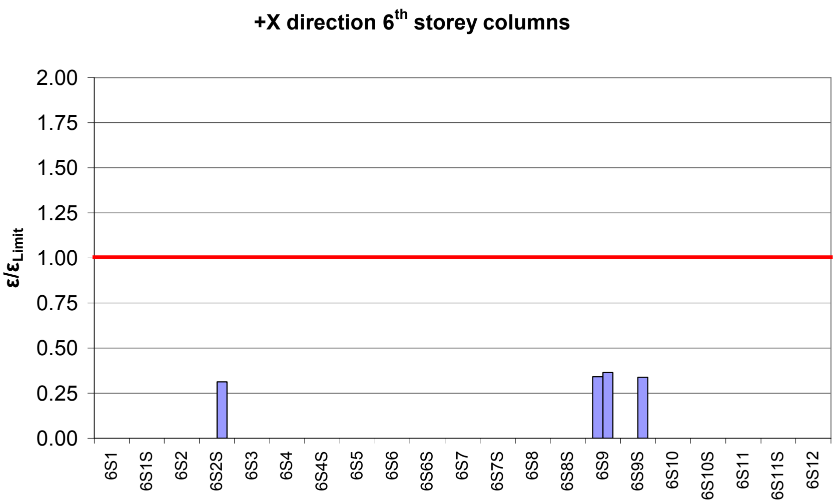
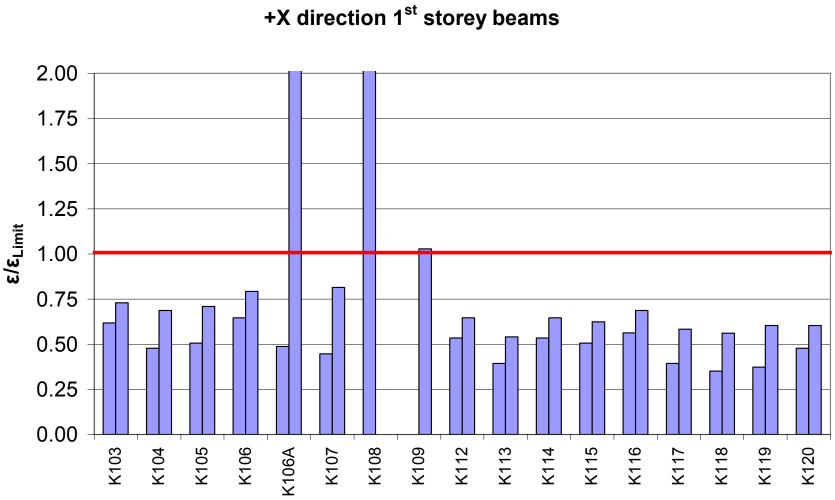
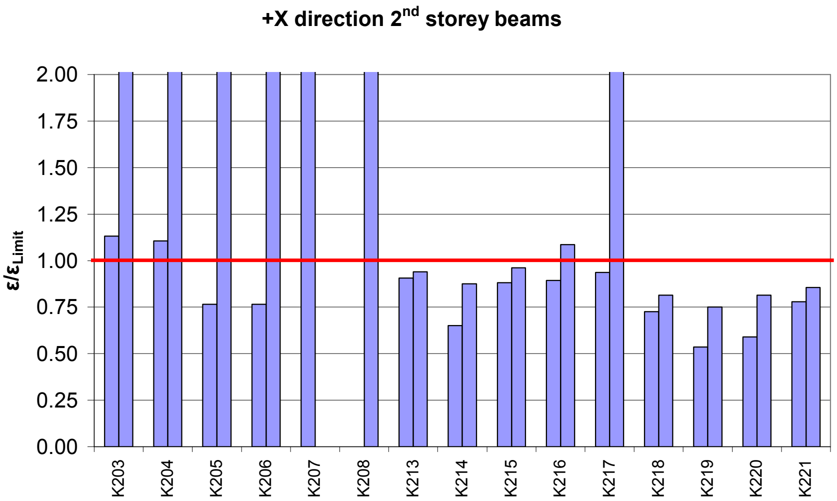


Figure 5.19 : ε / ε_{Limit} values (continued)





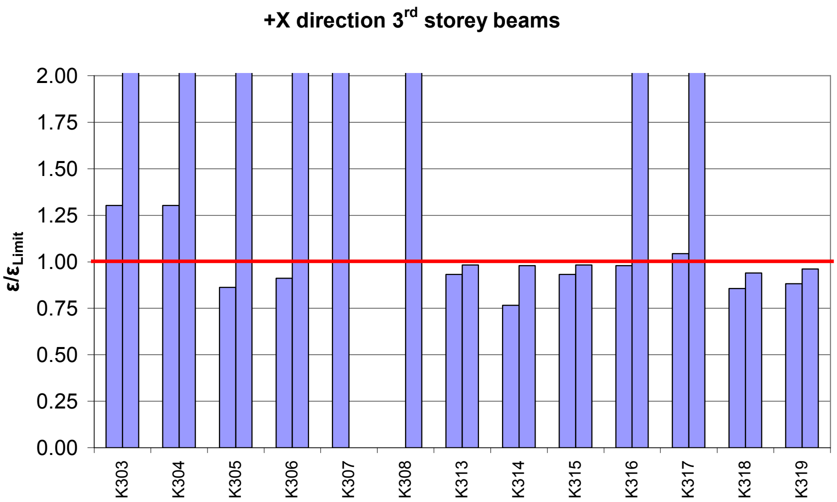
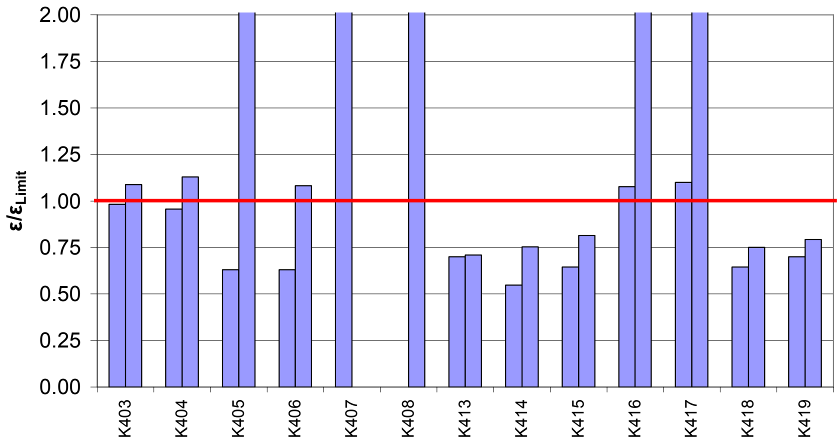
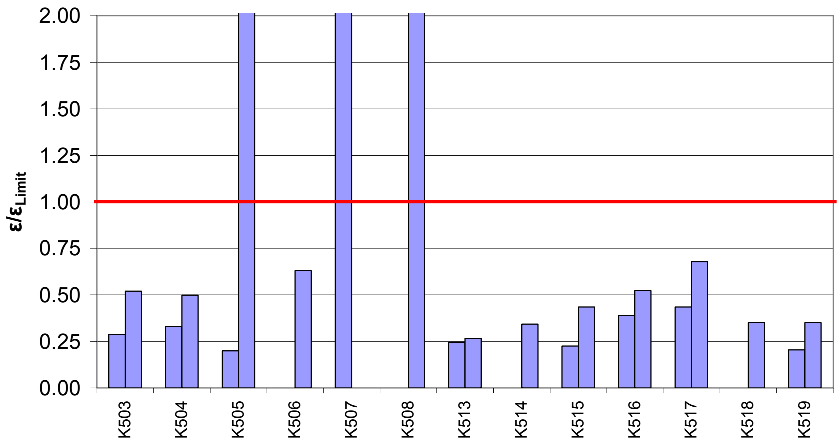


Figure 5.19 : ε / $\varepsilon_{\text{Limit}}$ values (continued)

+X direction 4th storey beams



+X direction 5th storey beams



+X direction 6th storey beams

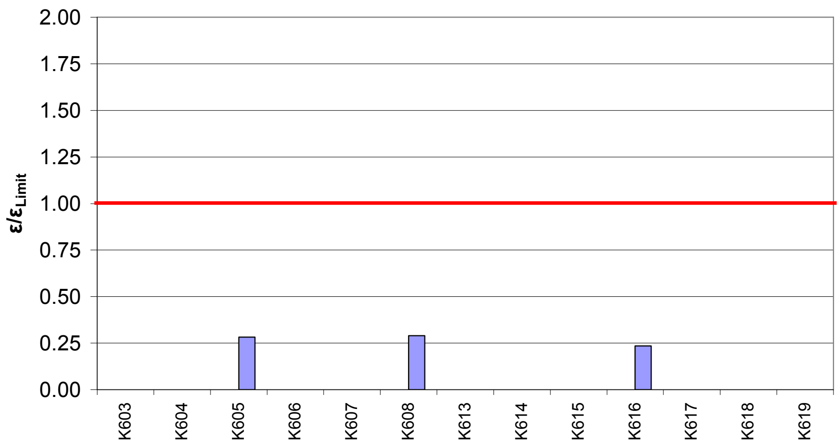
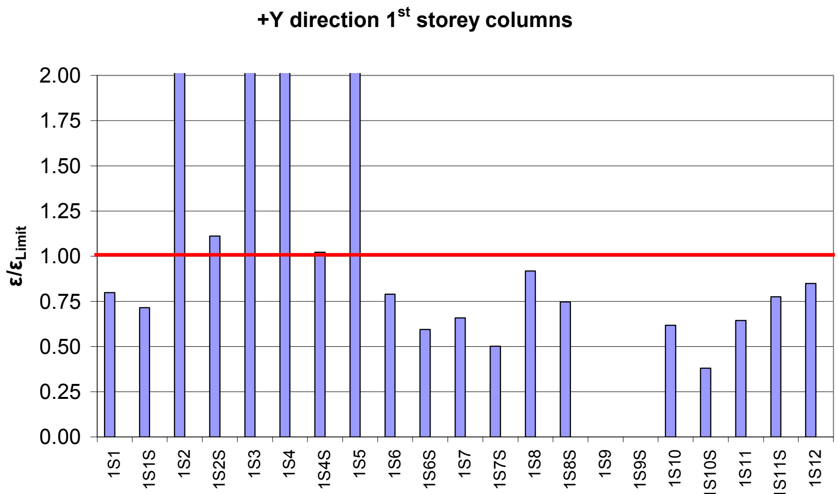
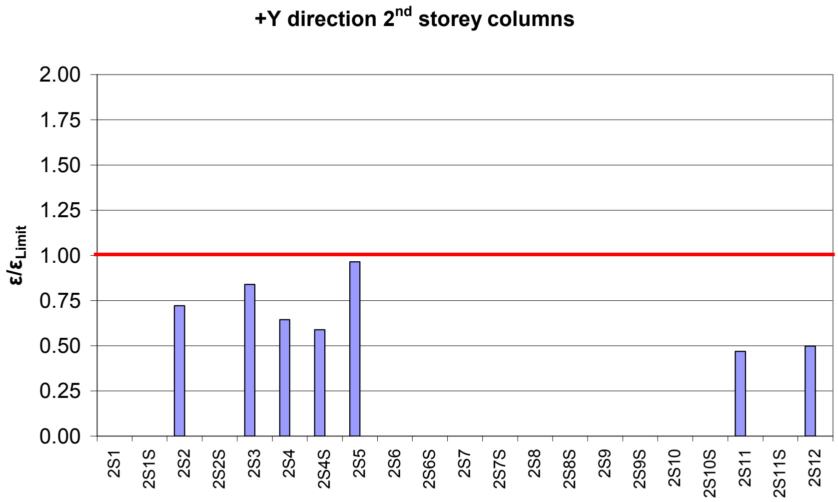


Figure 5.19 : ε / $\varepsilon_{\text{Limit}}$ values (continued)





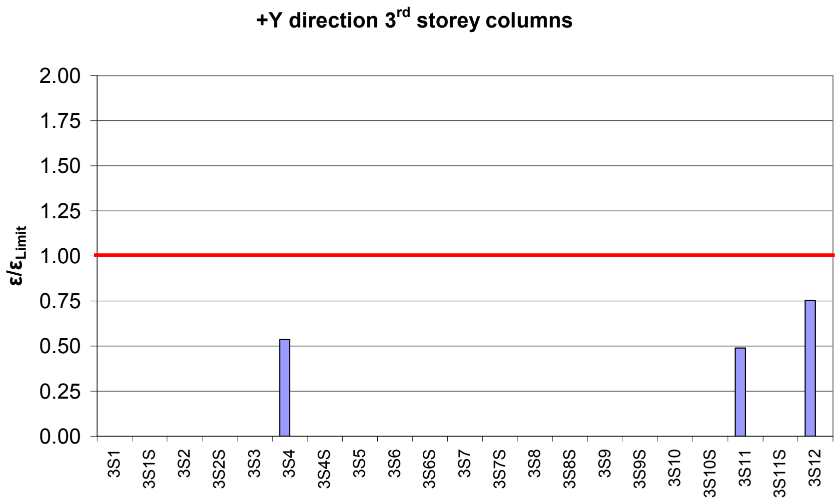
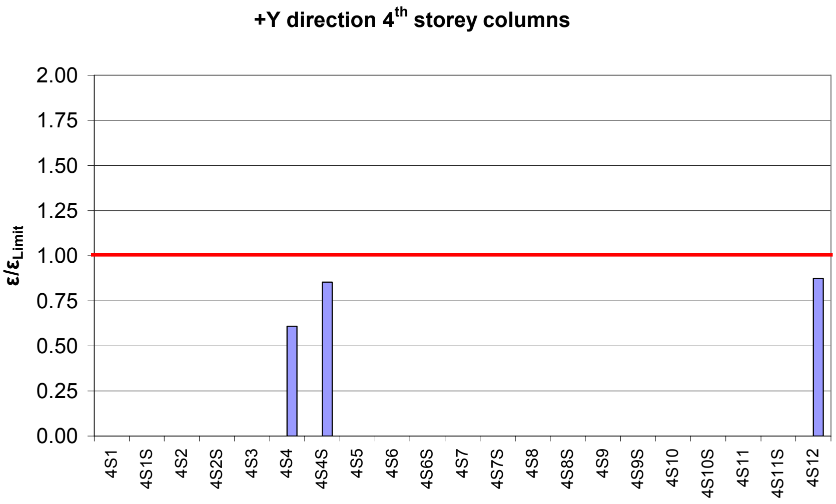
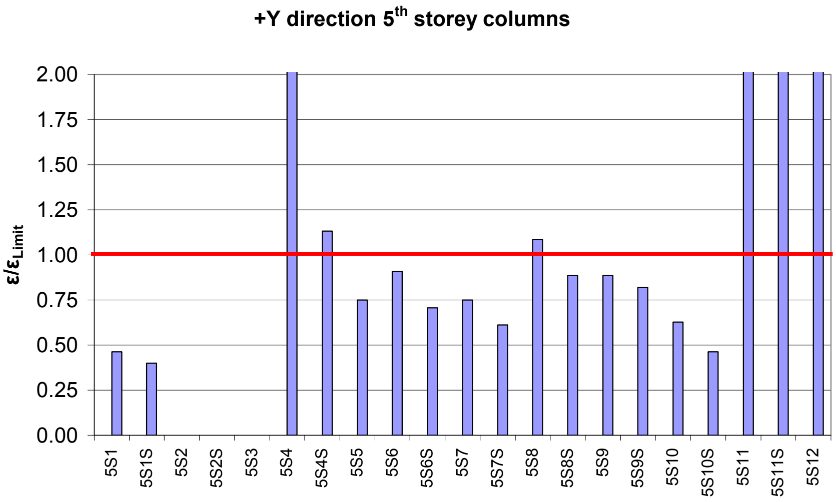


Figure 5.19 : $\varepsilon / \varepsilon_{Limit}$ values (continued)





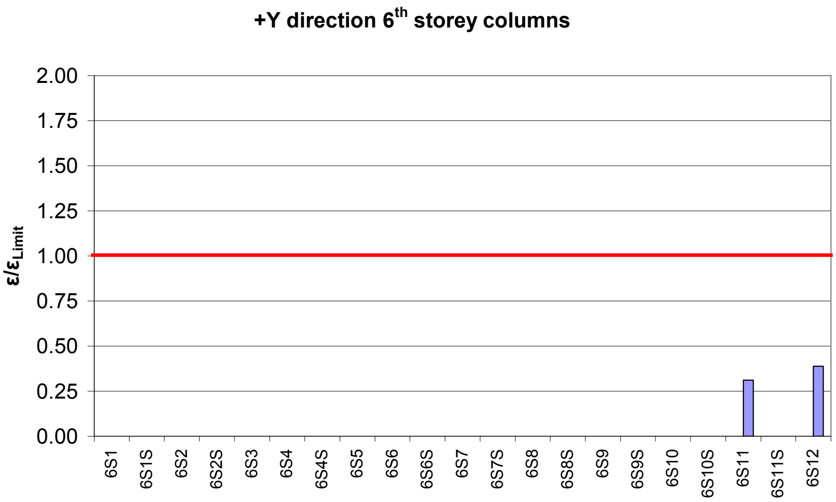


Figure 5.19 : ε / $\varepsilon_{\text{Limit}}$ values (continued)

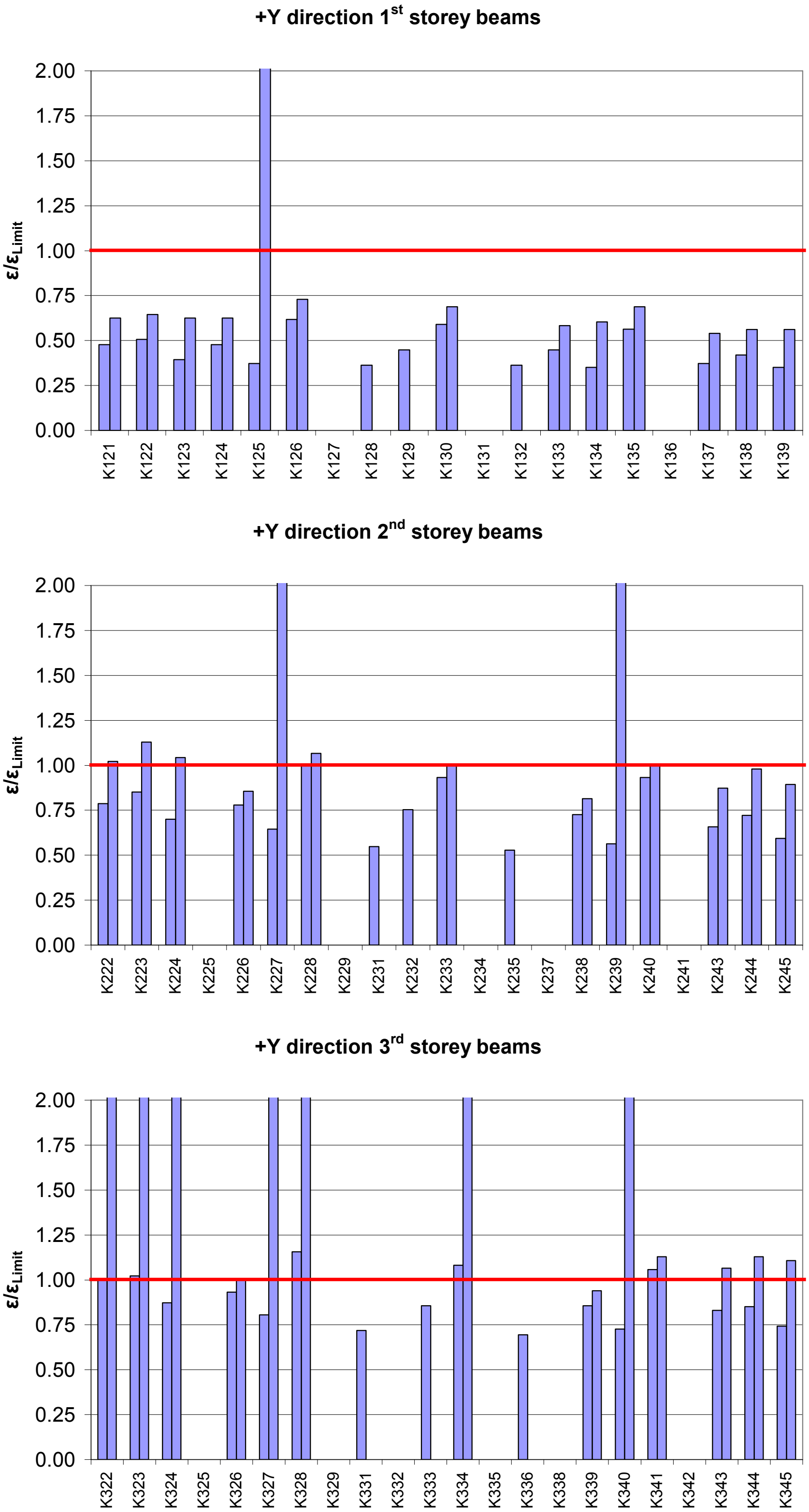
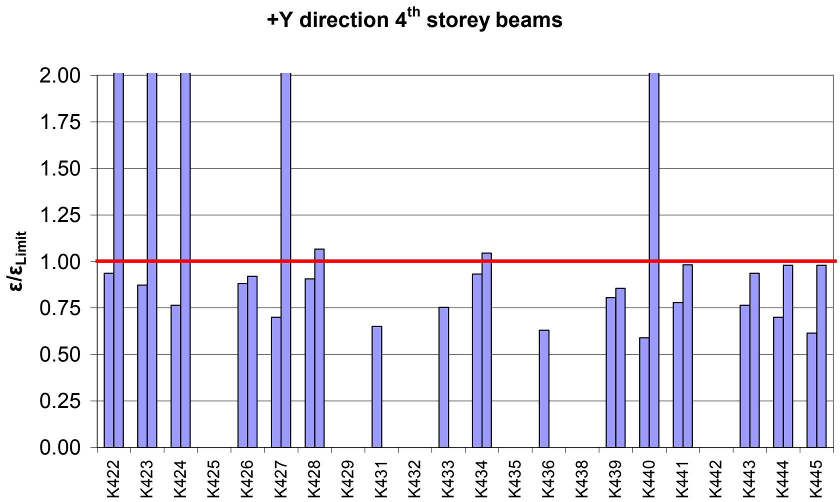
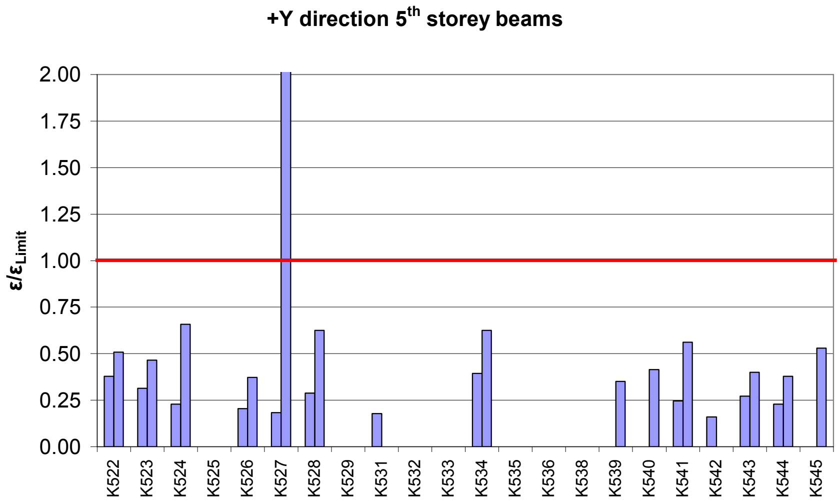


Figure 5.19 : ε / ε $_{Limit}$ values (continued)





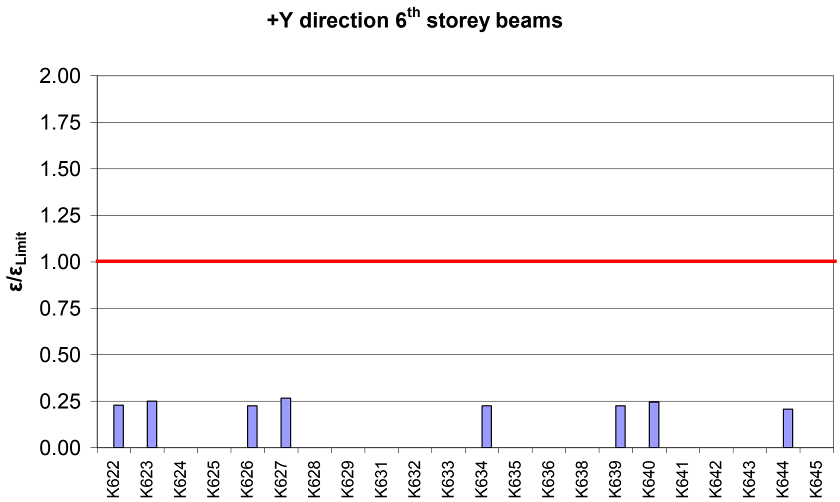


Figure 5.19 : ε / $\varepsilon_{\text{Limit}}$ values (continued)

5.3.4 Global Performance of the Building

Global performance evaluation of the building for all stories and for two orthogonal directions are given in Table 5.9. Calculated interstorey drift results are given in the Table 5.10.

Table 5.9 : Global performance of the members

	+X direction		+Y direction	
St	**Columns (%)**	**Beams (%)**	**Columns (%)**	**Beams (%)**
1	**37.9**	17.6	**29.0**	6.3
2	14.9	**53.3**	0.0	**50.0**
3	0.0	**61.5**	0.0	**75.0**
4	**33.5**	**61.5**	0.0	**43.8**
5	**41.1**	23.1	**43.8**	6.3
6	0.0	0.0	0.0	0.0

The reason of the drastic changes in unacceptable column shear percentages after third storey is that column dimensions were reduced from 40x25cm to 25x25cm, resulting in a lower moment capacity for upper stories. Global performance evaluation of the building was found to be ***not*** satisfactory for the Life Safety performance level.

Table 5.10 : Storey drifts

	+X direction			+Y direction		
St	**H_i (m)**	**$(\Delta_i)_{max}$**	**$(\Delta_i)_{max}$ / H_i**	**H_i (m)**	**$(\Delta_i)_{max}$**	**$(\Delta_i)_{max}$ / H_i**
1	2.7	0.024982	0.009253	2.7	0.018796	0.006962
2	2.8	0.053479	0.019099	2.8	0.038605	0.013787
3	2.8	0.068437	0.024442	2.8	0.054062	0.019308
4	2.8	0.065294	0.023319	2.8	0.057429	0.020510
5	2.8	0.041089	0.014675	2.8	0.040437	0.014442
6	2.8	0.011658	0.004164	2.8	0.010746	0.003838

CHAPTER VI

CASE STUDY 4 : BUILDING IN CASE STUDY 3 RETROFITTED WITH EXTERIOR COUPLED SHEAR WALLS

In this case study, the six-storey existing residential building in Case Study 3 which was retrofitted with exterior coupled shear walls using both linear elastic and nonlinear procedures described in the 2007 Turkish Earthquake code. Moreover, anchorage design of the coupled walls was given. Detailed calculations are not shown here since they were presented in the previous chapter.

6.1 Description of the Retrofit Option

Structural properties of the building was defined in the previous chapter. Two pairs of coupled exterior shear walls were added to the system in both orthogonal directions. TP and TK designations are used for new walls and beams respectively. Structural properties and associated parameters for the 2007 Turkish Earthquake Code are tabulated in Table 6.1 and location of the shear walls are illustrated in Figure 6.2.

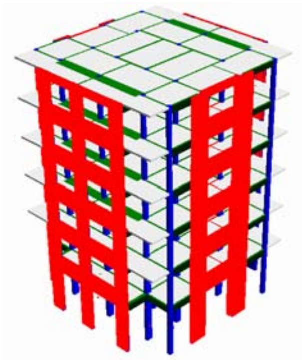


Figure 6.1 : 3D model of the building

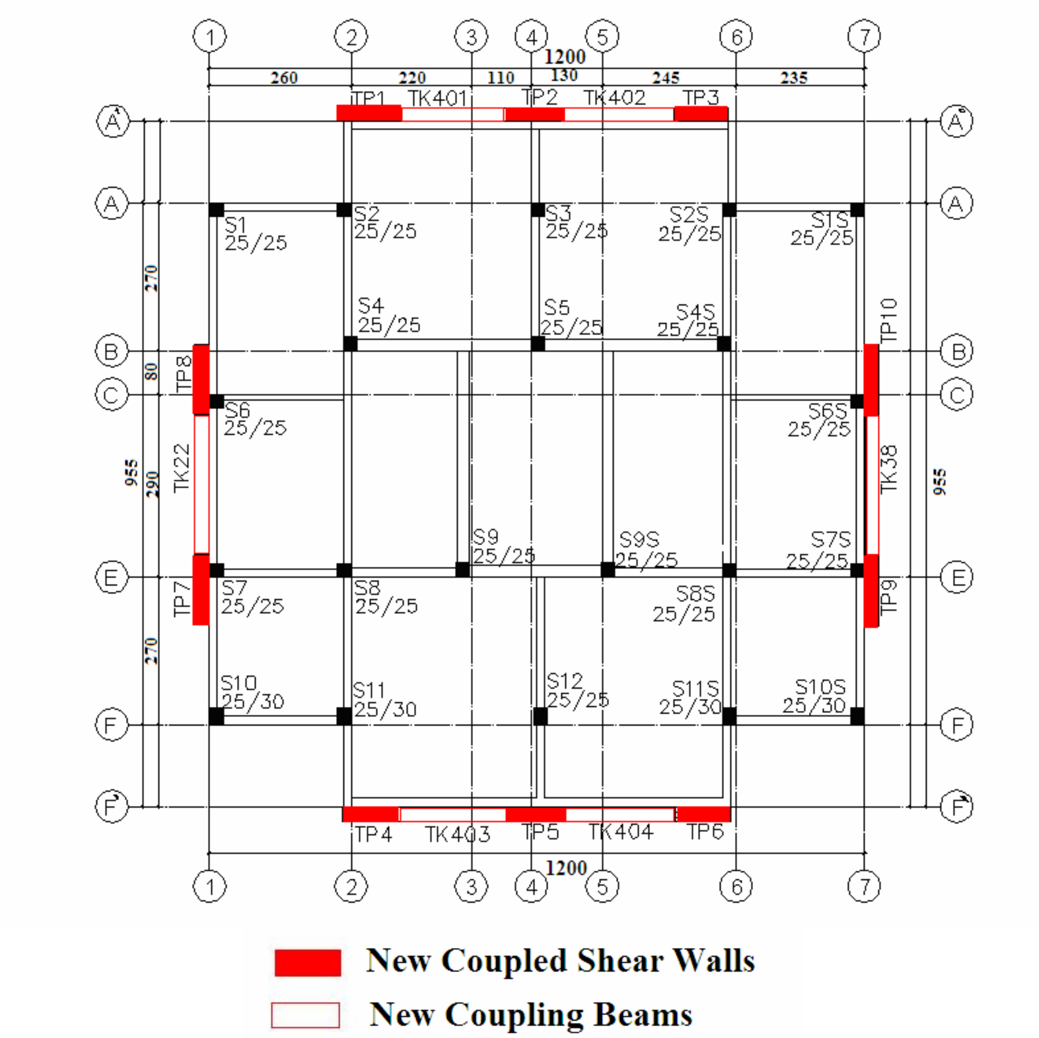


Figure 6.2 : Typical floor plan of the retrofitted system

Table 6.1 : Structural system properties and Code parameters

Building Properties	
Project available?	Yes
Knowledge level	Extensive
Knowledge level factor	1
Existing concrete strength	14 MPa (E_c = 26150 MPa)
Retrofitting concrete strength	25 MPa (E_c = 30250 MPa)
Existing reinforcement strength	300 MPa
Retrofitting reinforcement strength	420 MPa

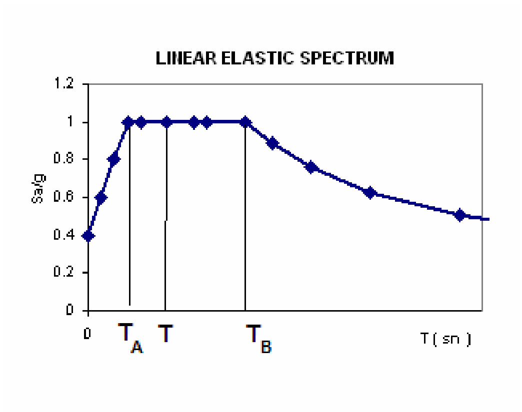
2007 Turkish Earthquake Code Parameters	
Seismic Zone	1
Seismic Zone Factor (A_o)	0.4
Building Importance Factor	1
Soil Class	Z3
Live Load Participation Factor	0.3
Target Performance Level	Life Safety

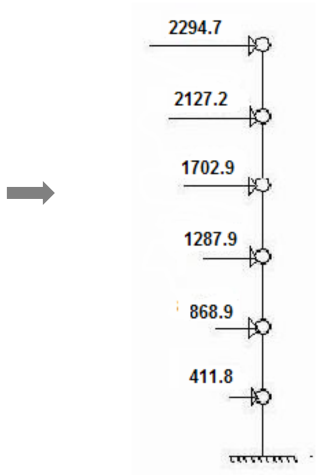
Section dimensions were given in Chapter 5. The dimensions for coupled shear walls are 100x25cm for walls in X direction, 130x25cm for walls in Y direction. Modal properties of the retrofitted building are given in Table 6.2.

Table 6.2 : Modal properties of the retrofitted building

MODE	PERIOD	INDIVIDUAL MODE (PERCENT)			CUMULATIVE SUM (PERCENT)		
		UX	UY	UZ	UX	UY	UZ
1 (Y)	0.44817	0.0003	77.3729	0.0001	0.0003	77.3729	0.0001
2 (X)	0.373738	80.8114	0.0003	0.0000	80.8117	77.3732	0.0001
3	0.261813	0.0055	0.0000	0.0000	80.8172	77.3732	0.0001
4	0.146614	0.0000	13.9419	0.0015	80.8173	91.3151	0.0016
5	0.122833	14.4131	0.0000	0.0000	95.2303	91.3151	0.0016
6	0.090251	0.0000	0.0002	16.2681	95.2303	91.3153	16.2697

First mode periods are illustrated with the Code spectrum in Figure 6.3.





$T_A = 0.15s$, $T_B = 0.6s$, $T_{1x} = 0.37s$, $T_{1y} = 0.45s$

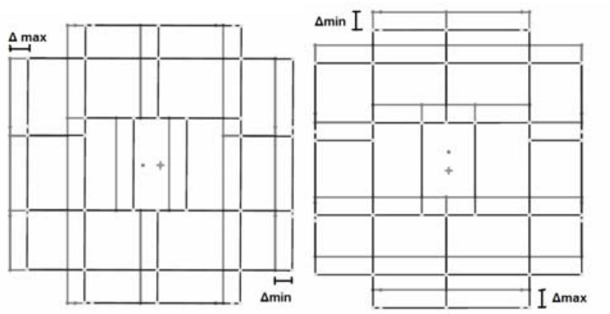
Figure 6.3 : Code spectrum and lateral load distribution

Equivalent lateral load distribution according to the Code spectrum is shown in Table 6.3.

Table 6.3 : Equivalent static lateral load distribution

St	Weight (W_i) (kN)	Height (m)	H_i (m)	$W_i H_i$ (kNm)	F_i (kN)
1	1760.2	2.7	2.7	4752.6	411.8
2	1823.6	2.8	5.5	10029.7	868.9
3	1791.0	2.8	8.3	14865.4	1287.9
4	1770.8	2.8	11.1	19655.9	1702.9
5	1766.4	2.8	13.9	24552.8	2127.2
6	1315.6	2.8	16.7	21970.8	2294.7

Torsional irregularity check has been done before the assessment procedure.



+ X direction + Y direction

	+X	+Y
$(\Delta_i)_{max}$	0.00939	0.01499
$(\Delta_i)_{min}$	0.00919	0.01497
$(\Delta_i)_{ort}$	0.00929	0.01498
η_b	1.01092	1.00096

Figure 6.4 : Torsional irregularity check

Retrofitted system was found suitable for the equivalent static lateral load method.

6.2 Linear Elastic Procedure

Linear elastic procedure calculations were represented in detail in Chapter 5, here only the r/r_{Limit} values and global performance of the structure will be introduced.

6.2.1 Comparison of Demand / Capacity Ratios (r) with Limit Values (r_{Limit})

Demand-Capacity ratios were calculated with the procedure described previously. Limit values were obtained from Table 2.2 for columns, Table 2.3 for beams and Table 2.4 for shear walls respectively with the calculated parameters. Normalized values of r / r_{Limit} are illustrated in Figure 6.5 and Figure 6.6.

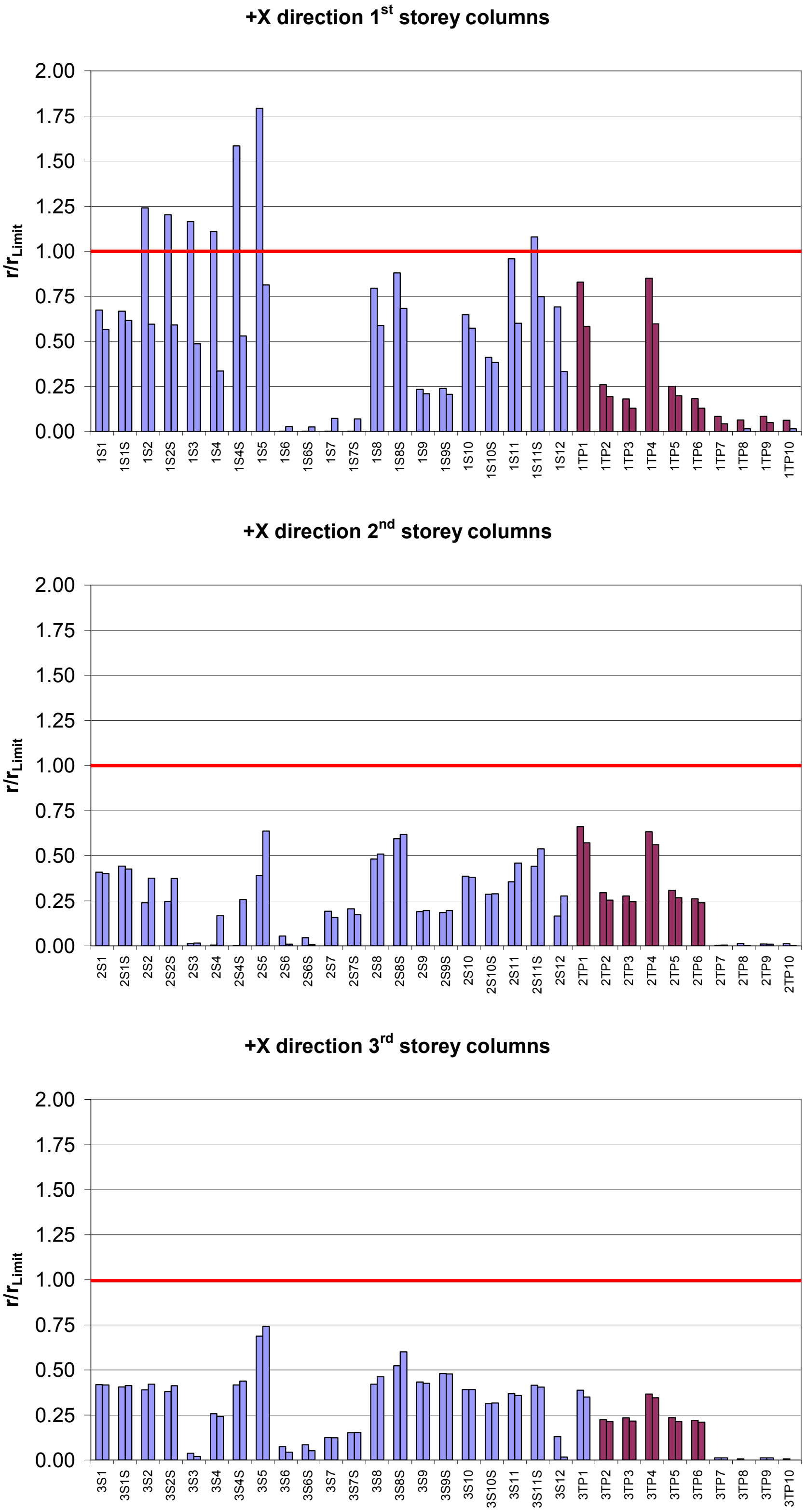


Figure 6.5 : r/r_{Limit} values

All r/r_{Limit} values of the columns and walls at stories 4, 5, 6 are smaller than 1, so they are not shown here.

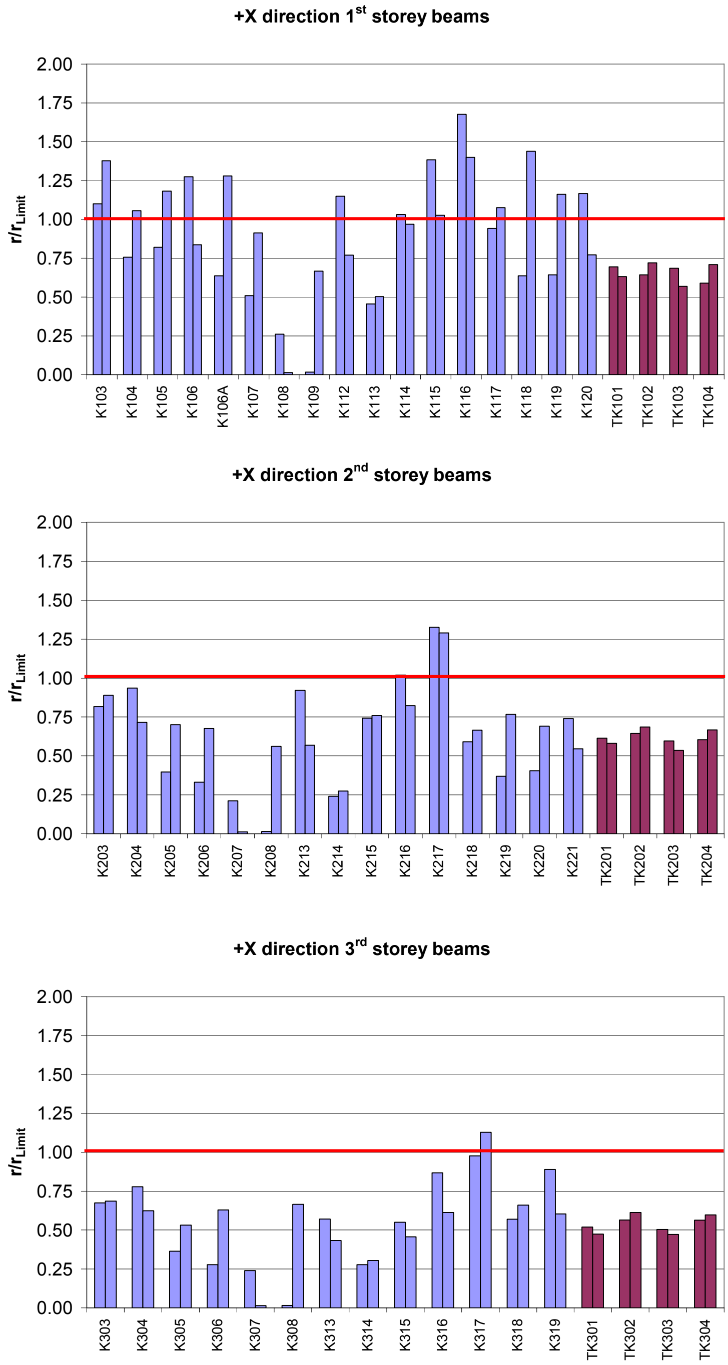
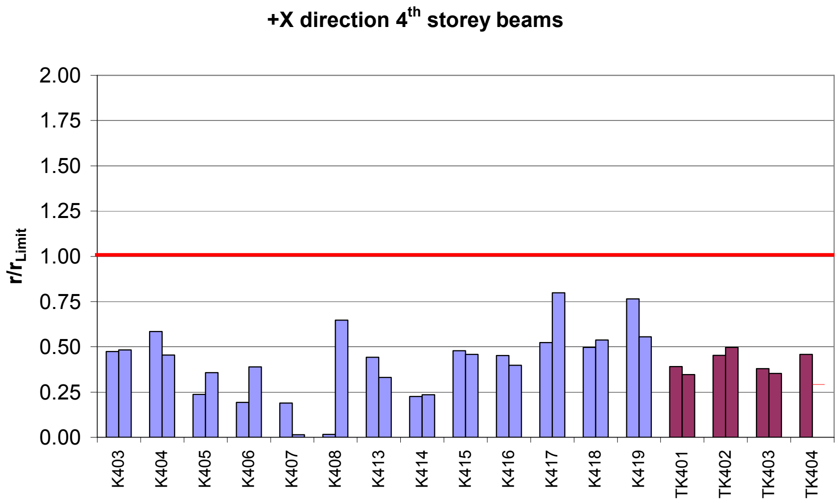
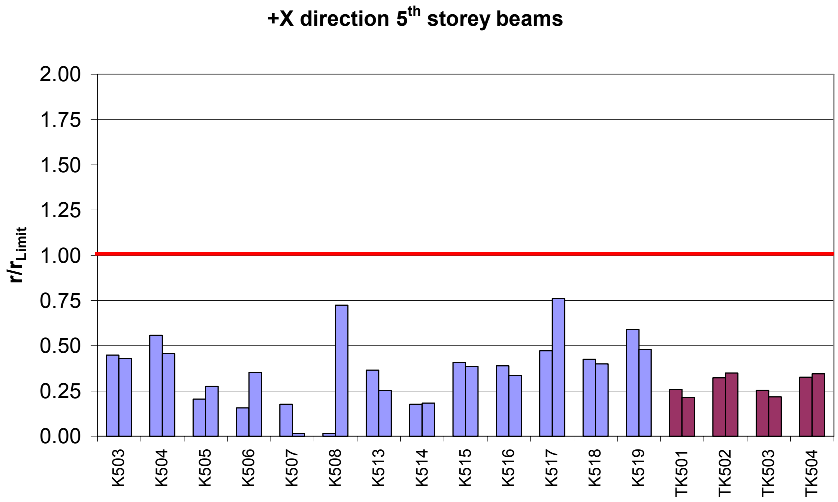


Figure 6.6 : r/r_{Limit} values





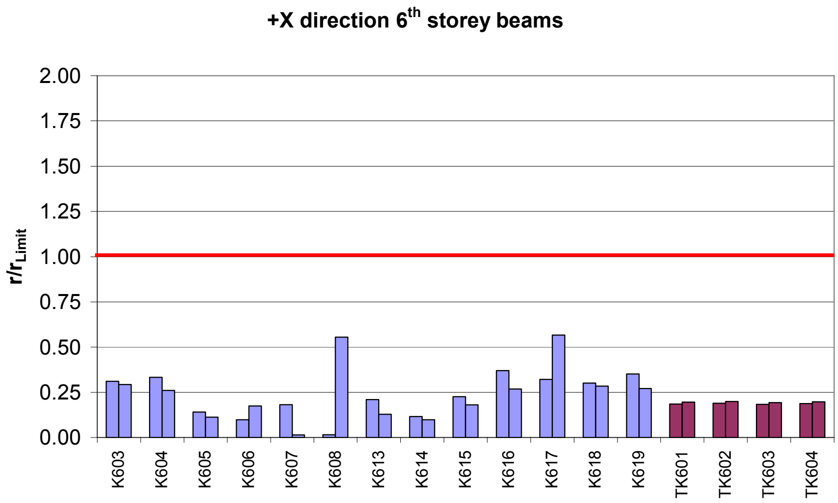
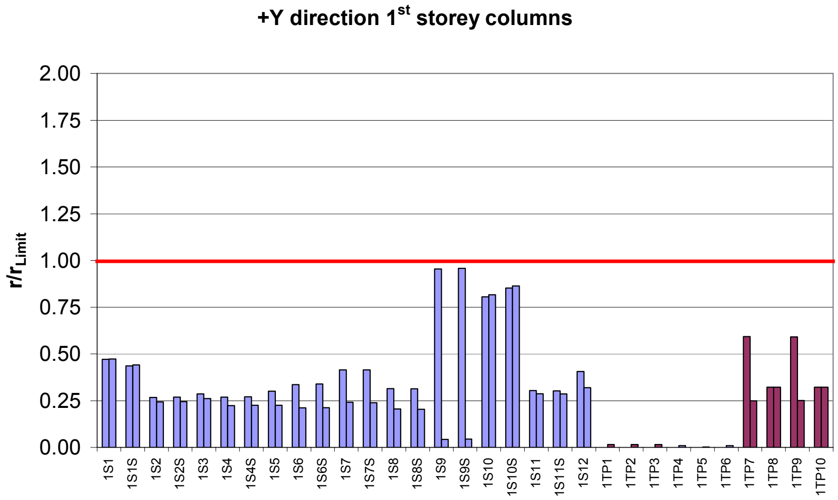
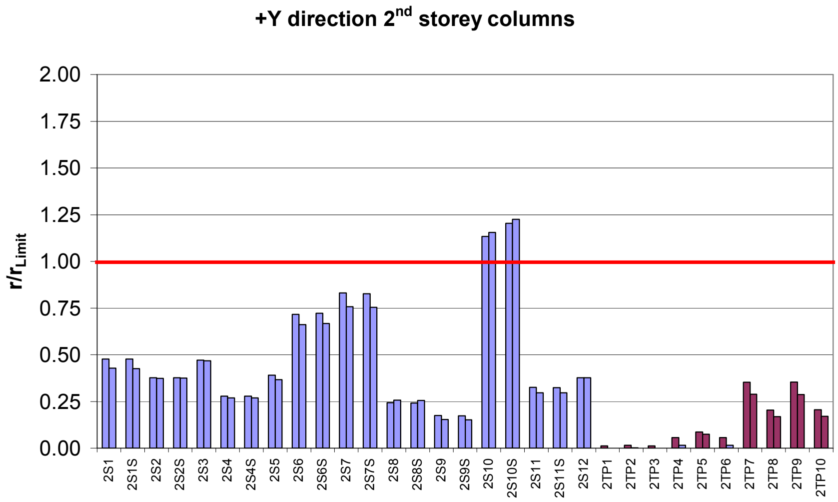


Figure 6.6 : r/r_{Limit} values (continued)





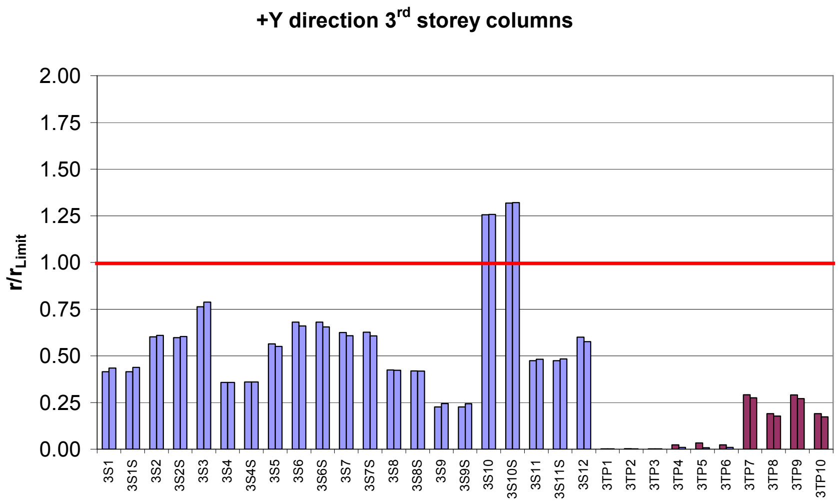
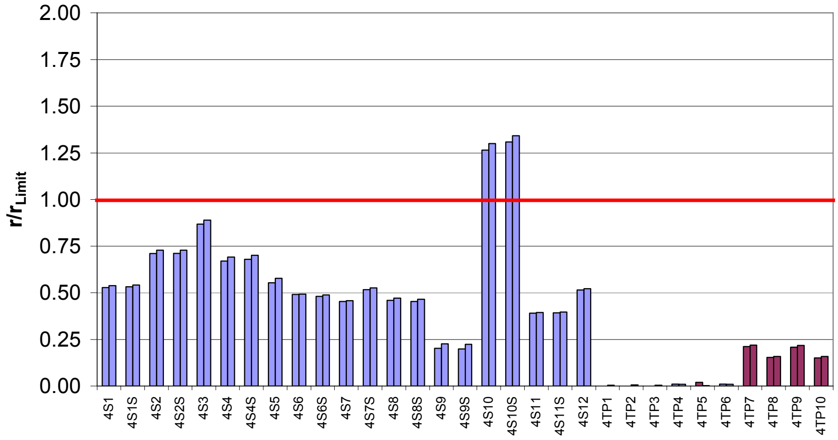
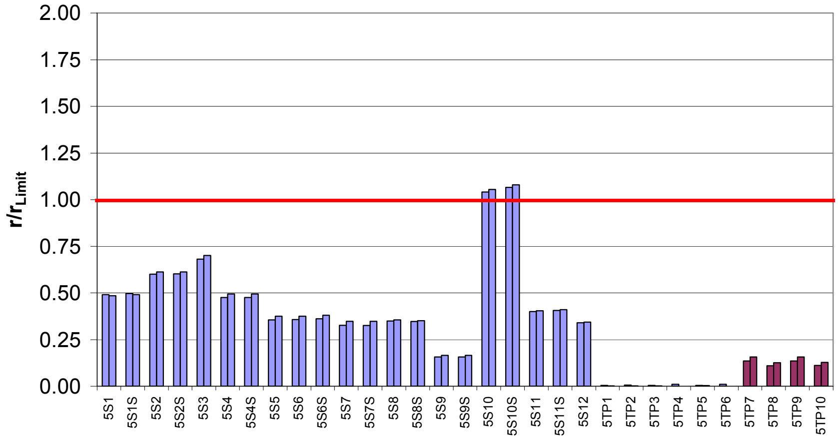


Figure 6.6 : r/r_{Limit} values (continued)

+Y direction 4th storey columns



+Y direction 5th storey columns



+Y direction 6th storey columns

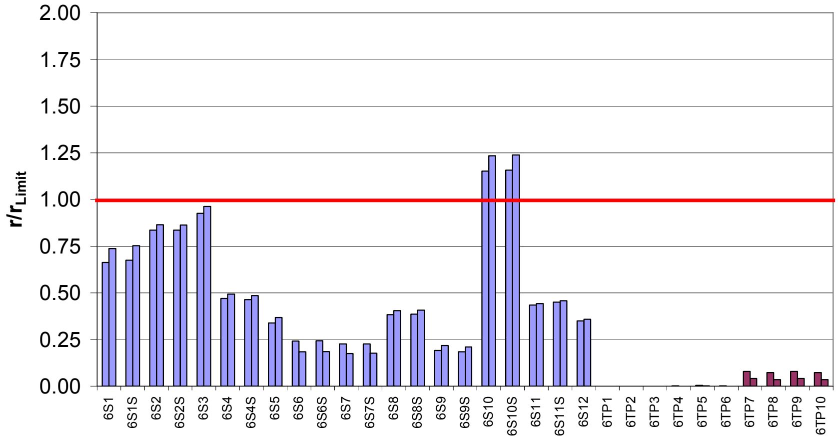


Figure 6.6 : r/r_{Limit} values (continued)

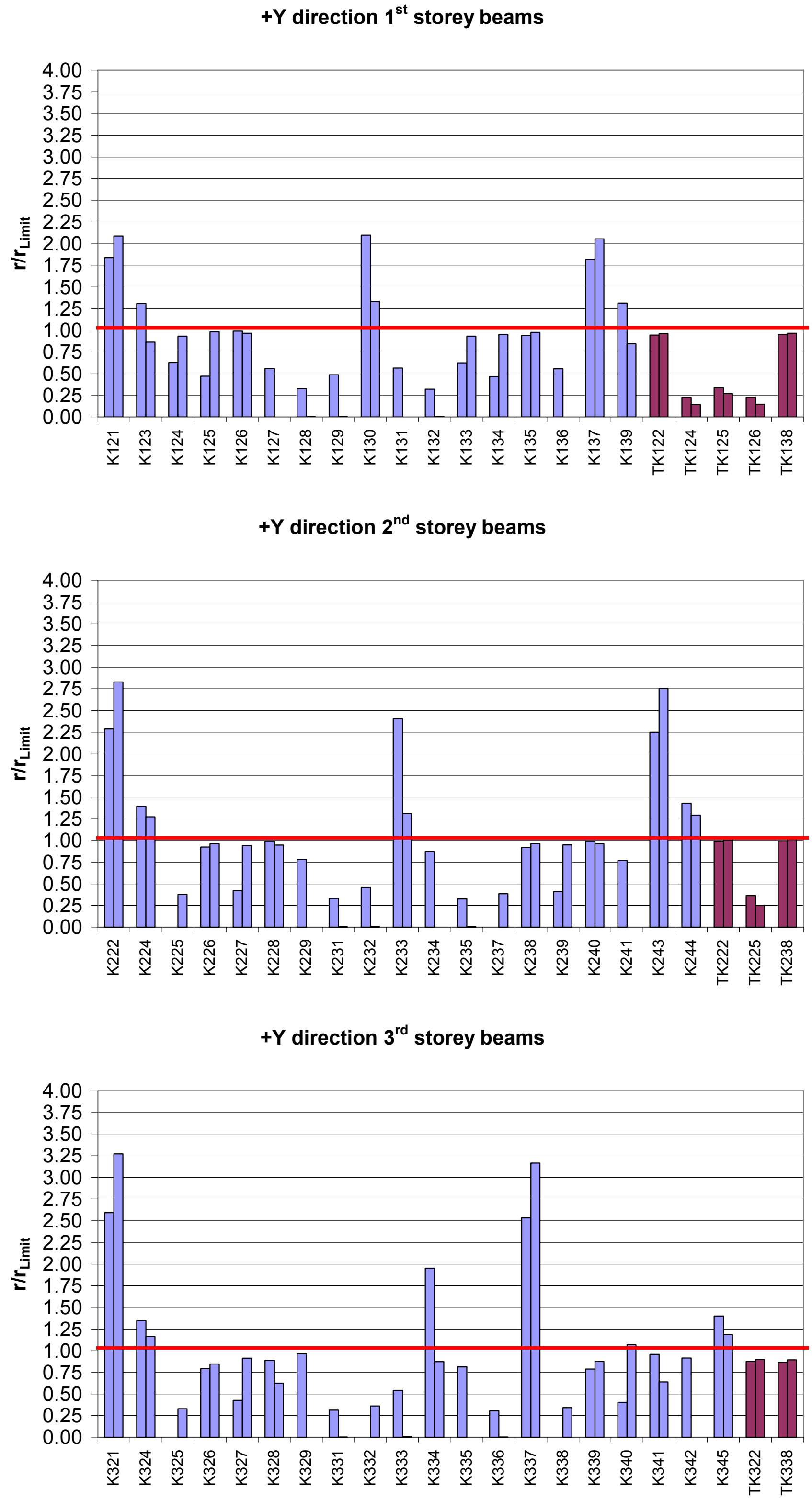
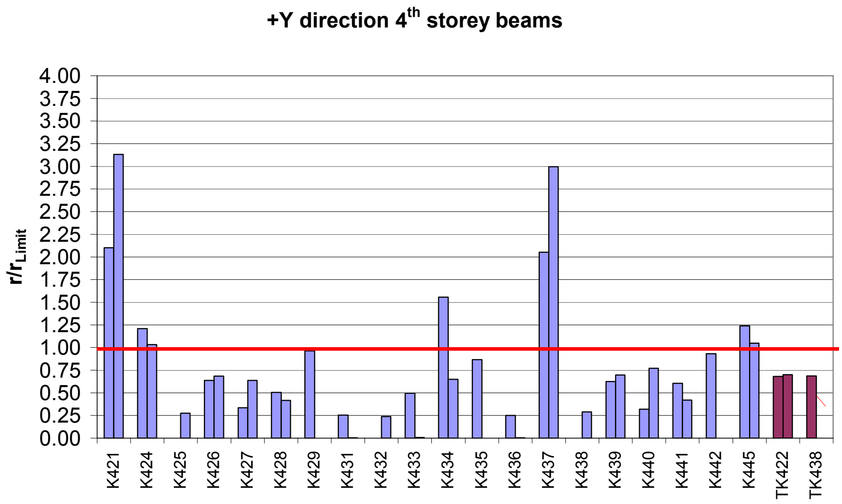
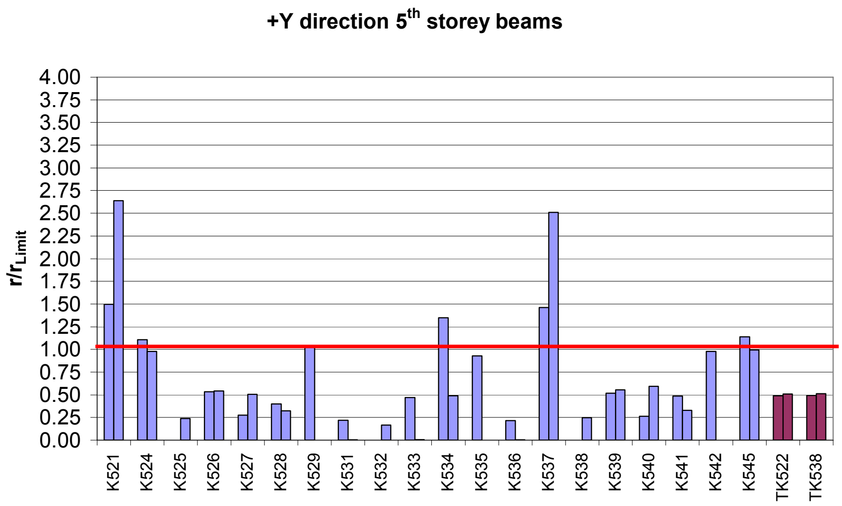


Figure 6.6 : r/r_{Limit} values (continued)





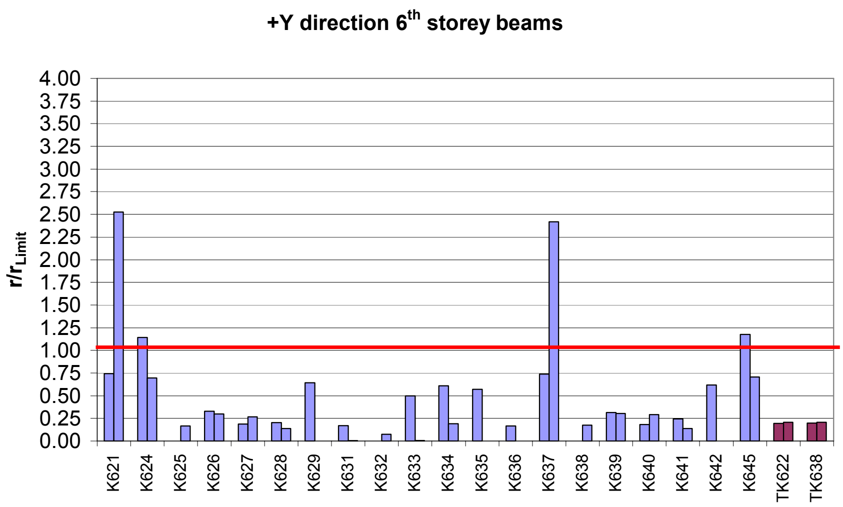


Figure 6.6 : r/r_{Limit} values (continued)

6.2.2 Global Performance of the Building

Global performance evaluation of the building for all stories and for two orthogonal directions are given in Table 6.4.

Table 6.4 : Global performance of the members

	+X direction		+Y direction	
St	**Columns and Walls (%)**	**Beams (%)**	**Columns and Walls (%)**	**Beams (%)**
1	12.1	**57.1**	0.0	22.7
2	0	5.3	2.0	22.7
3	0	5.9	2.4	27.3
4	0	0.0	2.9	22.7
5	0	0.0	2.2	22.7
6	0	0.0	4.7	18.2

Global performance evaluation of the building was found to be ***not*** satisfactory for Life Safety performance level because of the beams in X direction. However, in the +X direction if the beams that have r/r_{Limit} values less than 1.2 can be considered as acceptable, first storey beam ratio reduces to 28.6 and this makes the building acceptable according to Life Safety performance level. The results for interstorey drift ratios are given in Table 6.5.

Table 6.5 : Storey drifts

	+X direction			+Y direction		
St	**H_i (m)**	**$(\Delta_i)_{max}$**	**$(\Delta_i)_{max} / H_i$**	**H_i (m)**	**$(\Delta_i)_{max}$**	**$(\Delta_i)_{max} / H_i$**
1	2.7	0.008723	0.003231	2.7	0.008897	0.003295
2	2.8	0.008864	0.003166	2.8	0.013726	0.004902
3	2.8	0.009391	0.003354	2.8	0.015002	0.005358
4	2.8	0.009397	0.003356	2.8	0.014473	0.005169
5	2.8	0.008553	0.003055	2.8	0.012917	0.004613
6	2.8	0.008472	0.003026	2.8	0.012211	0.004361

6.2.3 Anchorage Design for Exterior Coupled Shear Wall With Linear Elastic Analysis Procedure

Anchorage design and detailing has a significant importance on the correct transfer of earthquake induced forces, and for new walls to work together with the existing system. Example calculations are given below for the coupled walls 1TP1 and 2TP1 adjacent to frame axis A'. In addition, anchorage reinforcement needed for all walls are given in Table 6.6.

Shear force demand for anchorage design of the wall was obtained from the difference of shear demands between the wall below and above the considered storey (Equation 6.1). Shear demands were flexural capacity related demands and calculated with Equation 6.2 given in the 2007 Turkish Earthquake Code where β_v is 1 for the assessment procedures.

$$V_a = V_{E\,i+1} - V_{E\,i} \tag{6.1}$$

$$V_E = \beta_v \frac{(M_K)_t}{(M_E)_t} V_E \tag{6.2}$$

Where;

$(M_K)_t$: Moment capacity at the bottom of the wall

$(M_E)_t$: Moment demand at the bottom of the wall from the earthquake loading with R=1.

V_E : Shear demand at the bottom of the wall from the earthquake loading with R=1

According to this formulation, shear demands of the walls 1TP1 and 2TP1 were calculated and given in tabular form in addition to the storey level shear demand for the wall which will be used in the design of anchorage bars. Maximum anchorage shear demand at the 1st storey was 199.1 – 184.6 = 14.5 kN.

Anchorage bar diameter needed to transmit this shear demand was calculated using Equations 6.3 and 6.4 where μ is 1 since it refers to the rough shear surface between new and existing concrete.

$$V_r \geq V_a \tag{6.3}$$

$$V_r = A_{wf} \times f_{yd} \times \mu \tag{6.4}$$

For the 1st storey level, TP1 wall;

$V_a = 14.5$ kN

$V_r = A_{wf} \times 365 \times 1$

A_{wf} was found to be 39.8 mm^2. According to the Section 7.10.5.1 of the 2007 Turkish Earthquake Code, minimum anchorage reinforcement is limited to φ16 / 400 mm. In order to be on the safe side, selection of φ18 diameter requires one bar for 1TP1 wall (39.8 / 254 = 1). However, this did not satisfy the minimum requirements so the anchorage requirement was φ18 / 400 mm which stands for three bars for a wall.

Same calculations were performed for other walls and for other stories and are given in Table 6.6. Moreover a schematic representation is shown in Figure 6.7 and Figure 6.8 which indicate anchorage bars designed and shear forces used in the anchorage design procedure with minimum bar spacing consideration. Here, only +X and +Y directions are represented but –X and –Y directions should also be calculated and maximum number of bars should be determined for a design project.

Table 6.6 : Anchorage bar design in the +X and +Y directions

WALLS IN X DIRECTION					WALLS IN Y DIRECTION				
Member	**Member Shear**	**Storey Shear**	**ϕ mm**	**Number of bars**	**Member**	**Member Shear**	**Storey Shear**	**ϕ mm**	**Number of bars**
1TP1	184.6	14.5	18	1	**1TP7**	301.4	13.5	18	1
2TP1	199.1	29.1	18	1	**2TP7**	314.9	3.4	18	1
3TP1	170.0	24.3	18	1	**3TP7**	318.3	48.7	18	1
4TP1	145.6	53.4	18	1	**4TP7**	269.6	70.7	18	1
5TP1	92.2	42.9	18	1	**5TP7**	198.9	112.3	18	2
6TP1	49.3	49.3	18	1	**6TP7**	86.5	86.5	18	1
1TP2	973.2	267.5	18	3	**1TP8**	1056.3	190.9	18	3
2TP2	1240.7	189.6	18	3	**2TP8**	865.3	13.9	18	1
3TP2	1051.1	198.0	18	3	**3TP8**	851.5	146.7	18	2
4TP2	853.1	205.8	18	3	**4TP8**	704.7	194.6	18	3
5TP2	647.3	363.1	18	4	**5TP8**	510.1	289.8	18	4
6TP2	284.2	284.2	18	4	**6TP8**	220.3	220.3	18	3
1TP3	792.6	103.1	18	2	**1TP9**	301.9	16.8	18	1
2TP3	895.7	67.3	18	1	**2TP9**	318.7	0.1	18	1
3TP3	828.4	96.1	18	2	**3TP9**	318.8	50.4	18	1
4TP3	732.3	191.5	18	3	**4TP9**	268.4	68.0	18	1
5TP3	540.8	283.8	18	4	**5TP9**	200.3	112.8	18	2
6TP3	257.0	257.0	18	3	**6TP9**	87.6	87.6	18	1
1TP4	181.3	6.2	18	1	**1TP10**	1057.9	188.8	18	3
2TP4	187.5	27.6	18	1	**2TP10**	869.1	28.8	18	1
3TP4	159.8	21.7	18	1	**3TP10**	840.3	142.9	18	2
4TO4	138.1	49.5	18	1	**4TP10**	697.4	182.5	18	2
5TP4	88.6	40.6	18	1	**5TP10**	514.9	294.0	18	4
6TP4	48.0	48.0	18	1	**6TP10**	220.8	220.8	18	3
1TP5	907.2	264.5	18	3					
2TP5	1171.7	58.6	18	1					
3TP5	1113.1	229.9	18	3					
4TP5	883.3	212.1	18	3					
5TP5	671.2	389.3	18	5					
6TP5	281.9	281.9	18	4					
1TP6	792.6	64.3	18	1					
2TP6	856.9	63.6	18	1					
3TP6	793.4	85.9	18	1					
4TP6	707.5	178.9	18	2					
5TP6	528.6	274.9	18	3					
6TP6	253.7	253.7	18	3					

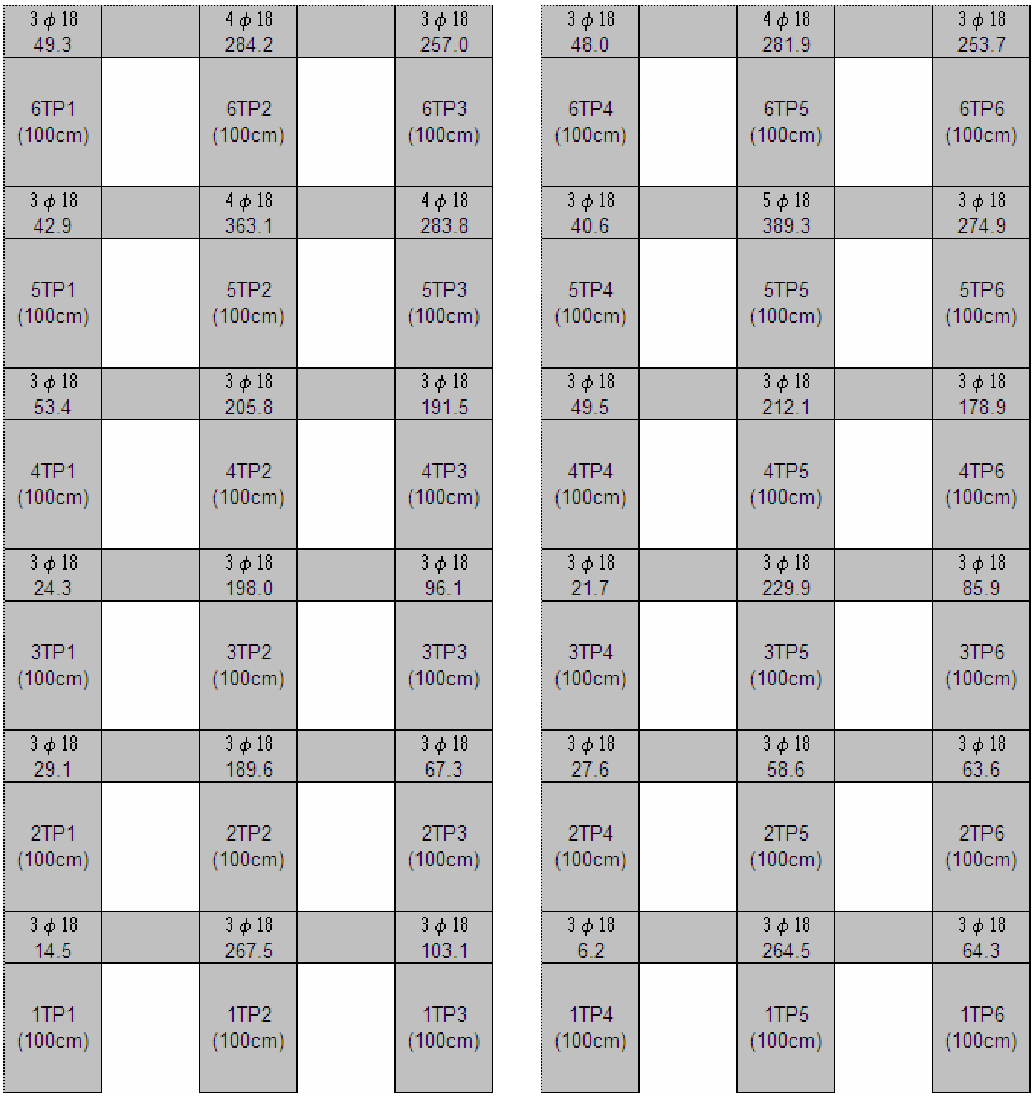


Figure 6.7 : Anchorage reinforcement representation for walls in the +X direction

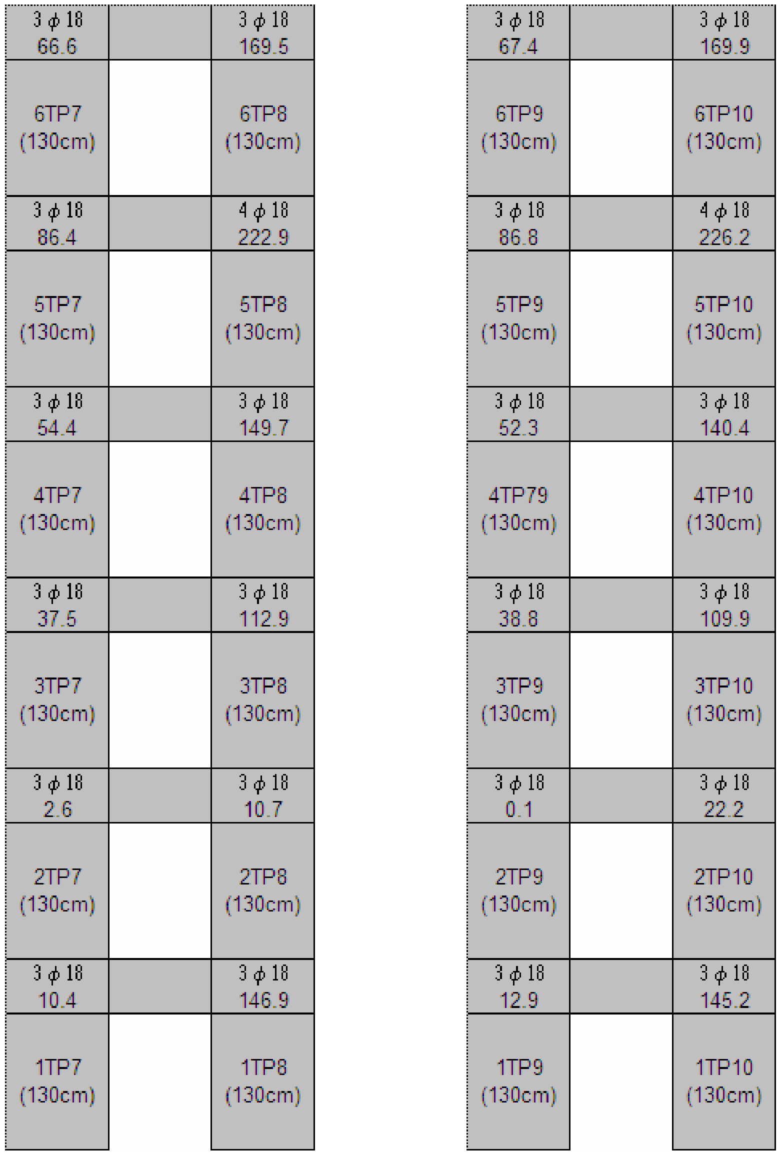


Figure 6.8 : Anchorage reinforcement representation for walls in the +Y direction

6.3 Nonlinear Static Procedure

The retrofitted building was also assessed by the non-linear procedure. The procedure was explained in the previous case study in detail. Therefore, for this case study, only the results will be summarized.

6.3.1 Target Displacement in the +X and +Y Directions

Target displacements are calculated as explained previously and found as 0.068m in the +X direction and 0.092m in the +Y direction respectively (.

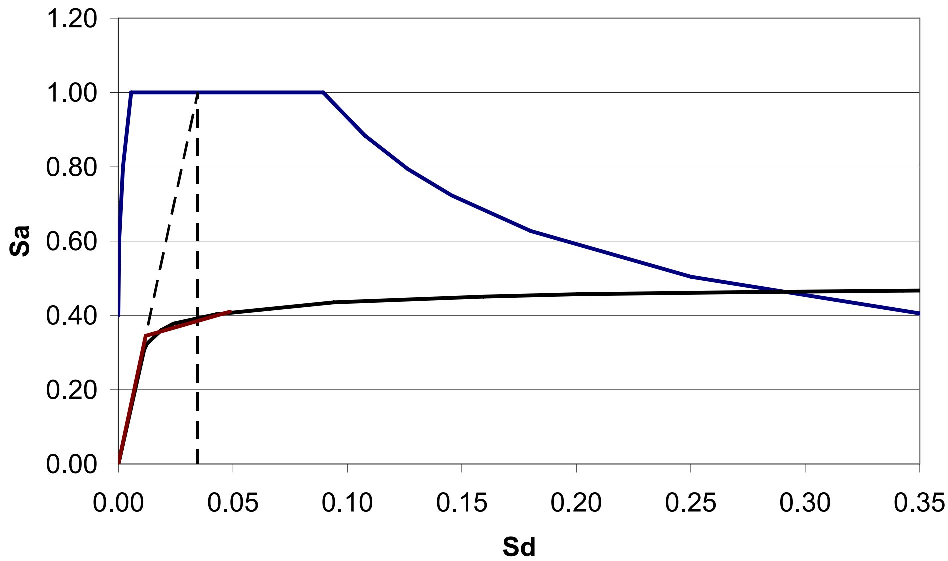


Figure 6.9 : Target displacement for +X direction

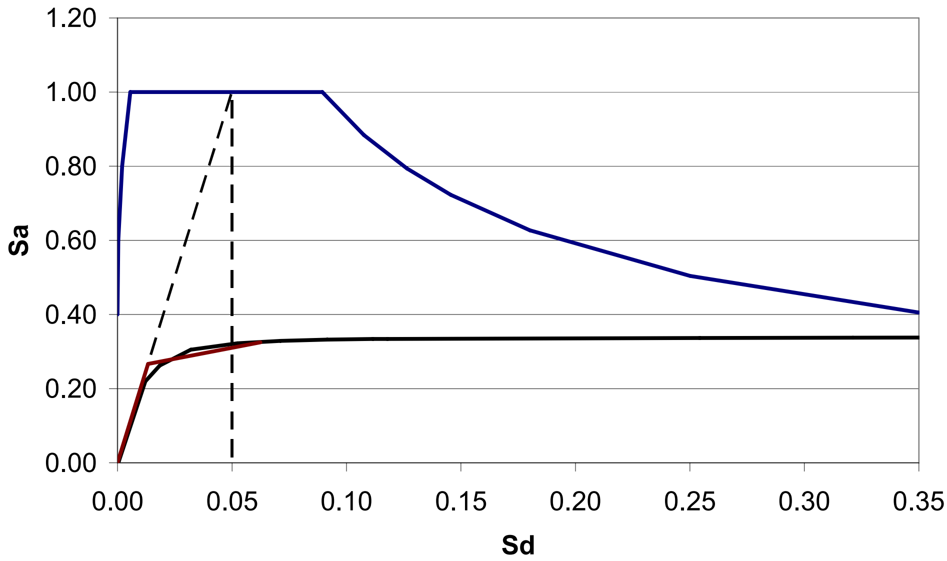


Figure 6.10 : Target displacement for +Y direction

6.3.2 Comparison of Strains (ε) with Strain Limits (ε_{Limit})

The ratio of ε to ε_{Limit} values are calculated for all member ends and presented in bar chart form for beams and columns separately in each storey.

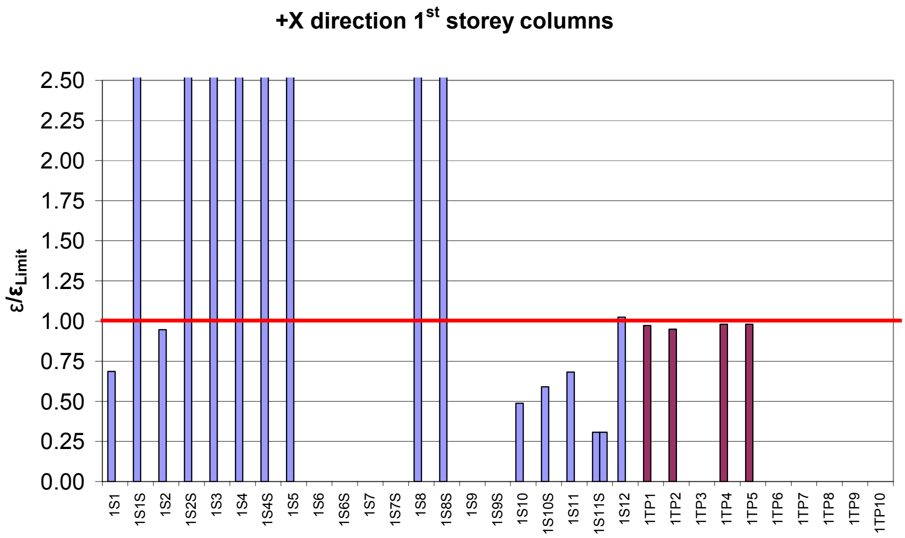


Figure 6.11 : ε / ε $_{\text{Limit}}$ values

All ε / ε $_{\text{Limit}}$ values of the columns at stories 2, 3, 4, 5, and 6 are smaller than 1, so they are not shown here.

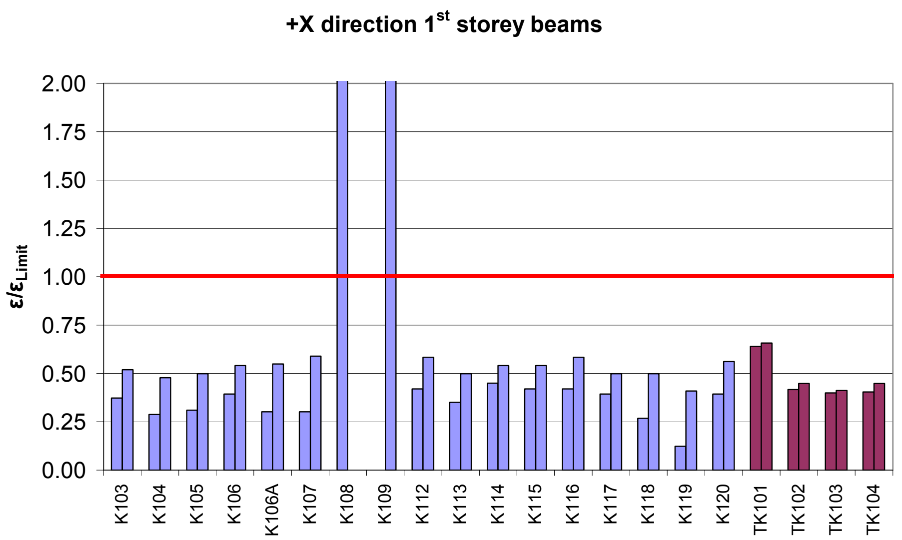
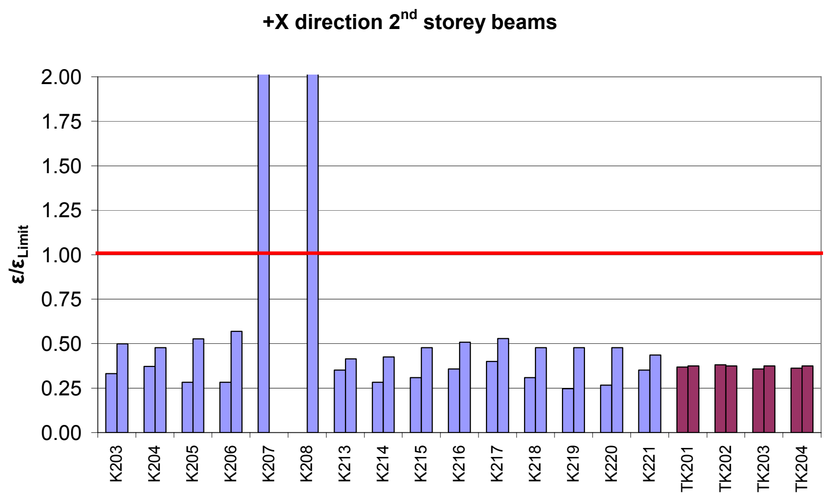
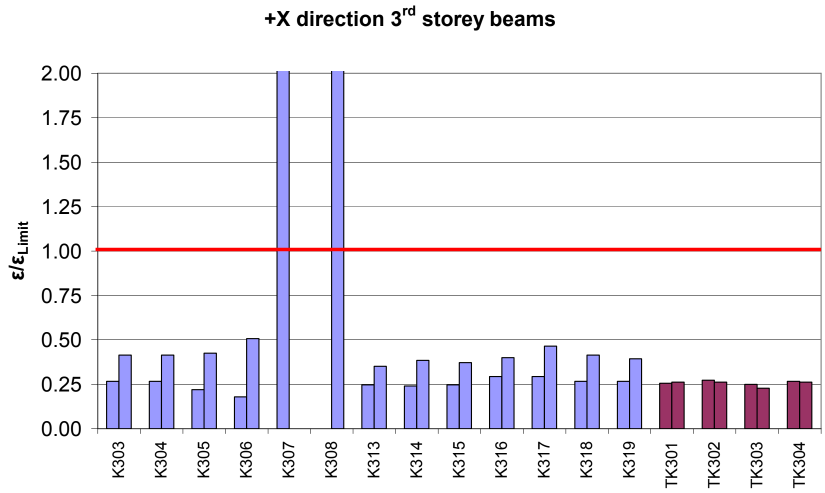


Figure 6.12 : ε / ε $_{\text{Limit}}$ values





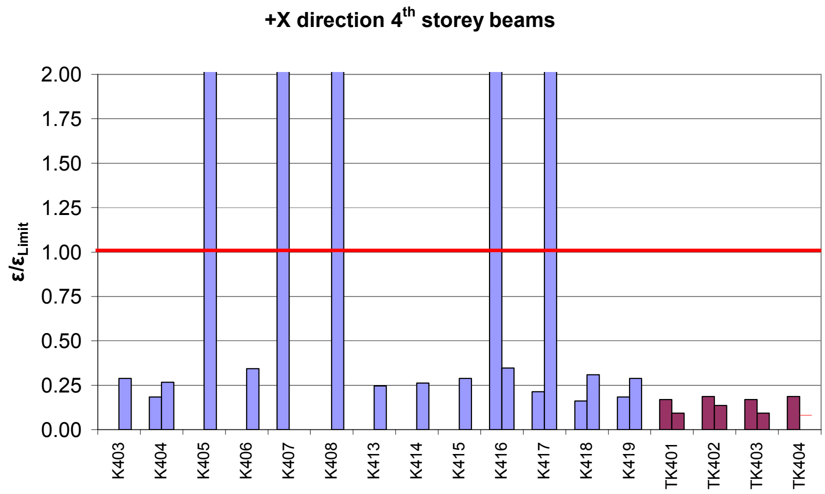
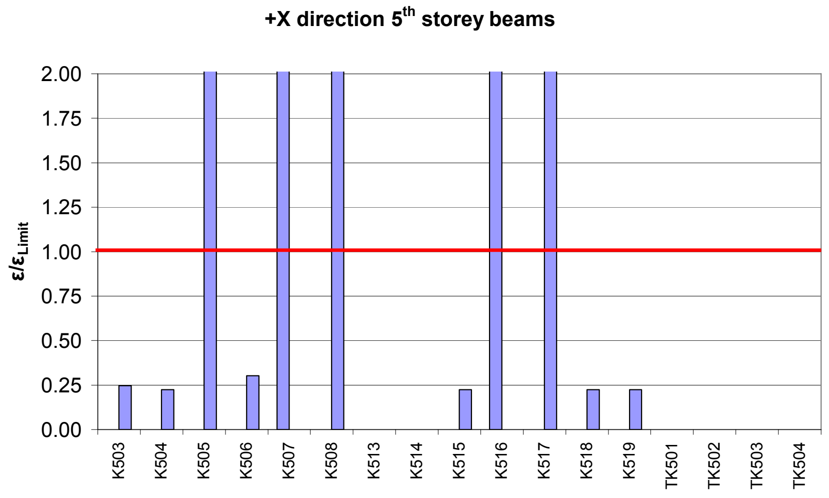
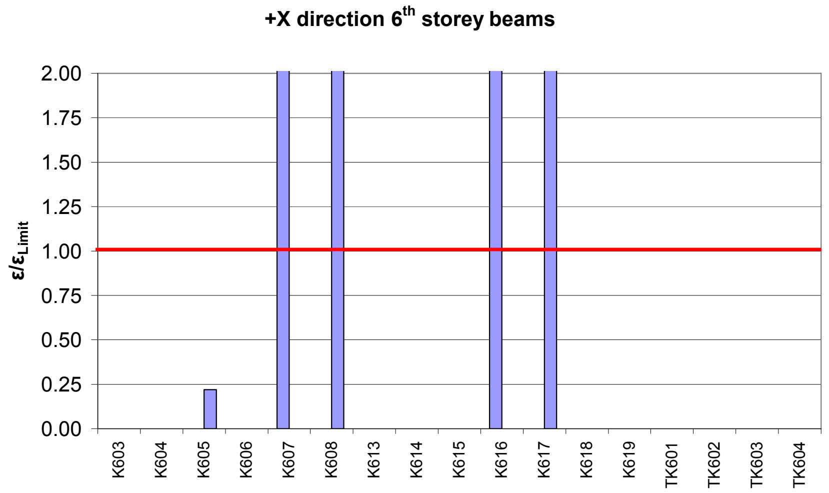


Figure 6.12 : $\varepsilon / \varepsilon_{Limit}$ values (continued)





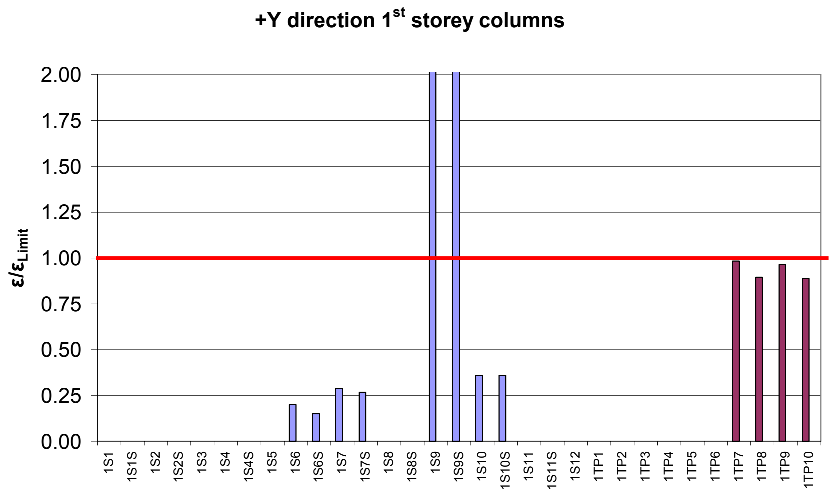


Figure 6.12 : ε / $\varepsilon_{\text{Limit}}$ values (continued)

All $\varepsilon / \varepsilon_{Limit}$ values of the columns at stories 2, 3, 4, 5, and 6 are smaller than 1, so they are not shown here.

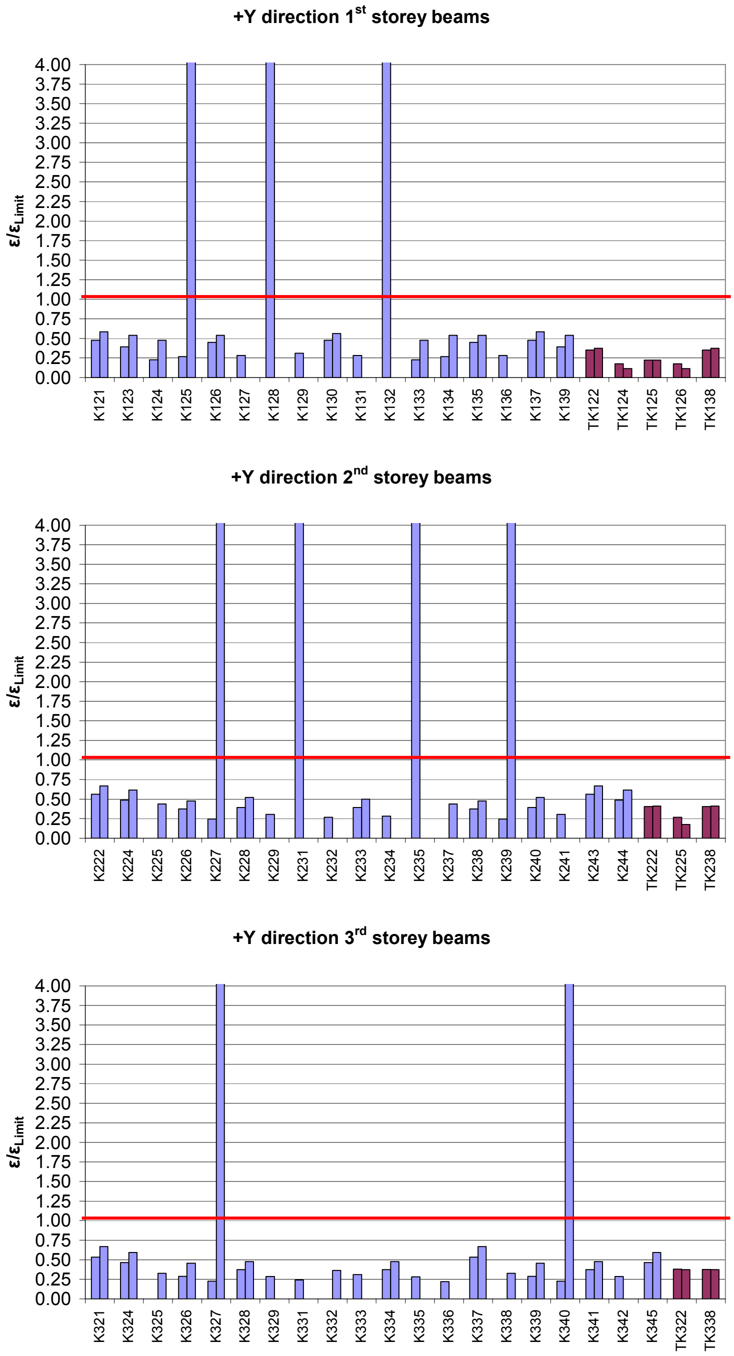
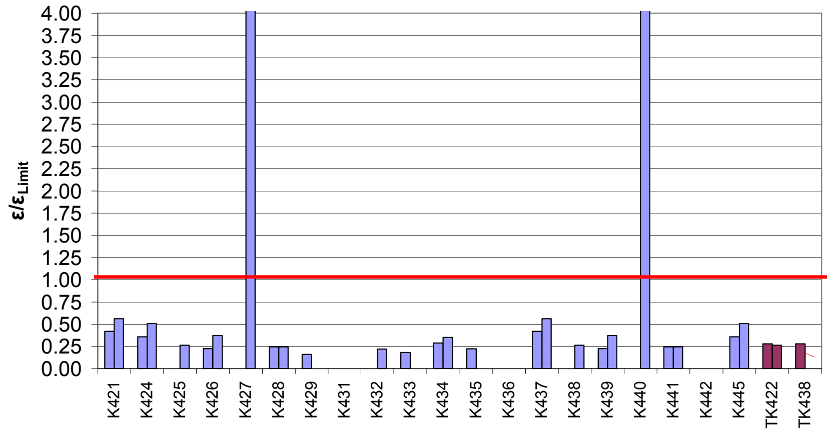
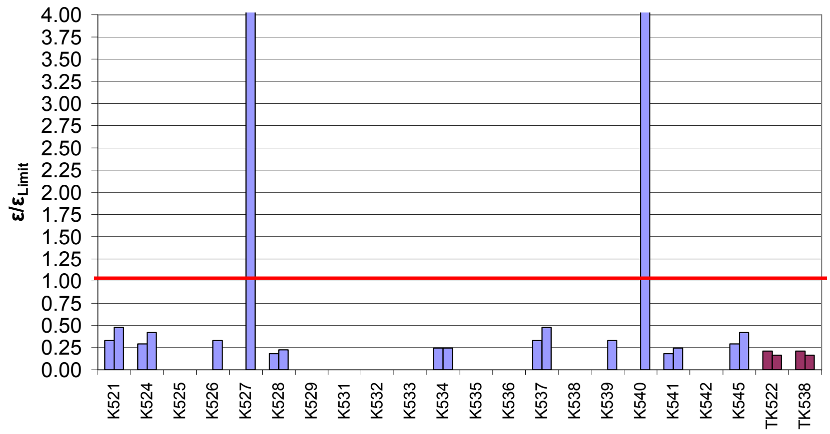


Figure 6.13 : $\varepsilon / \varepsilon_{Limit}$ values

+Y direction 4th storey beams



+Y direction 5th storey beams



+Y direction 6th storey beams

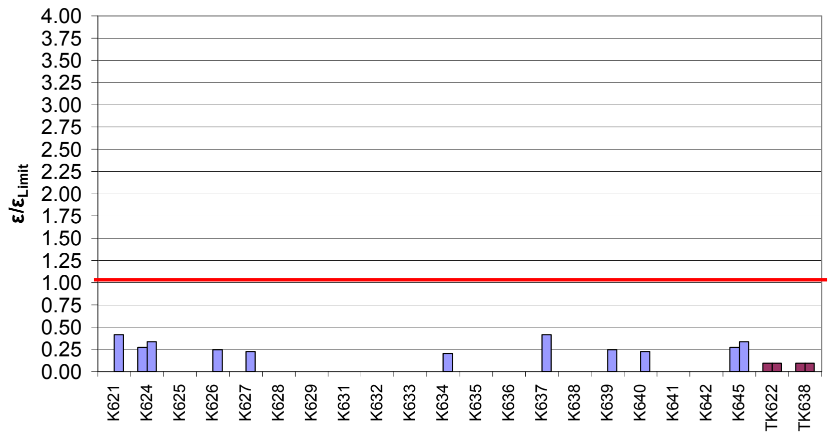


Figure 6.13 : ε / ε $_{\text{Limit}}$ values (continued)

6.3.3 Global Performance of the Building

Global performance evaluation of the building for all stories and for two orthogonal directions are given in Table 6.7.

Table 6.7 : Global performance of the members

	+X direction		+Y direction	
St	**Columns and Walls (%)**	**Beams (%)**	**Columns and Walls (%)**	**Beams (%)**
1	10.5	9.5	2.4	13.6
2	0.0	10.5	0.0	18.2
3	0.0	11.8	0.0	9.1
4	0.0	29.4	0.0	9.1
5	0.0	29.4	0.0	9.1
6	0.0	23.5	0.0	0.0

Since the case study was a residential building, the target performance level was selected as "Life Safety" and the calculated interstorey drifts should be less than 0.03. The results are given in Table 6.8 below.

Table 6.8 : Storey drifts

	+X direction			+Y direction		
St	**H_i (m)**	**$(\Delta_i)_{max}$**	**$(\Delta_i)_{max} / H_i$**	**H_i (m)**	**$(\Delta_i)_{max}$**	**$(\Delta_i)_{max} / H_i$**
1	2.7	0.005207	0.001929	2.7	0.009274	0.003435
2	2.8	0.005868	0.002096	2.8	0.013873	0.004954
3	2.8	0.006531	0.002333	2.8	0.01444	0.005157
4	2.8	0.006722	0.002401	2.8	0.01172	0.004186
5	2.8	0.006321	0.002257	2.8	0.007923	0.002830
6	2.8	0.006026	0.002152	2.8	0.005939	0.002121

Retrofit solution increased the strength and stiffness considerably in both directions, which is sufficient for satisfying the life safety performance level.

6.3.4 Anchorage Design of Exterior Coupled Shear Wall With Nonlinear Analysis Procedure

Anchorage design and detailing have similar procedure with linear elastic analysis.. Example calculations are given below for the coupled walls 1TP1 and 2TP1 adjacent to frame axis A'. In addition, anchorage reinforcement needed for all walls are given in Table 6.9.

The only difference between the linear and nonlinear procedures is that the calculation of shear demand. It is directly obtained from the pushover analysis at target displacement.

Shear demands of the walls 1TP1 and 2TP1 were calculated and given in tabular form in addition to the storey level shear demand for the wall which will be used in the design of anchorage bars. Maximum shear demand of the 1st storey level was 373.4 – 219.3 = 154.1 kN.

Anchorage bar diameter needed to transmit this shear demand is calculated using the Equations 6.5 and 6.6 where μ is 1 since it refers to the rough shear surface between new and existing concrete.

$$V_r \geq V_a \quad (6.5)$$

$$V_r = A_{wf} \times f_{yd} \times \mu \quad (6.6)$$

For the 1st storey level, TP1 wall;

$V_a = 154.1$ kN

$V_r = A_{wf} \times 365 \times 1$

A_{wf} was found to be 422.2 mm^2. According to the Section 7.10.5.1 of the 2007 Turkish Earthquake Code, minimum anchorage reinforcement is limited to

$\varphi 16$ / 400 mm. In order to be on the safe side, selection of $\varphi 18$ diameter requires two bars for 1TP1 wall (422.2 / 254 = 2). The anchorage requirement was $\varphi 18$ / 400 mm which stands for three bars for a wall.

Same calculations were performed for other walls and for other stories and are given in Table 6.9. Moreover a schematic representation was shown in Figure 6.14 and Figure 6.15 which include anchorage bars designed and shear forces used in the anchorage design procedure with minimum bar spacing consideration. Here, only +X and +Y directions are represented but –X and –Y directions should also be calculated and maximum number of bars should be determined for a design project.

Table 6.9 : Anchorage bar design in the +X and +Y directions

WALLS IN X DIRECTION				
Member	**Member Shear**	**Storey Shear**	**ϕ mm**	**Number of bars**
1TP1	219.3	154.1	18	2
2TP1	373.4	61.2	18	1
3TP1	312.2	7.1	18	1
4TP1	305.1	178.7	18	2
5TP1	126.4	95.2	18	2
6TP1	31.1	31.1	18	1
1TP2	608.8	81.8	18	1
2TP2	690.6	92.2	18	1
3TP2	598.4	66.5	18	1
4TP2	532.0	112.7	18	2
5TP2	419.3	278.8	18	4
6TP2	140.4	140.4	18	2
1TP3	524.4	188.1	18	3
2TP3	336.3	0.7	18	1
3TP3	337.0	104.7	18	2
4TP3	232.4	16.0	18	1
5TP3	216.4	75.6	18	1
6TP3	140.8	140.8	18	2
1TP4	166.1	202.2	18	3
2TP4	368.3	54.6	18	1
3TP4	313.7	12.5	18	1
4TO4	301.2	174.2	18	2
5TP4	127.0	98.9	18	2
6TP4	28.1	28.1	18	1
1TP5	595.7	65.7	18	1
2TP5	661.3	11.2	18	1
3TP5	650.2	111.7	18	2
4TP5	538.5	136.6	18	2
5TP5	401.9	274.3	18	3
6TP5	127.6	127.6	18	2
1TP6	532.5	188.8	18	3
2TP6	343.8	13.8	18	1
3TP6	330.0	96.2	18	2
4TP6	233.8	20.3	18	1
5TP6	213.5	79.0	18	1
6TP6	134.5	134.5	18	2

WALLS IN Y DIRECTION				
Member	**Member Shear**	**Storey Shear**	**ϕ mm**	**Number of bars**
1TP7	266.5	131.2	18	2
2TP7	397.7	61.6	18	1
3TP7	459.3	88.1	18	1
4TP7	371.3	92.8	18	1
5TP7	278.5	186.1	18	3
6TP7	92.3	92.3	18	1
1TP8	632.1	188.7	18	3
2TP8	443.4	17.4	18	1
3TP8	460.8	93.8	18	2
4TP8	367.1	120.5	18	2
5TP8	246.5	164.9	18	2
6TP8	81.6	81.6	18	1
1TP9	269.1	129.7	18	2
2TP9	398.8	63.6	18	1
3TP9	462.4	90.7	18	1
4TP9	371.8	93.1	18	2
5TP9	278.7	185.7	18	2
6TP9	93.0	93.0	18	2
1TP10	631.3	185.4	18	2
2TP10	446.0	7.5	18	1
3TP10	453.5	90.4	18	1
4TP10	363.1	113.8	18	2
5TP10	249.2	167.2	18	2
6TP10	82.0	82.0	18	1

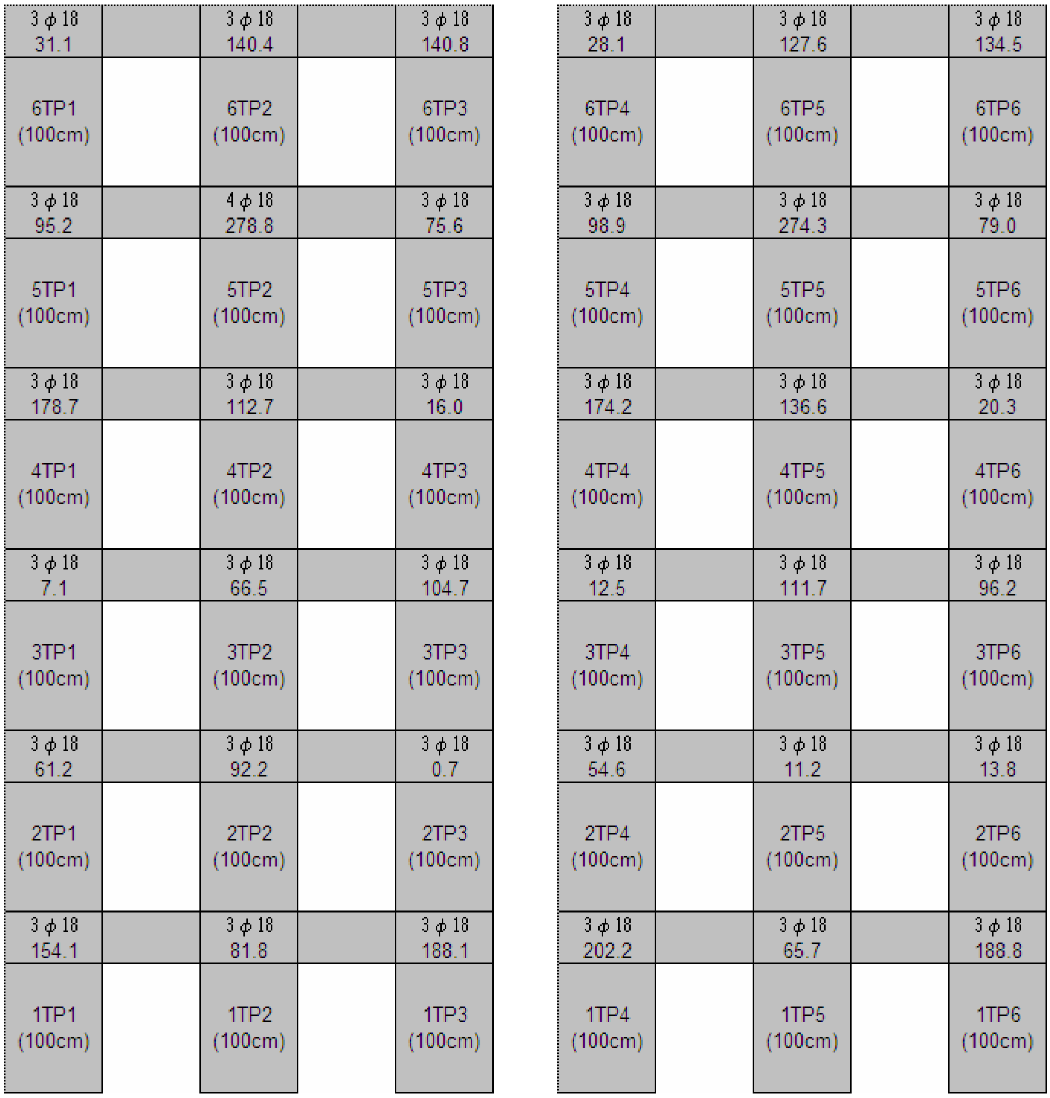


Figure 6.14 : Anchorage reinforcement representation for walls in the +X direction

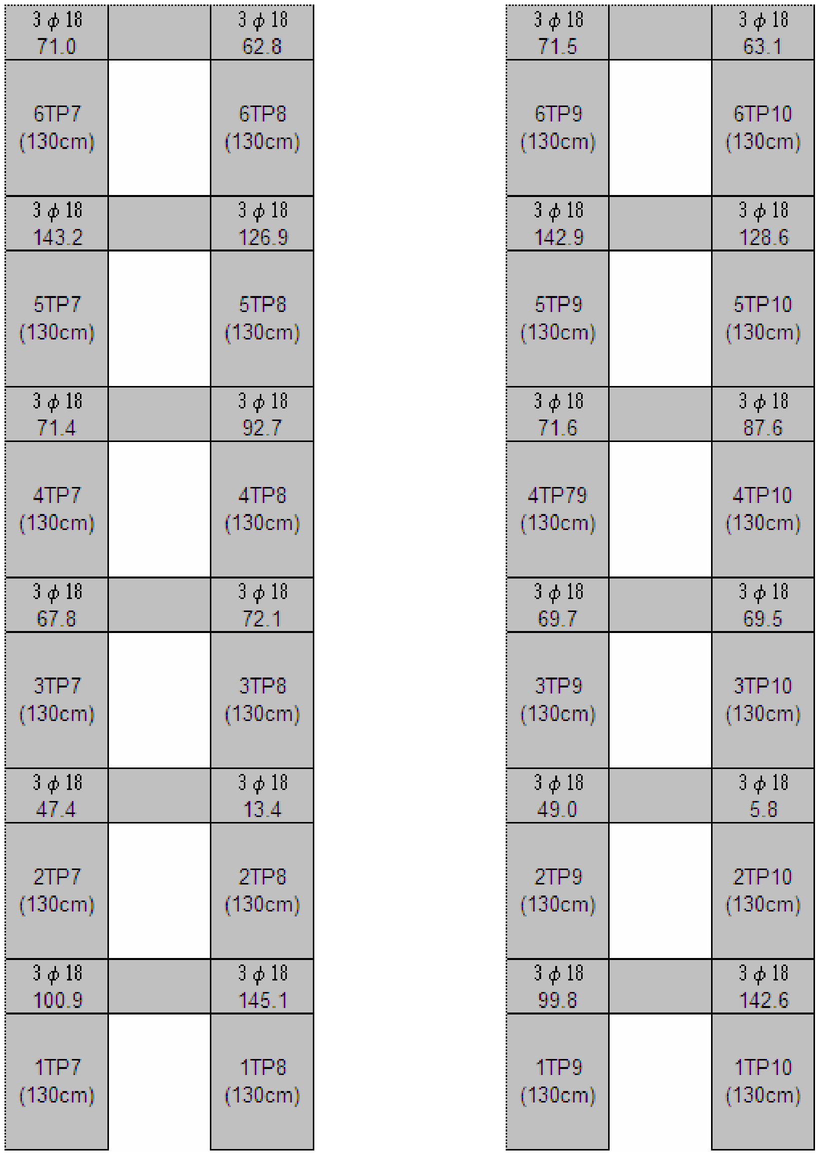


Figure 6.15 : Anchorage reinforcement representation for walls in the +Y direction

CHAPTER VII

DISCUSSION OF RESULTS AND CONCLUSION

7.1 Summary

A detailed study has been undertaken to evaluate the performance of assessment procedures described in the 2007 Turkish Earthquake Code. In order to achieve this goal, two pairs of existing residential buildings, both before and after retrofitting, were evaluated with the linear elastic and nonlinear static procedures of the 2007 Turkish Earthquake Code.

Firstly, a five-storey existing residential building located in Düzce, Turkey was assessed by using both linear elastic and nonlinear procedures. Performance of the retrofitted building was also evaluated by using the same procedures. Damage state at the first storey members during the 1999 Düzce Earthquake of the building was available, which permitted a comparative evaluation of the two performance assessment procedures with reference to the actual damage observations.

Next, a six-storey existing residential building located in Bakırköy, Turkey and its retrofitted state were evaluated with the linear elastic and nonlinear procedures of the 2007 Turkish Earthquake Code. Retrofit solution consists of exterior coupled shear walls added to the system for which an anchorage design methodology was also developed.

7.2 Discussion of Results

7.2.1 Comparative Assessment of the Existing Building in Düzce with Reference to Observed Earthquake Damage

Evaluation of the assessment procedures given in the 2007 Turkish Earthquake Code for prediction of damage state of the members was achieved by comparing the damage predicted by the Code procedures with actual damage observations. Results are given for only the first storey since damage observations were only available for the first storey of the building.

A bar chart representation was used for the illustration of the damage levels for all members. Three bars for a member indicate the observed damage after the 1999 Düzce earthquake, and predicted damages by the two procedures, namely linear elastic and nonlinear procedures.

Observing the results shown in Figure 7.1, it can be noticed that both procedures have a tendency to overestimate the observed damage levels. Considering shear walls only, nonlinear procedure gives closer results than linear elastic procedure with respect to the actual damage. Nonlinear procedure predicted significant damage for the columns 1S02 and 1S06 which have also exhibited significant damage during the earthquake. On the other hand, linear procedure damage predictions for columns 1S02 and 1S06 were collapse. Although there was no observed damage for the other columns, both assessment procedures predicted damage varying from minimum to collapse damage levels for these members. Considering columns only, it can be concluded that both procedures overestimate the observed damage.

The same situation was also valid in the assessment of beams. Although both procedures estimated damage for all beams, only some of them were actually damaged.

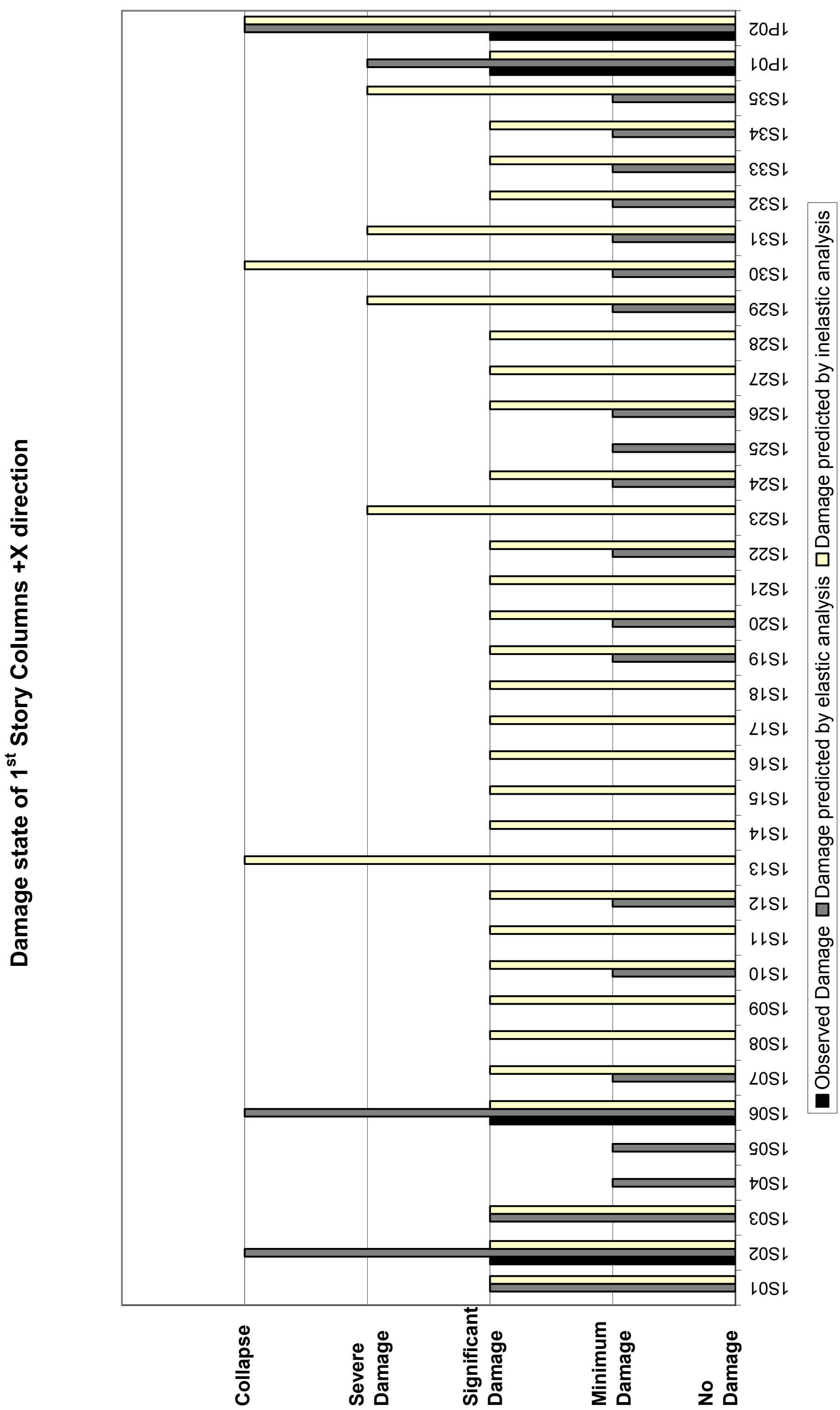


Figure 7.1 : Damage level comparison for case study 1

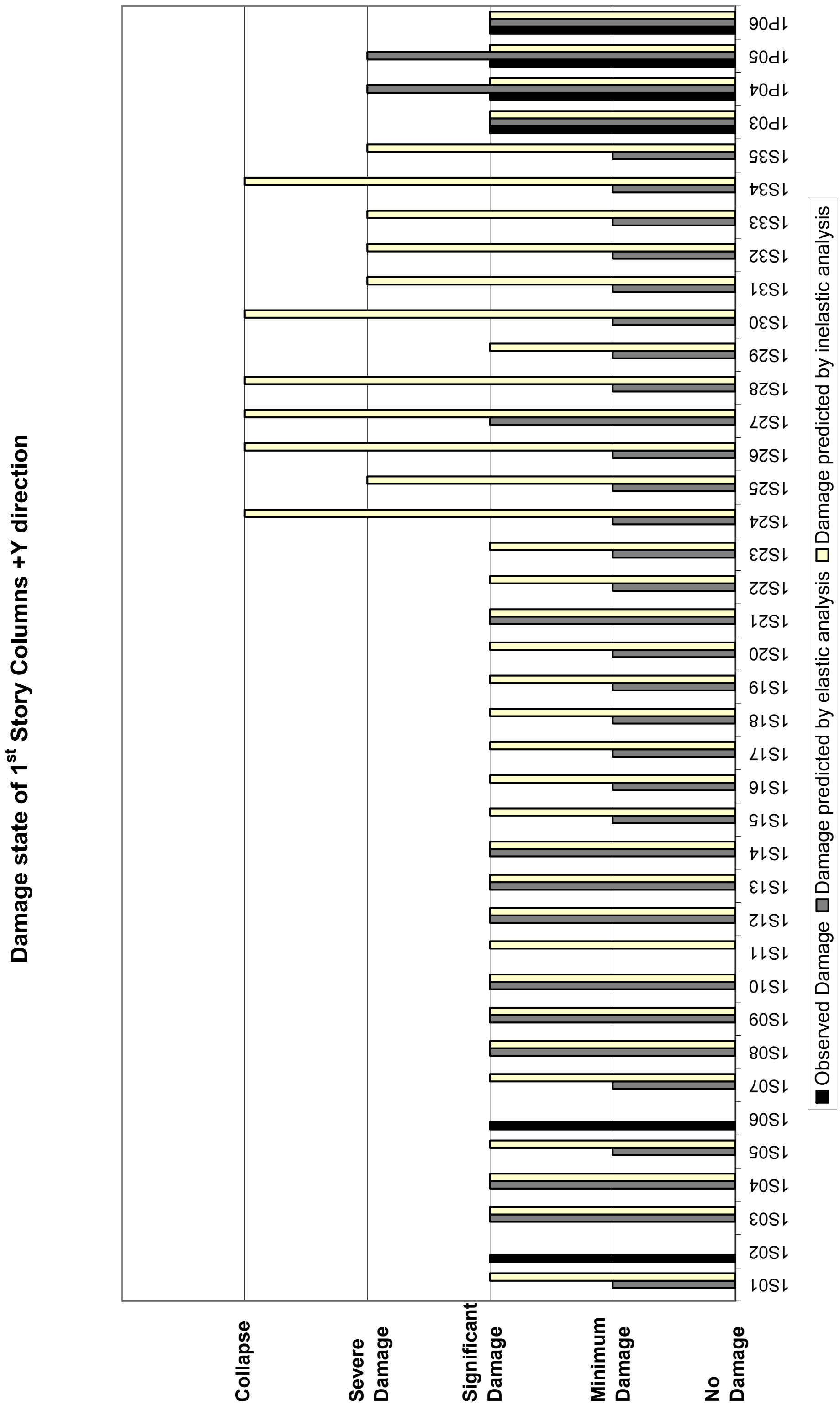


Figure 7.1 : Damage level comparison for case study 1 (continued)

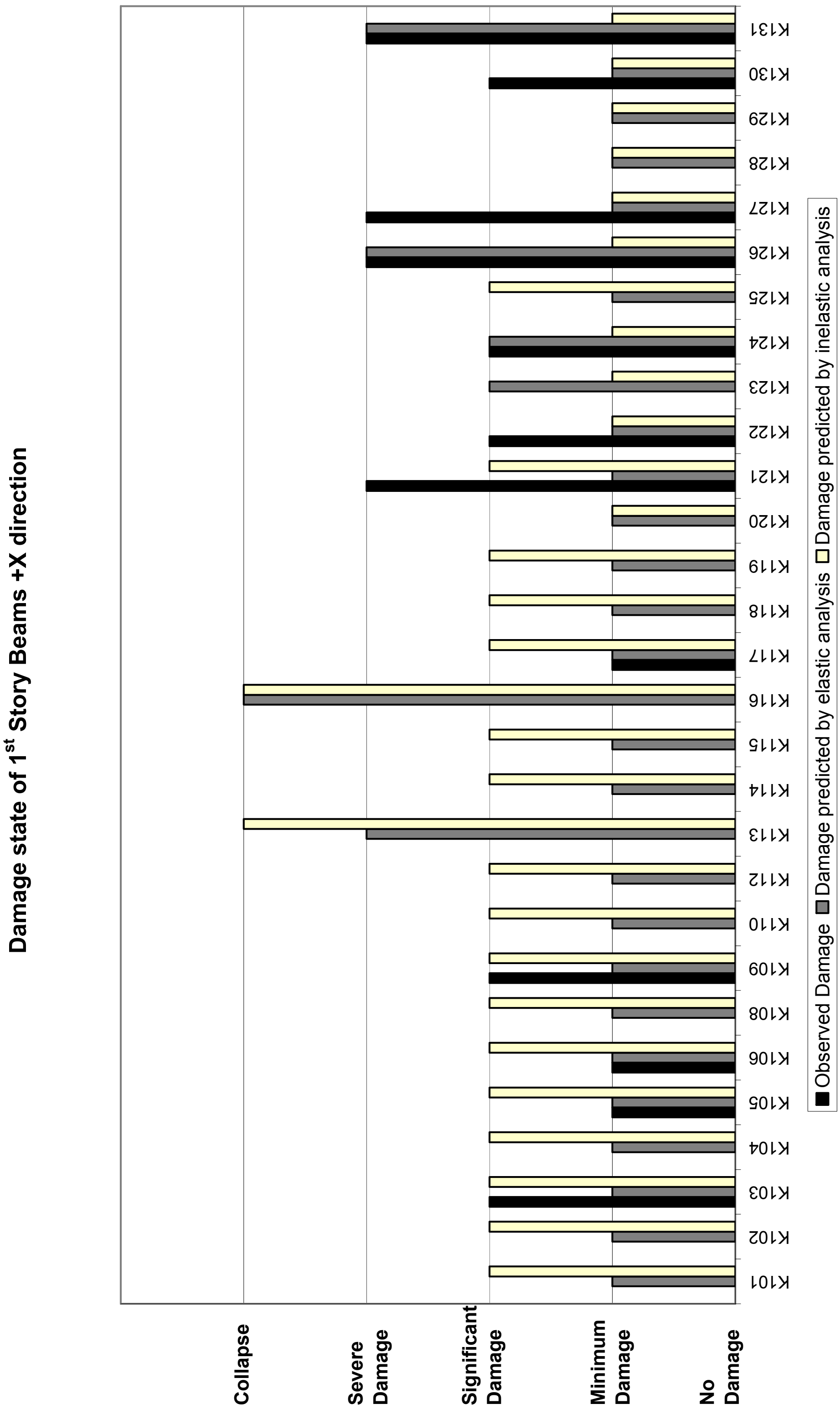


Figure 7.1 : Damage level comparison for case study 1 (continued)

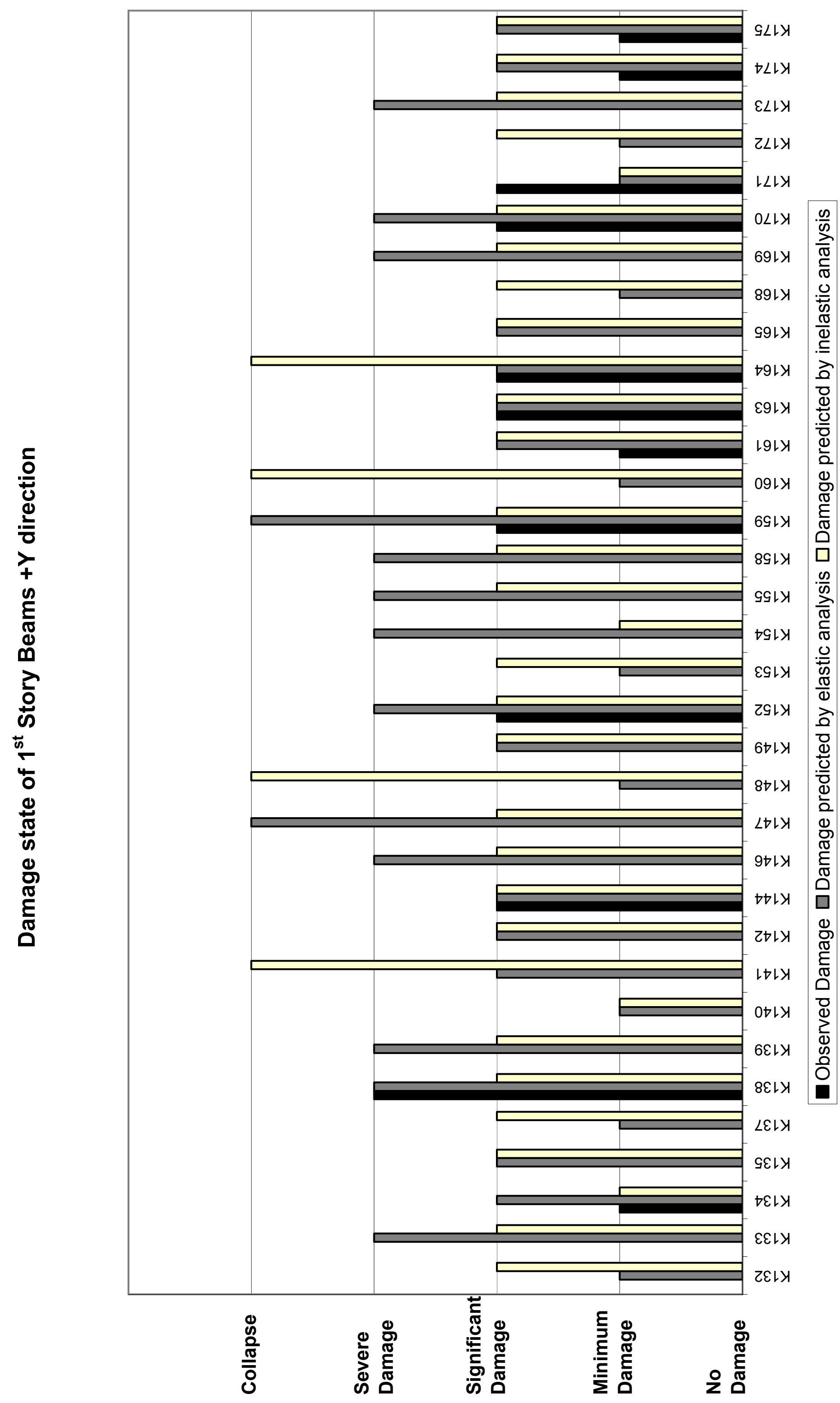


Figure 7.1 : Damage level comparison for case study 1 (continued)

The results are also presented in tabular form in Table 7.1 and Table 7.2 for the comparison of global performance estimation by the two procedures for the first storey.

Table 7.1 : Comparative global performance estimation for the first storey columns for case study 1

DAMAGE OBSERVATION : COLUMNS

1st Storey	Number of Columns +X	Number of Walls +X	% Shear Columns & Walls +X	Number of Columns +Y	Number of Walls +Y	% Shear Columns & Walls +Y
No Damage	33	0	17.6	33	0	0.4
Minimum Damage	0	0	0	0	0	0
Significant Damage	**2**	**2**	**82.4**	**2**	**4**	**99.6**
Severe Damage	0	0	0	0	0	0
Collapse	0	0	0	0	0	0
Total	35	2	100	35	4	100
Performance Level	**Life Safety**			**Life Safety**		

ELASTIC ANALYSIS : COLUMNS

1st Storey	Number of Columns +X	Number of Walls +X	% Shear Columns & Walls +X	Number of Columns +Y	Number of Walls +Y	% Shear Columns & Walls +Y
No Damage	13	0	4.9	3	0	0.7
Minimum Damage	18	0	11.0	22	0	23.5
Significant Damage	2	0	1.6	10	2	43.1
Severe Damage	0	**1**	**40.9**	0	**2**	**32.7**
Collapse	2	**1**	**41.5**	0	0	0.0
Total	35	2	100	35	4	100
Performance Level	**Collapse**			**Collapse Prevention**		

INELASTIC ANALYSIS : COLUMNS

1st Storey	Number of Columns +X	Number of Walls +X	% Shear Columns & Walls +X	Number of Columns +Y	Number of Walls +Y	% Shear Columns & Walls +Y
No Damage	3	0	5.3	2	0	5.8
Minimum Damage	0	0	0.0	0	0	0.0
Significant Damage	26	1	71.4	22	4	64.2
Severe Damage	**4**	**0**	**7.1**	**5**	**0**	**12.7**
Collapse	**2**	**1**	**16.2**	**6**	**0**	**17.3**
Total	35	2	100	35	4	100
Performance Level	**Collapse**			**Collapse**		

Table 7.2 : Comparative global performance estimation for the first storey beams for case study 1

DAMAGE OBSERVATION : BEAMS

1st Storey	Number of Beams +X	% Beams +X	Number of Beams +Y	% Beams +Y
No Damage	17	58.6	22	64.7
Minimum Damage	3	10.3	4	11.8
Significant Damage	5	17.2	7	20.6
Severe Damage	4	13.8	1	2.9
Collapse	0	0.0	0	0.0
Total	29	100	34	100
Performance Level	**Life Safety**		**Life Safety**	

ELASTIC ANALYSIS : BEAMS

1st Storey	Number of Beams +X	% Beams +X	Number of Beams +Y	% Beams +Y
No Damage	0	0.0	0	0.0
Minimum Damage	23	79.3	9	35.3
Significant Damage	2	10.3	12	35.3
Severe Damage	3	**6.9**	11	**23.5**
Collapse	**1**	**3.4**	**2**	**5.9**
Total	29	100	34	100
Performance Level	**Collapse Prevention**		**Collapse Prevention**	

INELASTIC ANALYSIS : BEAMS

1st Storey	Number of Beams +X	% Beams +X	Number of Beams +Y	% Beams +Y
No Damage	0	0.0	0	0.0
Minimum Damage	10	33.5	4	11.8
Significant Damage	17	58.6	26	76.5
Severe Damage	**2**	**6.9**	0	0.0
Collapse	0	0.0	**4**	**11.8**
Total	29	100	34	100
Performance Level	**Life Safety**		**Collapse Prevention**	

Linear elastic assessment procedure has estimated the building performance level as “Collapse” for the first storey. This also shows that there is no need for the evaluation of upper floors since “collapse” is dominant in the global performance evaluation of the building. Similar behavior was observed with the nonlinear procedure. In contrast, building was standing despite two destructive earthquakes and performance estimation was “Life Safety” considering the field damage observations.

7.2.2 Comparative Assessment of the Existing Substandard Buildings

Global performance comparison for the two existing buildings are given in Table 7.3 and Table 7.4 where “L” represent linear elastic procedure and “NL” represents nonlinear procedure. Values are the shear percentages taken by the unacceptable vertical members and percentages based on number of unacceptable beams. For both case studies, both buildings are substandard and resultant performance levels are not satisfactory for life safety, however linear procedure predicts lower performance than the nonlinear procedure. Despite the ultimate weakness of case study 3 building, nonlinear procedure still predicts less damage than that would be expected during a code level earthquake.

Table 7.3 : Global performance comparison for case study 1 building (Düzce)

Storey	+X Direction				+Y Direction			
	Columns and Walls		Beams		Columns and Walls		Beams	
	L	NL	L	NL	L	NL	L	NL
1	82.4	23.3	10.3	6.9	32.7	30.0	29.4	11.8
2	41.0	67.9	25.0	10.0	0.0	0.0	41.2	11.8
3	1.5	0.0	25.0	5.0	0.0	0.0	47.1	11.8
4	1.7	0.0	20.0	5.0	0.0	0.0	8.8	11.8
5	8.3	0.0	10.0	0.0	11.1	0.0	5.9	5.9

Table 7.4 : Global performance comparison for case study 3 building (Bakırköy)

Storey	+X Direction				+Y Direction			
	Columns		Beams		Columns		Beams	
	L	NL	L	NL	L	NL	L	NL
1	94.7	37.9	94.1	17.6	79.6	29.0	87.5	6.3
2	88.7	14.9	100.0	53.3	85.6	0.0	87.5	50.0
3	97.7	0.0	100.0	61.5	85.6	0.0	100.0	75.0
4	96.6	33.5	100.0	61.5	93.0	0.0	100.0	43.8
5	90.4	41.1	84.6	23.1	92.4	43.8	87.5	6.3
6	49.2	0.0	15.4	0.0	57.0	0.0	31.3	0.0

7.2.3 Comparative Assessment of the Retrofitted Buildings

The same comparison was also made for the retrofit solutions of the case study buildings. Both of the procedures predicted an increase in performance of vertical members with the introduction of new members. Added shear walls improved lateral strength of the buildings and reduced the demands on the existing vertical members.

Existing beams are incapable of carrying earthquake loads leading to higher percentage of unacceptable beams. Although there is no retrofitting scheme offered for beams, retrofitting decreased the demand on beams but some of them are still unacceptable due to insufficient dimensions and material properties. Moreover, beams joining to the new walls fall to the collapse region due to high demands which make them unacceptable too. As a result, performance of beams do not increase much, after retrofitting.

Global performance of the buildings have increased with the recommended retrofit solutions but yet linear procedure predicts that building is not satisfactory for life safety performance level because of the poor performance of beams. Increasing the acceptability limits of beams may result in a higher performance as predicted by the nonlinear procedure. Results revealed that linear elastic procedure may underestimate the performance of members, especially those of beams. Life Safety performance level was achieved when only column performances considered, but building performance level was estimated to be Collapse Prevention due to high elastic demand of beams in the linear procedure. On the contrary, nonlinear procedure predicted Life Safety performance level for both retrofitted buildings (Table 7.5 and Table 7.6).

Table 7.5 : Global performance comparison for case study 2 building (Düzce)

Storey	+X Direction				+Y Direction			
	Columns and Walls		Beams		Columns and Walls		Beams	
	L	NL	L	NL	L	NL	L	NL
1	0.5	0.0	7.4	3.7	0.0	0.0	24.2	6.1
2	0.0	0.0	11.1	11.1	0.0	0.0	42.4	12.1
3	0.0	0.0	11.1	11.1	0.0	0.0	39.4	9.1
4	0.0	0.0	11.1	11.1	0.0	0.0	21.2	6.1
5	1.7	0.0	11.1	0.0	14.7	0.0	15.2	6.1

Table 7.6 : Global performance comparison for case study 4 building (Bakırköy)

Storey	+X Direction				+Y Direction			
	Columns and Walls		Beams		Columns and Walls		Beams	
	L	NL	L	NL	L	NL	L	NL
1	12.1	10.5	57.1	9.5	0.0	2.4	22.7	13.6
2	0.0	0.0	5.3	10.5	2.0	0.0	22.7	18.2
3	0.0	0.0	5.9	11.8	2.4	0.0	27.3	9.1
4	0.0	0.0	0.0	29.4	2.9	0.0	22.7	9.1
5	0.0	0.0	0.0	29.4	2.2	0.0	22.7	9.1
6	0.0	0.0	0.0	23.5	4.7	0.0	18.2	0.0

Strength increase provided by the retrofitting solutions can be observed from Figure 7.2 to Figure 7.5. Nonlinear assessment of case study buildings revealed global strength reduction factors (R) and ductility factors (μ) as tabulated in Table 7.7. Retrofitting reduced R and μ demands significantly especially for the Bakırköy building which is weaker and has a stronger retrofitting solution. As strength increases with addition of nonductile members (shear walls), ductility decreases as expected.

First mode shapes and storey displacements for a specific roof displacement are given in Figure 7.6 to Figure 7.9. Mode shapes are normalized with respect to roof displacements. Storey displacements correspond to the ultimate roof displacement of existing and retrofitted buildings, whichever is smaller. Observing these figures, the change in mode shape with the addition of new members significantly changes the storey drift and accordingly the member rotation demands. In case study 4 building, bottom end of the column 1S03 collapses and thus the pushover analysis stops at a roof displacement demand of 0.113m. However, same column did not even yield at same roof displacement of the existing building. The reason is that the first storey drift is 0.010m for existing building, whereas it is 0.028m for retrofit option at same top storey displacement. Similar behavior is observed for the Düzce building. In the +X direction, first storey drift capacity of the existing building is less than the retrofitted building and thus existing building reaches its collapse point before retrofitted building due to collapse of an existing wall. On the other hand, in the +Y direction mode shapes and storey displacement demands are close to each other, resulting in closer collapse displacements. The reason is that shear walls govern the displacement shape in both existing and retrofitted cases of the Düzce building. The situation is different in case study 3 and 4 (Bakırköy) where a frame system is transformed into a wall system by retrofitting. Accordingly drift demands change considerably. Displacement demand of a brittle first storey column in a frame increases significantly due to the added wall, even where the roof displacement demands of the two systems are equal.

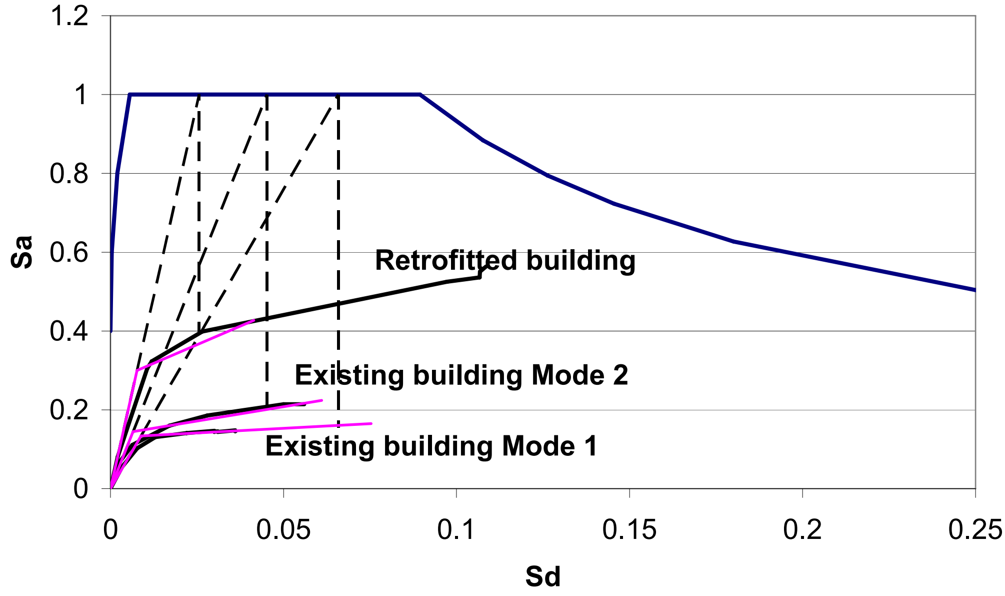


Figure 7.2 : ADRS curves for case study 1 (existing) and 2 (retrofitted) buildings in the +X direction

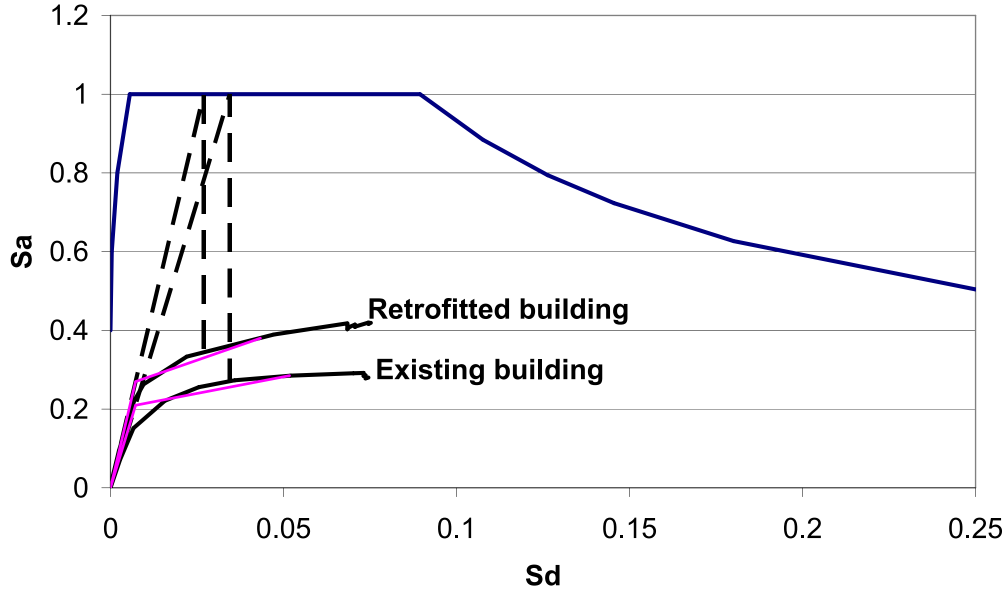


Figure 7.3 : ADRS curves for case study 1 (existing) and 2 (retrofitted) buildings in the +Y direction

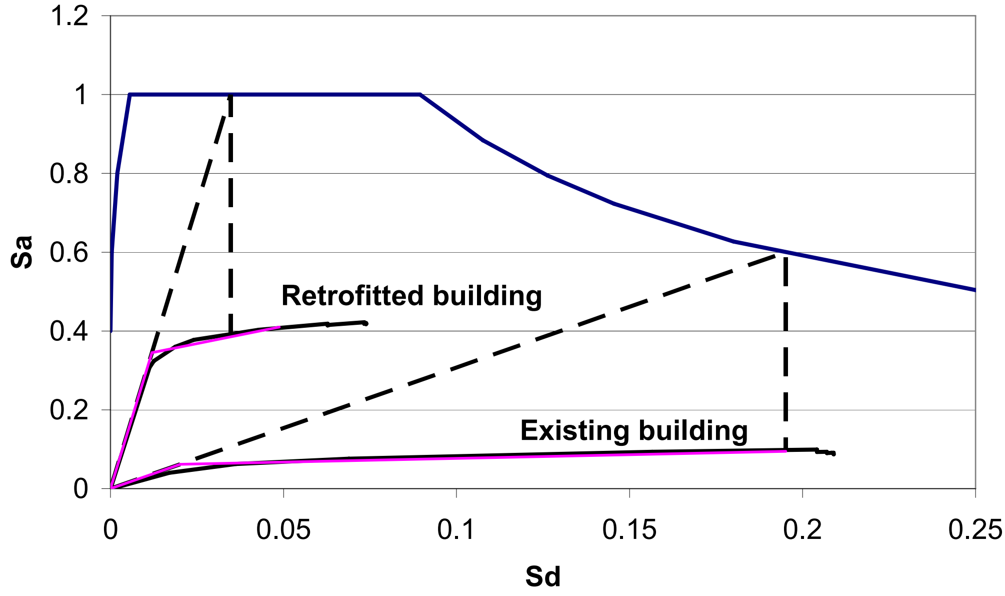


Figure 7.4 : ADRS curves for case study 3 (existing) and 4 (retrofitted) buildings in the +X direction

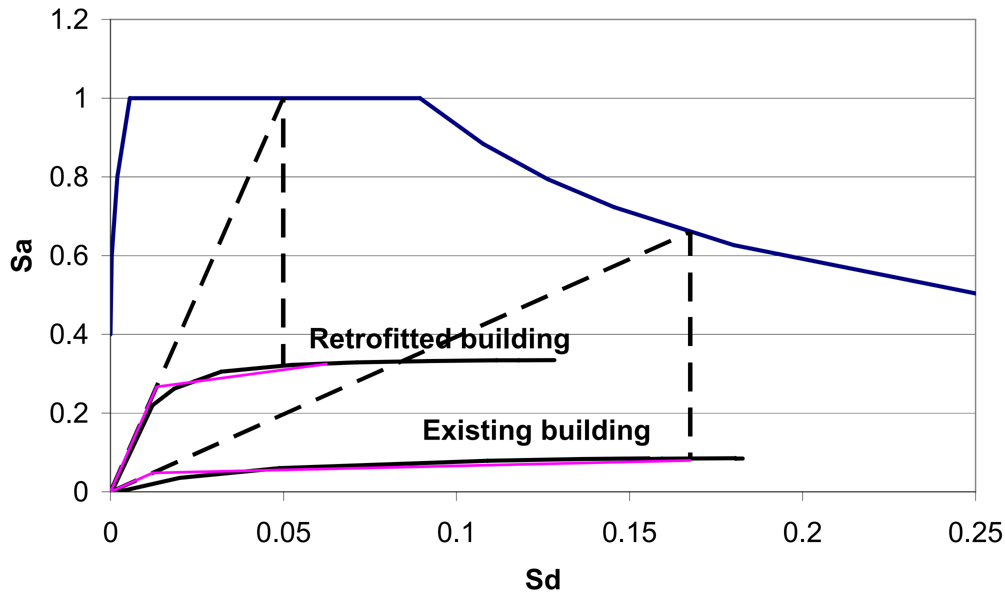
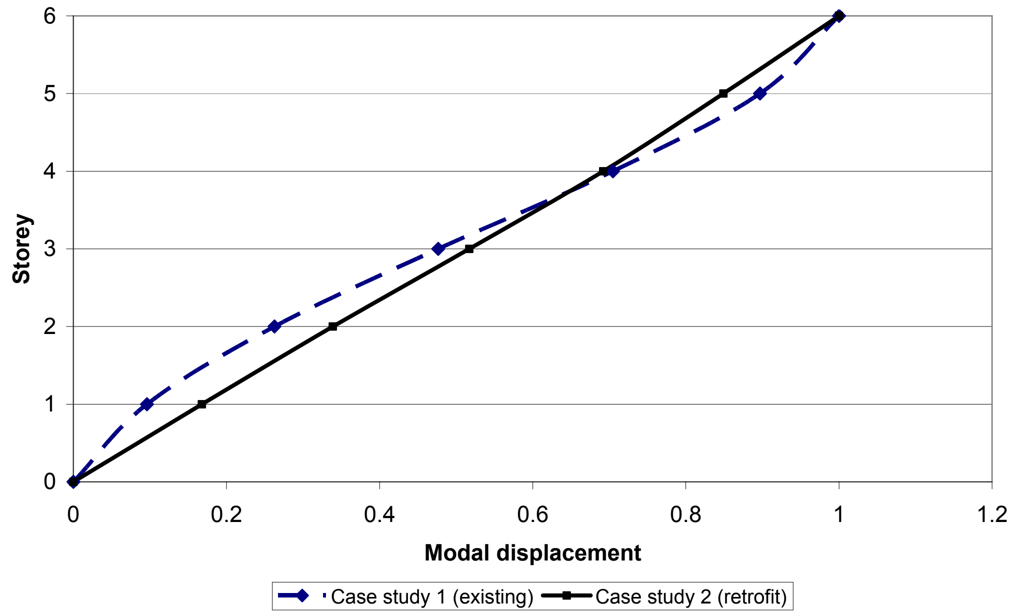


Figure 7.5 : ADRS curves for case study 3 (existing) and 4 (retrofitted) buildings in the +Y direction



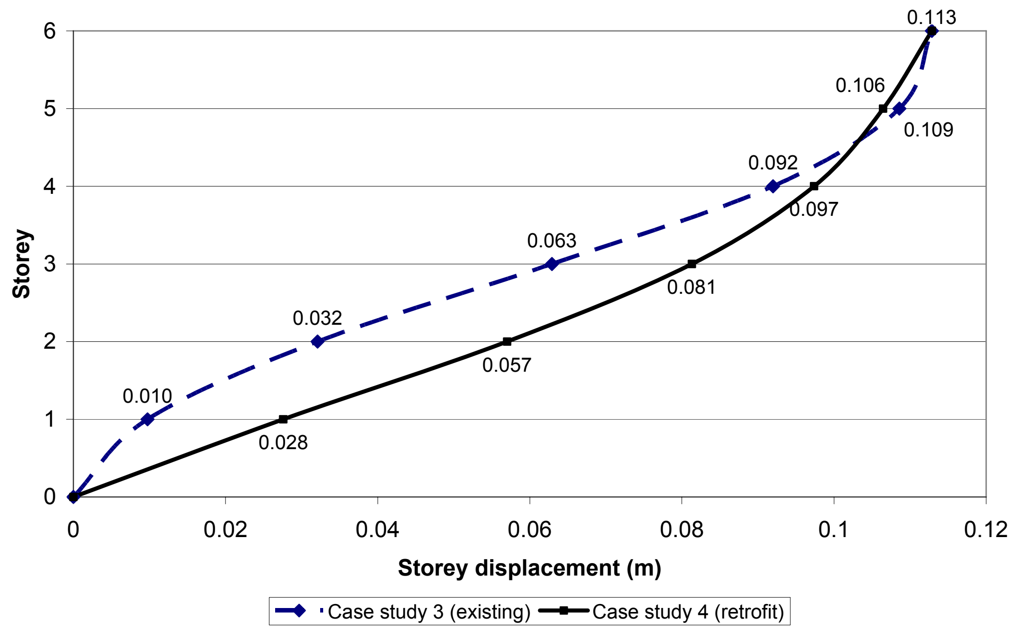
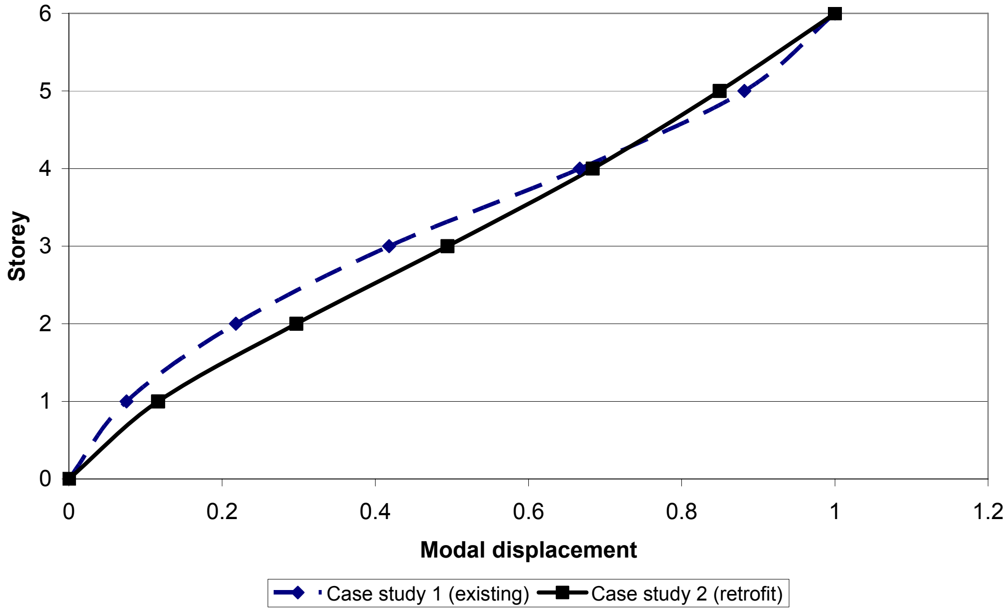


Figure 7.6 : Storey displacements for Case Study 3 (existing) and 4 (retrofitted) buildings in the +X direction



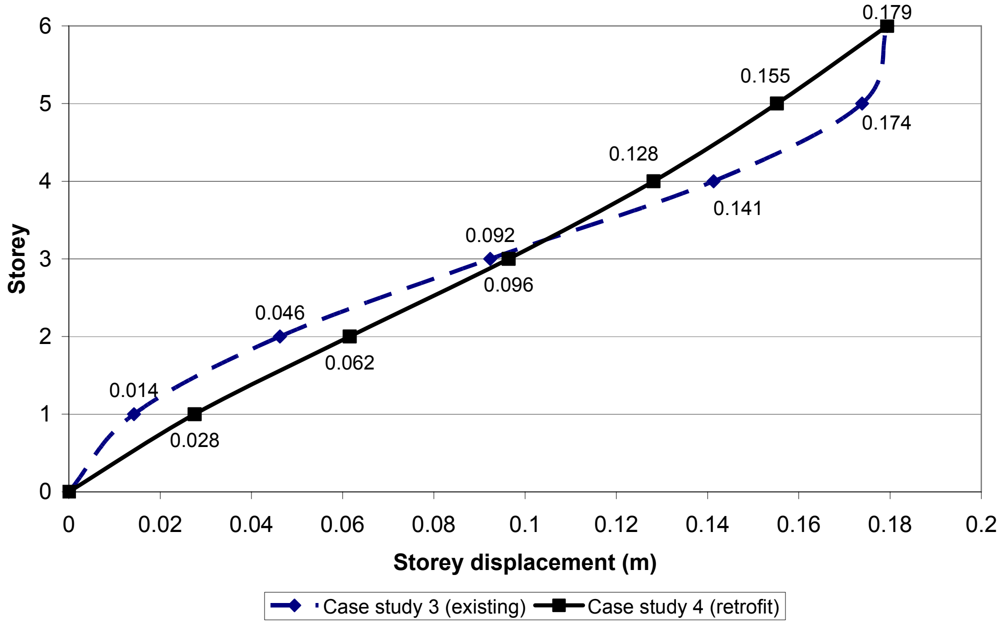
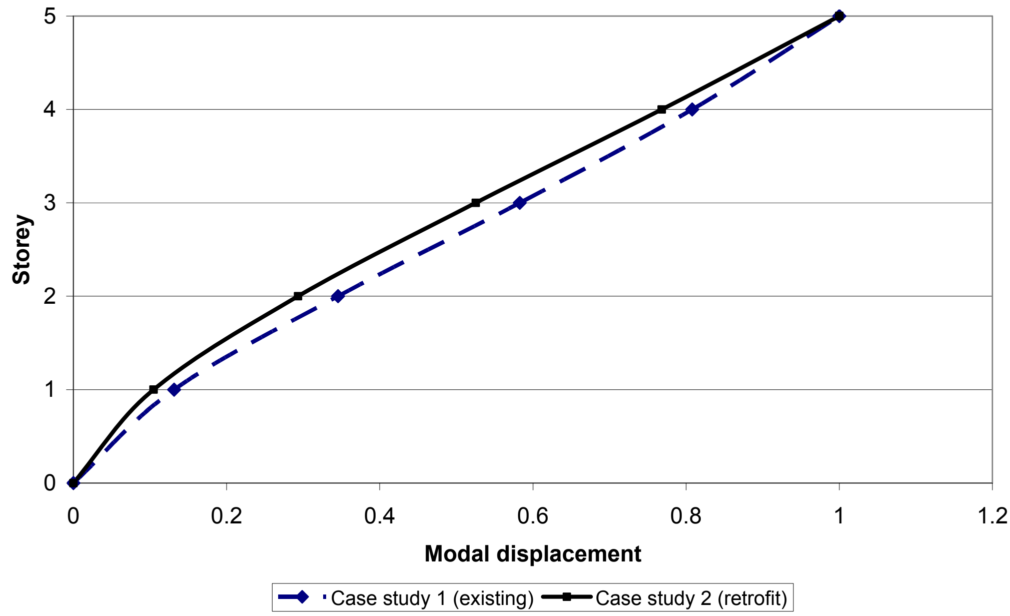


Figure 7.7 : Storey displacements for Case Study 3 (existing) and 4 (retrofitted) buildings in the +Y direction



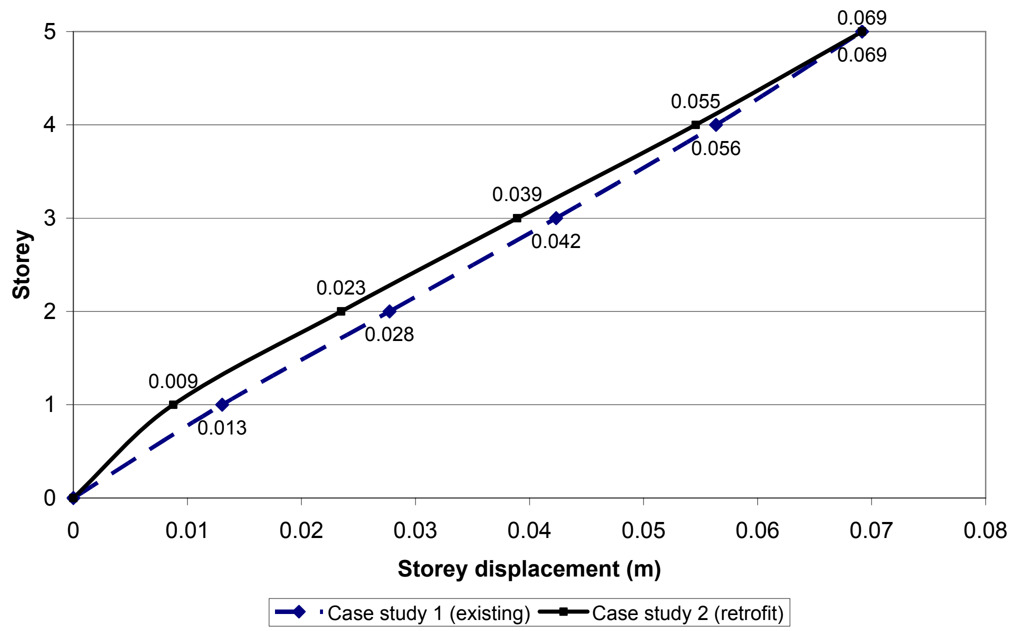
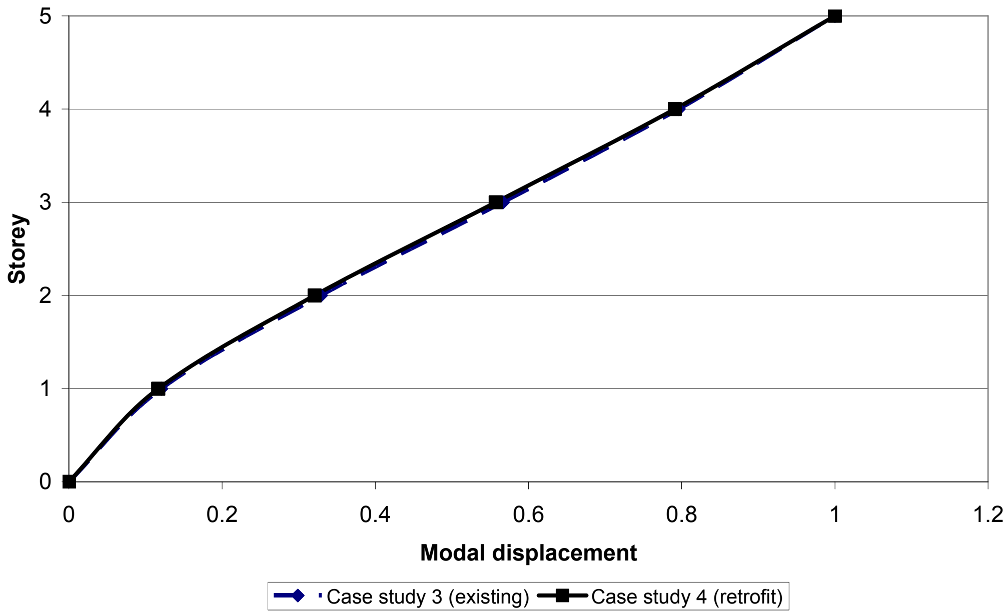


Figure 7.8 : Storey displacements for Case Study 1 (existing) and 2 (retrofitted) buildings in the +X direction



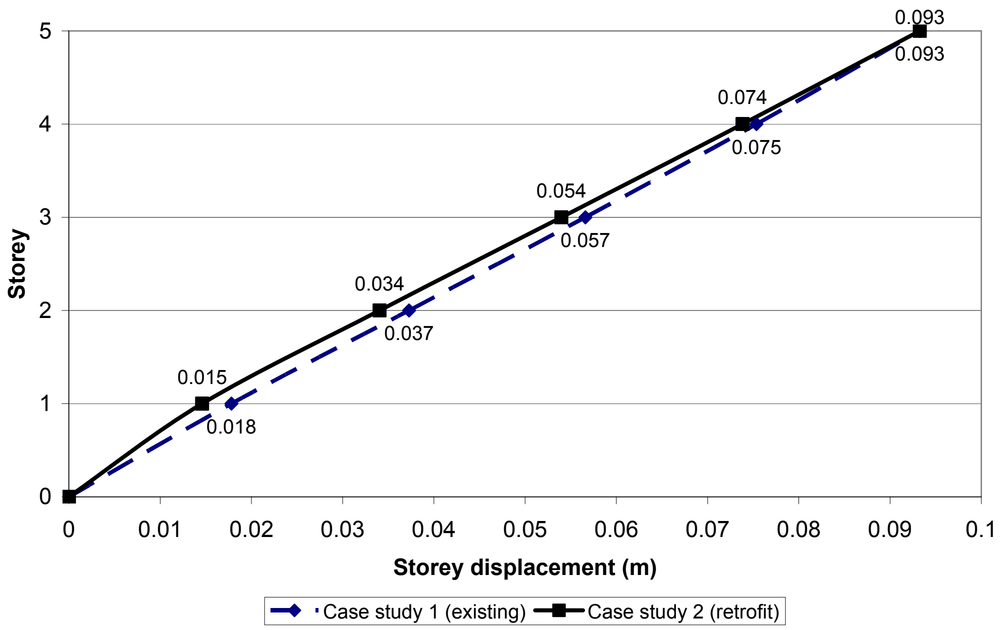


Figure 7.9 : Storey displacements for Case Study 1 (existing) and 2 (retrofitted) buildings in the +Y direction

Table 7.7 : R and μ demands for case studies

Case study	R		μ	
	+X	+Y	+X	+Y
1 **Düzce existing**	7.5 (mode 1)	4.8	8.5 (mode 1)	7.1
	6.9 (mode 2)		9.3 (mode 2)	
2 **Düzce retrofitted**	3.3	3.7	5.4	6.0
3 **Bakırköy existing**	9.7	13.8	9.7	13.8
4 **Bakırköy retrofitted**	2.9	3.7	4.1	4.7

7.2.4 Comparative Assessment for Anchorage Design

An anchorage methodology is represented in this study for both linear and nonlinear procedures. The only difference is the calculation of shear demands. Nonlinear procedure uses shear demand obtained form the pushover analysis directly, but linear procedure modifies elastic shear demands with capacity shears. This modification reduces the elastic demands to those of inelastic demands but still there exists some differences. Detailed storey shears of walls are given in Figure 7.10 and Figure 7.11.

Although storey anchorage shear demands vary between the two procedures, minimum anchorage requirement defined in the 2007 Turkish Earthquake Code governs the calculations for both procedures.

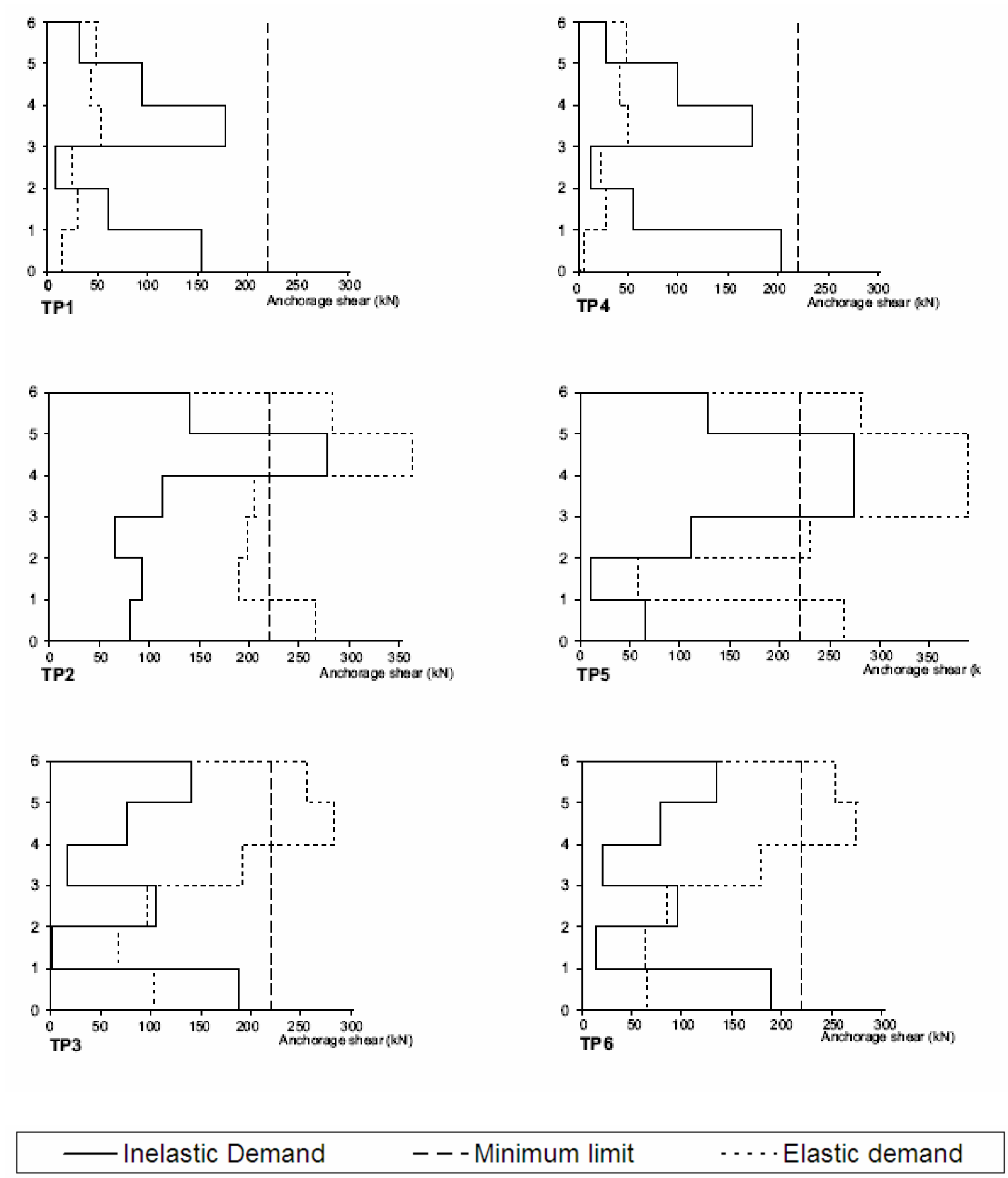


Figure 7.10 : ΔV diagram from linear elastic analysis for new walls

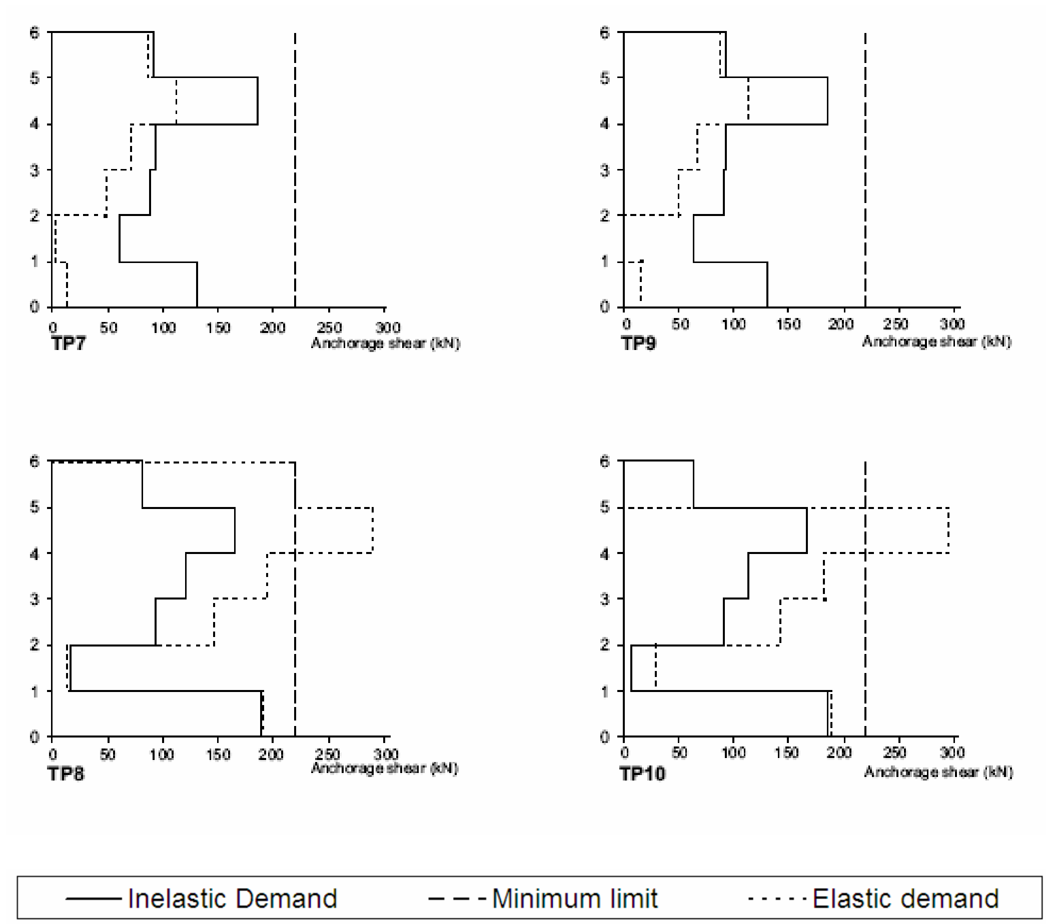


Figure 7.11 : ΔV diagram from nonlinear analysis for new walls

7.3 Conclusions

Seismic assessment of existing buildings was added to the 2007 Turkish Earthquake Code as a new chapter. Two different alternative procedures are introduced, namely linear elastic and nonlinear assessment procedures. In this study performance of both procedures are examined comparatively.

Based on the research performed in this study, following conclusions were drawn:

- Both assessment procedures have some advantages and disadvantages. 2007 Turkish Earthquake Code does not impose any limitation for the selection of one of the two procedures but the difference between these procedures and theory behind them must be well understood by the engineer in order to obtain correct and reliable results. Same building can have different results with different engineers even if the same procedure was used if the theory behind the procedures is not known by the engineer.
- By the comparison of actual damage observations with the damage prediction of the procedures, both procedures have overestimated the actual observed damage. Acceptance limits of both methodologies can be reassessed.
- Although both procedures underestimate the observed performance of the buildings, linear elastic procedure is much more conservative for beams when compared with the nonlinear procedure. The differences are less for columns and walls but they are still conservative.
- 2007 Turkish Earthquake Code gives equal importance to both procedures but the performance evaluation by using both procedures for a building can be different. This may cause conflict between owners and designers because both procedures are legally valid.
- In the linear elastic procedure, two comparative methods are suggested for moment capacity calculation of vertical members. Capacity Control Method sets the limit for the axial load calculated from the graphical procedure and it is noticed that generally capacity control method governs the axial load. Moreover, graphical procedure predicts different moment

capacities for the bottom and top ends of one column which is not possible. As a result, capacity control method can be accepted as sufficient for the calculation of moment capacity of vertical members.

- Both of the procedures are complicated for hand calculations and engineers need to trust the software products, that automate the procedures, for the results. This will make the procedures "black box" and engineers would never know if there is something wrong with the software procedure. Procedures can be simplified for better understanding.

7.4 Recommendations for Further Studies

This study includes a comparative assessment of both linear and nonlinear procedures for existing and retrofitted case studies. For further studies, increasing the number of case studies may result in a better correlation between the procedures. Moreover results can be compared with other guidelines such as FEMA 356, ATC-40 and Eurocode 8 in order to compare the results with other code procedures. Finally, a three dimensional time history analysis can be performed for the comparison of actual damage observed during the earthquake of case study 1 building if software products would become capable of this analysis.

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APPENDIX A

STRESS STRAIN DIAGRAM FOR CONCRETE

Moment-Curvature calculation requires a concrete model that takes into account the confinement of the section. In this study Modified Kent & Park (1971) model is selected and described below.

Different curves must be used for confined and unconfined sections. Kent and Park defines confined region as the region within the stirrup's outermost fiber.

Stress-strain diagrams shown in Figure A.1 are defined in two parts; parabolic curves and linear portions. They are defined shown below.

Unconfined Concrete *Confined Concrete*

Parabolic Curve

$$\sigma_c = f_c\{\frac{2\varepsilon_c}{\varepsilon_{co}} - (\frac{\varepsilon_c}{\varepsilon_{co}})^2\} \qquad \sigma_c = f_{cc}\{\frac{2\varepsilon_c}{\varepsilon_{coc}} - (\frac{\varepsilon_c}{\varepsilon_{coc}})^2\} \tag{A.1}$$

Where;

$$\varepsilon_{coc} = K\varepsilon_{co} \qquad K = 1 + \frac{\rho_s f_{ywk}}{f_c} \tag{A.2}$$

Unconfined Concrete *Confined Concrete*

Linear portion

$$\sigma_c = f_c(1 - Z_u(\varepsilon_c - \varepsilon_{co})) \qquad \sigma_c = f_{cc}(1 - Z_c(\varepsilon_c - \varepsilon_{coc})) \geq 0.2f_{cc} \quad (A.3)$$

$$Z_u = \frac{0.5}{\varepsilon_{50u} - \varepsilon_{co}} \qquad Z_c = \frac{0.5}{\varepsilon_{50u} + \varepsilon_{50h} - \varepsilon_{coc}} \quad (A.4)$$

$$\varepsilon_{50u} = \frac{3 + 0.285f_c}{142f_c - 1000} \geq \varepsilon_{co} \qquad \varepsilon_{50h} = 0.75\rho_s\left(\frac{b_k}{s}\right)^{1/2} \quad (A.5)$$

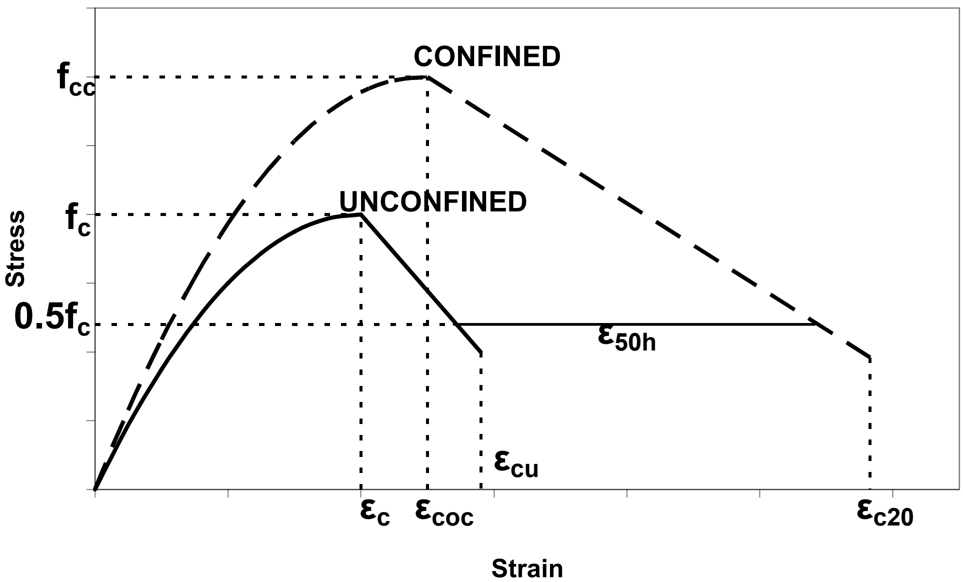

Figure A 1 : Stress - Strain curves