COMPARISON OF SEISMIC ASSESSMENT PROCEDURES IN THE CURRENT TURKISH CODE

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ABSTRACT

COMPARISON OF SEISMIC ASSESSMENT PROCEDURES IN THE CURRENT TURKISH CODE

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In Turkey, most of the existing buildings are vulnerable to earthquakes due to their poor material quality and inaccurate design. Besides, so many destructive earthquakes occurred in the past, because Turkey is located on a seismically active region. Therefore, existing buildings should be assessed and necessary precautions should be taken before a probable earthquake. To assess seismic performance of the existing buildings, the 2007 Turkish Earthquake Code offers two methods which are linear and nonlinear. For linear assessment, members are controlled by comparing the force demands and capacities where for nonlinear assessment, strains corresponding to the plastic rotations of the members are compared with the limits given in the code.

In this study, the building, which stands in Bakırköy district of İstanbul, was assessed according the linear elastic and nonlinear static procedures given in the 2007 Turkish Earthquake Code. In addition, it was retrofitted by adding shear walls to the structural system and same assessment procedures were performed. In the last case study, building is re-designed according to the code and re-assessed. Comparative results and conclusions were summarized in the last chapter.

Keywords: 2007 Turkish Earthquake Code, assessment, linear elastic procedure, nonlinear static procedure, global performance, retrofit

ÖZ

MEVCUT TÜRK YÖNETMELİĞİNDEKİ SİSMİK DEĞERLENDİRME PROSEDÜRLERİNİN KARŞILAŞTIRILMASI

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Türkiye'de mevcut binaların çoğu kötü malzeme kalitesi ve hatalı tasarım nedeniyle depremlere karşı zayıftır. Ayrıca, Türkiye'nin sismik olarak aktif bir bölgede yer alması sebebiyle geçmişte bir çok yıkıcı deprem meydana gelmiştir. Bu sebeplerden dolayı, olası bir depremden once mevcut binalar değerlendirilmeli ve gerekli önlemler alınmalıdır. Binaların sismik performansını değerlendirmek için, 2007 Türk Deprem Yönetmeliği doğrusal ve doğrusal olmayan olmak üzere iki yöntem sunmaktadır. Doğrusal değerlendirmede elemanların kuvvet talepleri ve kapasiteleri karşılaştırılırken doğrusal olmayan değerlendirmede plastik dönmelere denk gelen gerilmeler yönetmelikteki sınırlar ile karşılaştırılmaktadır.

Bu çalışmada İstanbul'un Bakırköy ilçesinde bulunan bir bina 2007 Türk Deprem Yönetmeliği'nde bulunan doğrusal elastik ve doğrusal olmayan statik yöntemlerle değerlendirilmiştir. Buna ek olarak, bina yapısal sistemine perde duvarlar eklenerek güçlendirilmiş ve aynı değerlendirme yöntemleri uygulanmıştır. Son durumda ise bina yönetmeliğe göre yeniden tasarlanmış ve değerlendirilmiştir. yönetmeliğe gore yeniden tasarlanmıştır. Karşılaştırmalı neticeler ve çalışmanın sonuçları son bölümde özetlenmiştir.

Anahtar kelimeler : 2007 Türk Deprem Yönetmeliği, doğrusal elastik analiz, doğrusal olmayan statik analiz, global performans, güçlendirme

To My Family

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CHAPTER I

INTRODUCTION

1.1 General

Earthquake is one of the major sociological, psychological, economical and engineering problems in Turkey. Especially after Marmara and Düzce Earthquakes in 1999, community is more interested in this natural event. Many buildings were heavily damaged or collapsed after these earthquakes which resulted in great loss of life. These observations are believed to be the result of several factors including poor material quality and workmanship, inadequate detailing and proportioning, irregularities and insufficient supervision during their constructions. Considering that most of the existing buildings in Turkey have these problems, they are vulnerable to severe damage or collapse in an earthquake. Available assessment procedures in literature provide ways of determining the seismic performance of the existing buildings and probable retrofit options.

The 2007 Turkish Earthquake Code offers two main procedures for assessment of the existing buildings [1]. One of them is linear and the other is nonlinear assessment procedure. Linear assessment procedure compares flexural and shear capacities of the members with the demands calculated by elastic analysis. On the other hand, the nonlinear procedure is displacement based where member deformations are taken into consideration. Each procedure has certain limitations and considered to be more reliable under certain circumstances. The use of the more appropriate procedure requires a thorough understanding of their limitations. This study aims to evaluate these procedures by their application to an existing building and its modified models. By examination of the relative results their validity is evaluated.

1.2 Other Studies

There are several procedures for performance assessment in the literature. Although methodologies and some descriptions may be different, main steps of the procedures are same. For linear assessment procedure, calculation of elastic earthquake forces may be different from one code to another. However, final assessment procedure is same where elastic demands and member capacities are compared. For nonlinear assessment procedure, main difference is in calculating target displacement of the building and the criteria employed for the acceptance. The acceptance criteria are generally based on the performance limit values that may differ.

The most common assessment procedures are explained in three main guidelines/codes which are ATC-40, FEMA 356 and Eurocode 8. As it is mentioned earlier, they have many similarities and some differences. These are explained briefly in the following paragraphs.

1.2.1 American Technology Council - ATC-40 Procedure [2]

Traditional retrofit design techniques assume that buildings respond elastically to earthquakes. In reality, large earthquakes can severely damage building causing inelastic behavior that dissipates the energy. Therefore, in modern codes displacement based analysis and design by using nonlinear parameters are used to assume the real behavior of a building under an earthquake.

ATC-40 provides a procedure that is based on nonlinear analysis. Failure mode of each member is estimated depending on whether it is ductile or brittle. Then, plastic rotation capacities of each member are estimated from moment curvature diagrams. After performing a nonlinear analysis, pushover curve is obtained for the building. ATC-40 evaluates the performance point of the building using capacity spectrum method. This method requires that both the demand response spectra and pushover curves be plotted in the spectral acceleration vs. spectral displacement domain. Spectra plotted in this format are known as

Acceleration – Displacement Response Spectra (ADRS) [2]. Figure 1.1 basically shows how to estimate performance point by using capacity spectrum method.

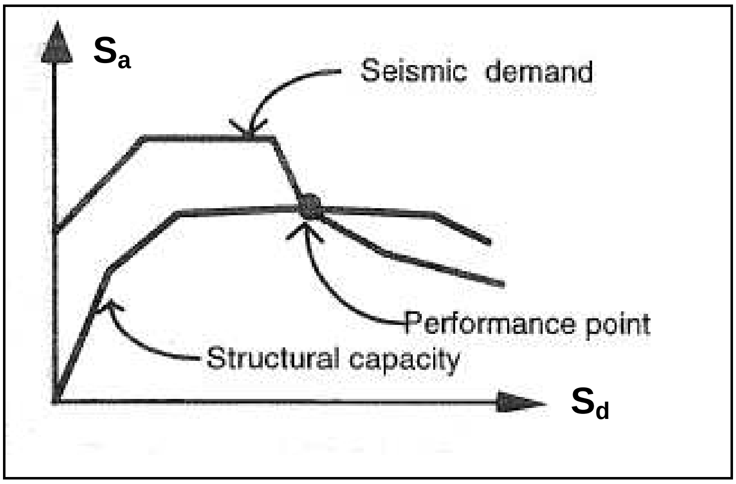

Figure 1.1 Capacity Spectrum Method [2]

After estimating the plastic rotations of each member, they are compared with the performance based limit values which are given for each member type according to three performance levels; Immediate Occupancy, Life Safety and Collapse Prevention.

1.2.2 Federal Emergency Management Agency - FEMA 356 Procedure [3]

FEMA 356 is a code to evaluate the seismic performance of the buildings by using both linear and nonlinear procedures. In this code, Demand Capacity Ratios are calculated for each member for linear procedure (Equation (1.1)).

$$DCR = \frac{Q_{UD}}{Q_{CE}} \tag{1.1}$$

where Q_{UD} = force due to the gravity and earthquake loads

Q_{CE} = expected strength of the component or element

DCR is calculated for each end and the largest of DCRs is critical for a member.

FEMA restricts the applicability of the linear analysis for the following cases.

1. If all component DCRs $\leq$ 2.0, then linear procedure is applicable.

2. If one or more component DCRs exceed 2.0, and no horizontal or vertical irregularities are present, then linear procedures are applicable,

3. If one or more component DCRs exceed 2.0 and any irregularity is present, then linear procedures are not applicable, and shall not be used.

For nonlinear procedure, nonlinear parameters of the members are imported to the model. Analysis is performed and pushover curve is obtained. An effective period is defined in FEMA 356 which is calculated by using Equation (1.2). It is related to initial period, initial stiffness and effective lateral stiffness of a building.

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \tag{1.2}$$

where T_i = elastic fundamental period in the direction under consideration calculated by elastic demand.

K_i = elastic lateral stiffness of the building in the direction under consideration

K_e = effective lateral stiffness of the building in the direction under consideration

Effective lateral stiffness, K_e, is the slope of the line which passes through 0,60 V_y of the pushover curve.

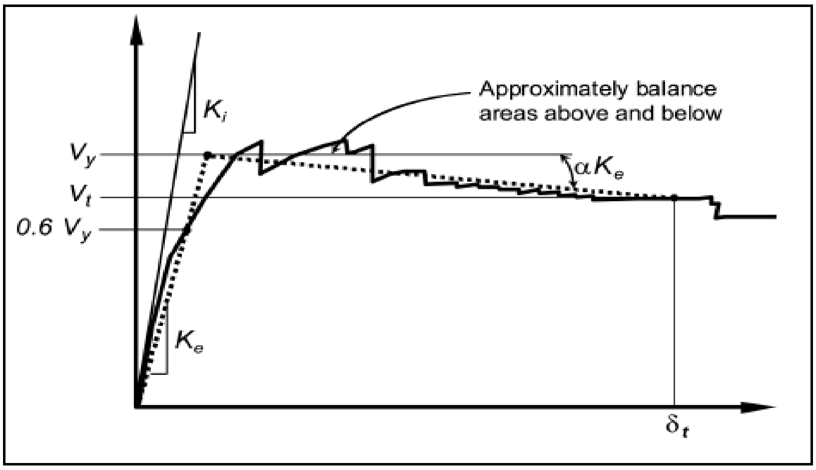

Figure 1.2 Negative Post-yield Slope [3]

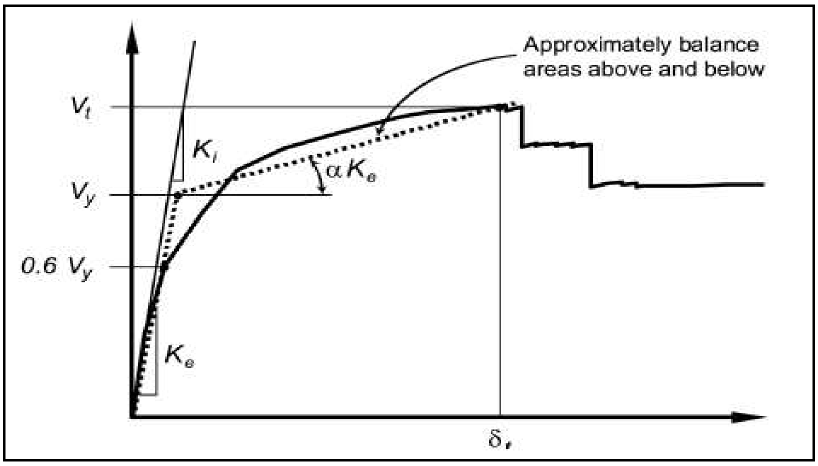

Figure 1.3 Positive Post-yield Slope [3]

In FEMA 356, target displacement is calculated by the Coefficient Method. According to this method, target displacement is calculated by multiplying the elastic single degree of free displacement with coefficients which are simply related to load pattern used in analysis, effective period, frame type and post-yield stiffness shown in Figures 1.2 and 1.3.

The building is pushed up to the target displacement level and plastic rotations in the members are checked with the limit values.

Acceptance criterion for nonlinear assessment is shown below.

$$m\kappa Q_{CE} > Q_{UD} \tag{1.3}$$

where m = component or element demand modifier to account for expected ductility associated with this action at the selected structural performance level

κ = knowledge factor

Q_{CE} = expected strength of the component or element at the deformation level under consideration for deformation-controlled action

1.2.3 Eurocode 8 Procedure [4]

Eurocode 8 provides guidelines for equivalent lateral force analysis, multi-modal response spectrum analysis, nonlinear static analysis and nonlinear time history analysis that are used in the assessment.

Assessment procedures are nearly same as the other codes. However, calculating target displacement for nonlinear static analysis is different. Eurocode 8 offers a method called Equivalent Single Degree of Freedom (SDOF) Method. According to this method, mass, force and displacement parameters of the building are transformed to an equivalent SDOF system. Later, period of the idealized equivalent SDOF system is determined and finally target displacement is calculated.

Damage limits in Eurocode 8 are also different from other codes as itemized below [3].

- *Limit State of Near Collapse* means that capacities shall be based on appropriately defined ultimate deformations for ductile elements and on ultimate strengths for brittle ones.
- *Limit State of Significant Damage* means that capacities shall be based on damage–related deformations for ductile elements and on conservatively estimated strengths for brittle ones.
- *Limit State of Damage Limitation* means that capacities shall be based on yield strengths for all structural elements both ductile and brittle.

There are also some recent studies about the 2007 Turkish Earthquake Code. Düzce Z. (2005) performed both linear and nonlinear assessments and concluded that linear procedure was more conservative than nonlinear [12]. Besides, Şengöz A. (2007) assessed a building which was slightly damaged during Düzce Earthquake. Linear and nonlinear assessment procedures were applied to the building and it was concluded that although linear assessment was more conservative than nonlinear, they both overestimated the actual damage. He offered to re-assess the acceptance limits for both procedures in the 2007 Turkish Earthquake Code [13].

1.3 Objective and Scope

In this study, two assessment procedures in the 2007 Turkish Earthquake Code are studied in detail. Moreover, nonlinear limits of FEMA 356 are applied to the buildings. Main objective is to reach some comparative results and check the consistency of the assessment procedures in the code. In the second chapter, steps of each procedure are explained briefly. Later, a residential building which is located in Bakırköy district of İstanbul is assessed by linear and nonlinear procedures in the chapter three. In the chapter four, this building is retrofitted by adding four shear walls in the X direction and two shear walls in the Y direction. Retrofitted building is also assessed by using the same procedures. Furthermore, in the fifth chapter, it is re-designed by changing the cross section and reinforcement details of each member according to the 2007 Turkish Earthquake Code. Similarly, re-designed building is evaluated by linear and nonlinear procedures. Three models of the case study building are also assessed according to FEMA 356 limits and all results are compared in the last chapter of the study.

It is expected that, this study gives some ideas about

- application of linear and nonlinear assessment procedures,
- comparison between assessment results of a building for its three different models,
- comparative results between linear and nonlinear procedures of the code,
- comparative results between the 2007 Turkish Earthquake Code and FEMA 356.
- consistency of design and assessment sections of the code.

CHAPTER II

DESCRIPTION OF ASSESSMENT METHODS FOR EXISTING BUILDINGS IN THE 2007 TURKISH EARTHQUAKE CODE

The current seismic design code in Turkey recommends two procedures to be used for seismic performance assessment of existing reinforced concrete buildings. These procedures are based on linear and nonlinear analyses of the structure to be assessed. In the linear assessment, equivalent static lateral load analysis or dynamic analysis can be used. The nonlinear assessment is carried out based on either nonlinear static (pushover) or nonlinear dynamic analyses. In this thesis, linear elastic and nonlinear (pushover) assessment procedures of 2007 Turkish Earthquake Code are employed. These procedures are summarized below.

2.1 Linear Assessment Procedure

Linear elastic procedure is an assessment in which the building is analyzed elastically under vertical (gravity and live loads) and earthquake loads separately. After analysis, demands and capacities are calculated for each member of the building. The members are classified as either brittle or ductile. To identify the type of expected behavior, shear forces are compared by the corresponding capacities. For brittle members, assessment is done based on shear force whereas for ductile members flexural forces are checked by comparing the demand forces with the capacities of members. Based on these comparisons, the expected damage of each member and overall performance of the structure is estimated.

The choice of the assessment and analysis procedure is based on certain criteria. Linear elastic procedure can be applied to the buildings which

- are at most 25 m in height from ground level,
- have at most 8 stories,
- have torsional irregularity constant smaller than 1.4, (η_{bi} < 1.4).

As summarized in the the flow chart given in Figure 2.1, the linear elastic assessment procedure has two main steps.

1) Modeling and analysis
2) Performance assessment and acceptance

These two main components involve sub steps which are briefly explained next. It is important to note that this flow chart does not include the data collection requirements of the procedure but focuses on the analysis and assessment.

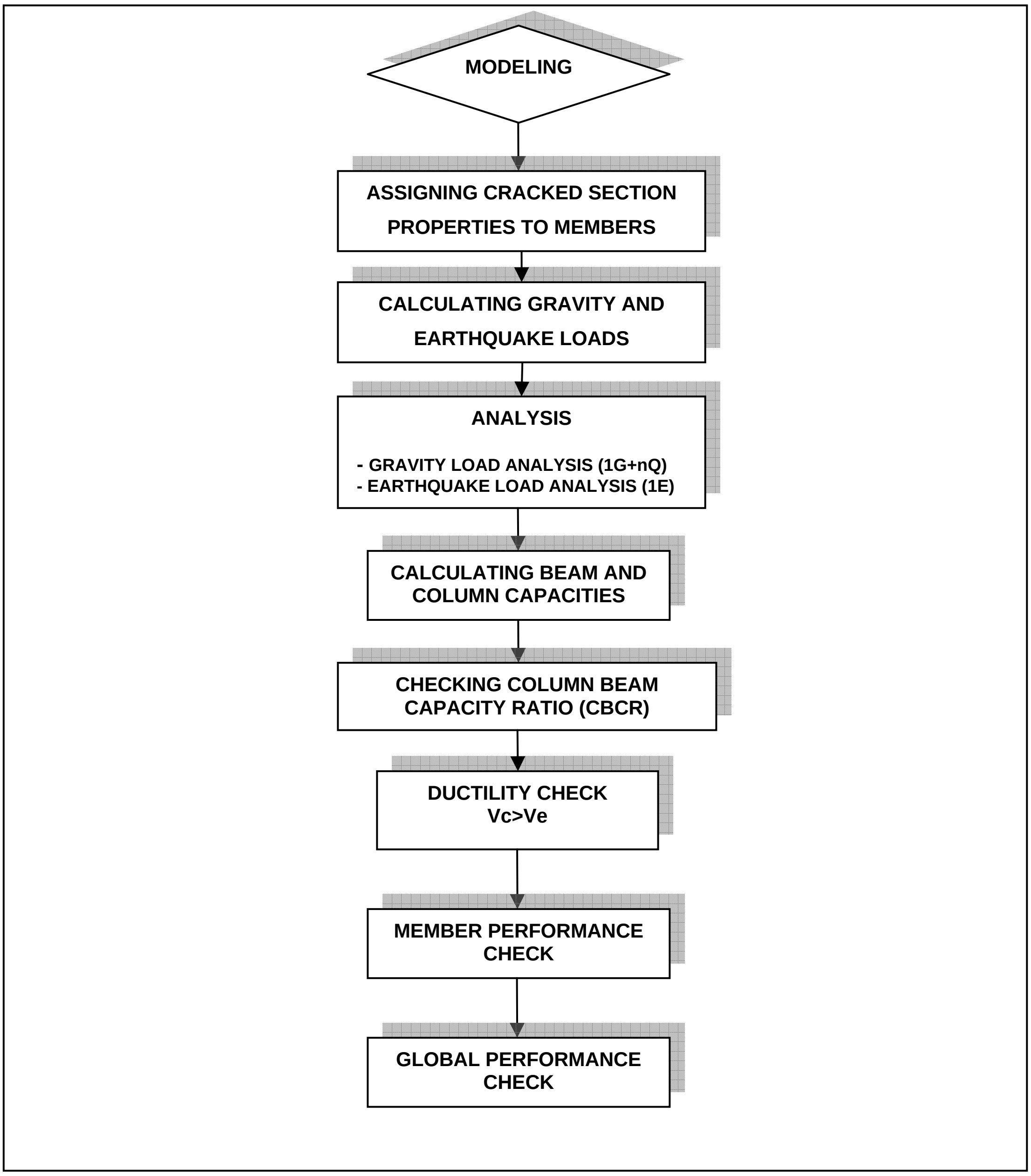

Figure 2.1 Linear Assessment Steps

2.1.1 Modeling and Analysis

Modeling is the first step of the linear assessment procedure. 3-D model of the building is prepared in a computer program. By looking at the design drawings or building field survey results, beams, columns, shear walls, slabs and other load carrying members are modeled. Beams and columns are modeled as line elements which are connected to each other at the joints. However, shear walls and slabs are more complicated. Shear walls can be modeled either as wide columns or by finite element meshes (shell elements). In this study, wide column model is used. They are assumed as columns which stand in the center of the wall, and they are connected to the system by rigid beams. Also, slabs can be modeled by finite element meshes. However instead of this, assigning a rigid diaphragm to floors, lumping the slab mass at the center, and distributing the gravity loads of a slab to the beams is more common and practical way of modeling.

Members are assumed to have cracked cross sections in this assessment. Therefore, their rigidities should be reduced according to the gravity load level, cross sectional area and strength of concrete. The code requires that cracked stiffness of the members should be as follows:

a) Beams, $(EI)_c = 0.40\ (EI)_0$ (2.1)

b) Columns and Shear Walls, $N_d / (A_c\ f_{cm}) \leq 0.10 \rightarrow (EI)_c = 0.40\ (EI)_0$

$N_d / (A_c\ f_{cm}) \geq 0.40 \rightarrow (EI)_c = 0.80\ (EI)_0$

where $(EI)_c$ = stiffness of cracked section

$(EI)_0$ = stiffness of uncracked section

N_d = axial load computed from gravity loads (1G+nQ)

A_c = cross section of the member

f_{cm} = concrete strength

Interpolation can be used if $N_d / (A_c\ f_{cm})$ is between 0.1 and 0.4

After an acceptable modeling, earthquake loads to be applied are calculated. The most common procedure to calculate earthquake load is *Equivalent Static Load Method*, which is given in the second chapter (Chapter 2.7) of the 2007 Turkish Earthquake Code. In this method, it is assumed that the dominant response mode of the structure expected during an earthquake is the first mode. The base shear force is calculated from the design acceleration response spectra given in the code and it is distributed to floors in a triangular pattern using Equations (2.2), (2.3) and (2.5).

$$V_t = \lambda \frac{W\,A(T)}{R(T)} \tag{2.2}$$

$$A(T) = A_0\, I\, S(T) \tag{2.3}$$

where A_0 = seismic zone coefficient

I = importance factor of the building

R = earthquake load reduction factor

$S(T)$ = spectrum constant

W = weight of the building

λ = 1 for the buildings which have at most 2 storeys (except basement) and 0.85 for the others.

Importance factor (I) and reduction factor (R) are assumed as 1 for linear elastic analysis. This means that the estimated earthquake load is applied to the building without any reduction. Earthquake forces are distributed to the floors proportional to floor heights and masses by using Equation (2.5).

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

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

where ΔF_N = additional force to top floor

N = number of storeys

V_t = total shear force

w_i = i^{th} floor weight

H_i = height of i^{th} building from ground level

For each direction, gravity (1G+nQ) and earthquake (1E) forces are applied to the building separately and demands are recorded for further steps of the analysis.

2.1.2 Calculation of Member Capacities

In the 2007 Turkish Earthquake Code, seismic moment capacity of a member is defined as residual moment capacity which is the remaining amount of bending moment capacity after excluding the moment coming from gravity loads.

According to this definition, member capacity is calculated by the following equation.

$$M_c = M_p - M_d \tag{2.6}$$

where M_c = residual moment capacity

M_p = positive or negative bending moment capacity

M_d = positive or negative moment computed from gravity load analysis

Capacity calculation is straightforward and simple for beams. Ultimate capacity of the member is calculated from section analyses first. Then, the bending moment demand computed from the gravity loads is subtracted to determine the residual moment capacity. However, for columns, because they carry axial load and axial load affects the capacity of a column, capacity depends on the gravity and earthquake analysis results. Therefore, a new methodology for capacity of the columns is given in the 2007 Turkish Earthquake Code. According to this procedure, first of all, moment interaction diagram of a column is plotted. Then, moments and axial loads [(N_D, M_D) and (N_E, M_E)] are obtained from vertical load (1G+nQ) and earthquake load (1E) analyses respectively. Later, these points are plotted on the moment interaction diagram. Finally, intersection of (N_D, M_D) - (N_D+N_E, M_D+M_E) line and interaction diagram gives the capacity of the column, (N_K, M_K). This methodology is displayed in Figure 2.2.

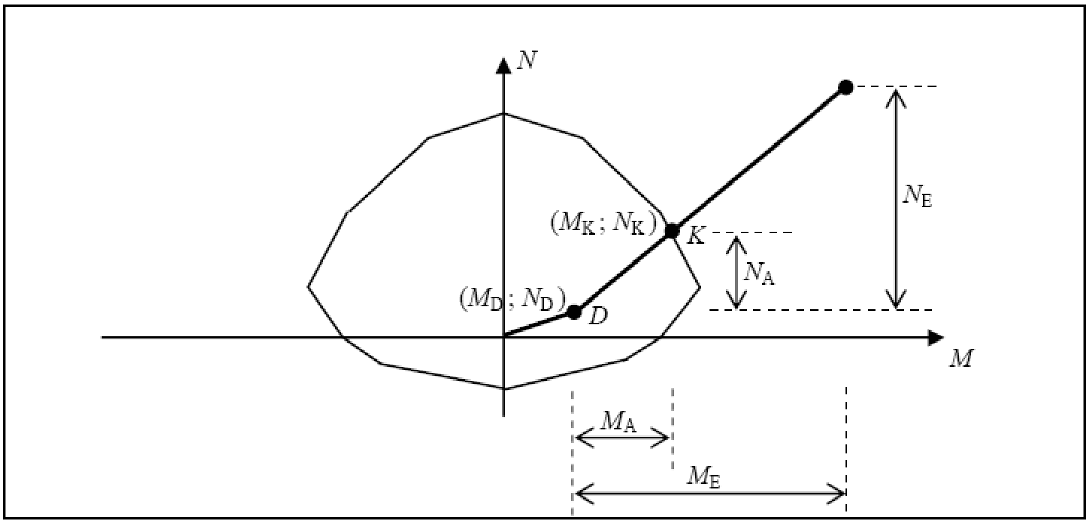

Figure 2.2 Calculation of Column Capacity [1]

Axial load capacities of the vertical members are constrained by an upper limit in the code. It is known that axial loads in the columns come from both the beams that are connecting to those columns and the columns above. Therefore, maximum axial load in a column will be equal to sum of the total shear forces in the beams determined based on the ultimate beam capacities and total axial load comes from upper columns.

2.1.3 Ductility Check

Ductility is a behavior which allows a material to go beyond large plastic deformations without a significant change in strength. More ductility means more displacement capacity during an earthquake.

A brittle member fails due to shear forces before reaching its moment capacity. On the other hand, a ductile member fails due to flexure before reaching its shear capacity. Therefore, while estimating the performance of a building, it is important to know which members are ductile and which are not.

Ductility is checked by comparing the shear force demand with the shear force capacity of a member.

Shear force demand of a beam is estimated by the following equation.

$$V_e = V_{dy} \pm (M_{pi} + M_{pj}) / l_n \qquad (2.7)$$

where V_e = shear force carried by beam

V_{dy} = shear force calculated from the simply supported beam analysis

M_{pi} = ultimate moment capacity of a beam in i end

M_{pj} = ultimate moment capacity of a beam in j end

M_{ri} = bending moment capacity of a beam in i end

M_{rj} = bending moment capacity of a beam in j end

l_n = net length of the beam

Unless detailed analysis are performed, M_{pi} and M_{pj} can be assumed as 1.4 M_{ri} and 1.4 M_{rj} in Equation (2.7).

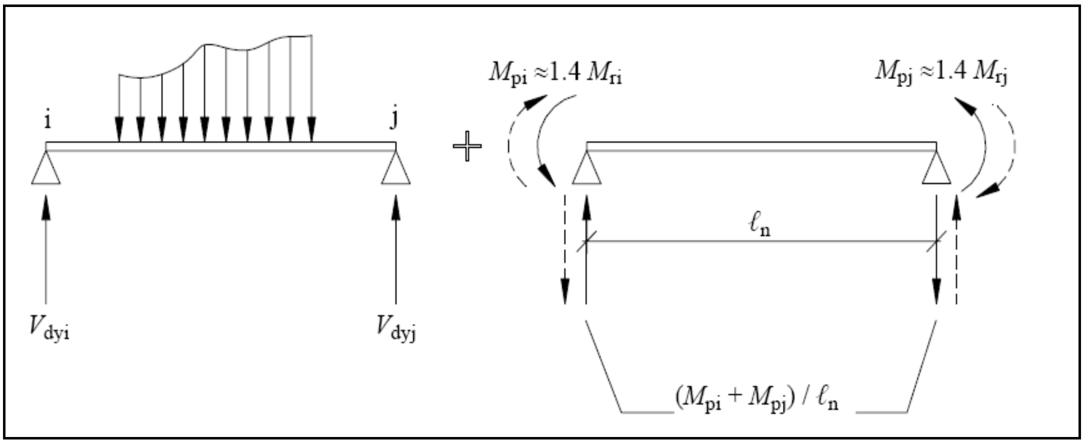

Figure 2.3 Calculation of Shear Force Carried by a Beam (V_e) [1]

Equation (2.7) shows that a beam is subjected to shear forces from two main sources, one of which is gravity loads and the other is the capacity of itself. Shear forces from gravity laods is calculated by assuming the beam as simply supported (Figure 2.3). Furthermore, shear forces from flexural capacity are calculated by assuming that the ends of the beam are yielded (even they are not).

For columns, shear demand calculation is similar to beams. It is assumed that upper and lower end of a column reach their ultimate moment capacity and total shear demand is estimated from these moments.

$$V_e = (M_a + M_ü) / l_n \tag{2.8}$$

where M_a = bottom moment capacity of a column

$M_ü$ = top moment capacity of a column

l_n = net length of a column

However, capacity calculated as shown in Figure 2.2 may not be the actual capacity for a column. It should be controlled by applying Column Beam Capacity Ratio (CBCR) to the column-beam joint. CBCR is a ratio which checks strong column weak beam state. The 2007 Turkish Earthquake Code requires that, the total moment capacity of columns should be at least 20% more than moment capacity of beams at that joint (Equation (2.9)).

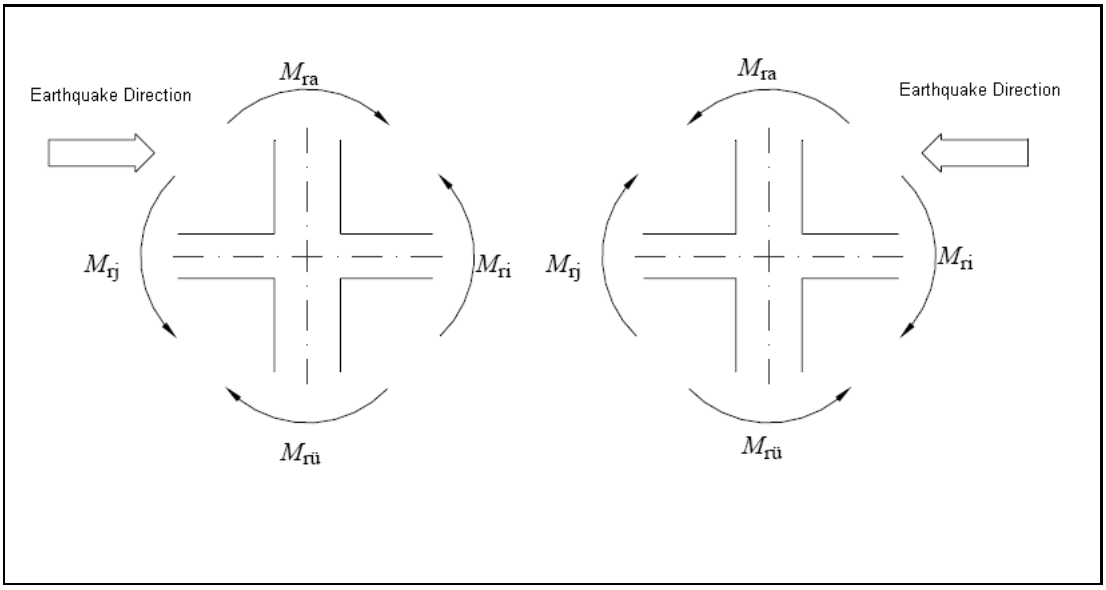

Figure 2.4 Moment Capacities of Beams and Columns at a Joint [1]

$$(M_{ra} + M_{rü}) \geq 1.2\,(M_{ri} + M_{rj}) \tag{2.9}$$

where M_{ra} = bending moment capacity of a column at lower end

$M_{rü}$ = bending moment capacity of a column at upper end

M_{ri} = bending moment capacity of a beam at i end

M_{rj} = bending moment capacity of a beam at j end

If CBCR is satisfied then moment capacities are modified as follows.

$$M_{ü} = \frac{M_{hü(i)}}{M_{hü(i)} + M_{hü(i+1)}} \sum M_p \tag{2.10}$$

and

$$M_{a} = \frac{M_{ha(i)}}{M_{ha(i)} + M_{ha(i+1)}} \sum M_p \tag{2.11}$$

where $M_{hü(i)}$ = top moment at ith floor column from horizontal load analysis

$M_{hü(i+1)}$ = top moment at (i+1)th floor column from horizontal load analysis

$M_{ha(i)}$ = bottom moment at ith floor column from horizontal load analysis

$M_{ha(i+1)}$ = bottom moment at (i+1)th floor column from horizontal load analysis

ΣM_p = total moment capacities of a beam connecting to a column ($M_{pi} + M_{pj}$)

If CBCR is not satisfied then the following equations are valid for column capacity.

$$M_a = M_{pa} \tag{2.12}$$

$$M_ü = M_{pü} \tag{2.13}$$

After calculating shear demands of the members, they are compared with the shear capacities. Shear capacity of a concrete beam or column is calculated by the following formulas in TS-500 [5].

$$V_r = 0.8\ V_{cr} + V_w \tag{2.14}$$

$$V_w = \frac{A_{sw}}{s} f_{ywk} d \tag{2.15}$$

$$V_{cr} = 0.65 f_{ctk} b_w d \left(1 + \gamma \frac{N_d}{A_c}\right) \tag{2.16}$$

where V_r = shear capacity of a member

V_w = contribution of stirrups to shear capacity

A_{sw} = total reinforcement area resisting shear force

A_c = cross sectional area of a member

f_{ywk} = yield strength of shear reinforcement

f_{ctk} = tensile strength of concrete

N_d = axial load in a member

b_w = width of cross section

d = depth of cross section

s = stirrup spacing

γ = 0.07 if member is under compression, -0.3 if it is under tension and 0 if f_{ctk} < 0.5 MPa

Unlike beams and columns, shear demand and capacity calculations have a different methodology for shear walls. According to the 2007 Turkish Earthquake Code, shear demand of a shear wall is calculated by the following equation.

$$V_e = \beta_v \frac{(M_p)_t}{(M_d)_t} V_d \tag{2.17}$$

where V_e = shear demand of shear wall

V_d = shear force calculated from gravity and earthquake analysis

$(M_p)_t$ = ultimate moment capacity of shear wall

$(M_d)_t$ = moment calculated from gravity and earthquake analysis

β_v = dynamic magnification coefficient

The capacities of shear walls are computed by using Equation (2.18) below.

$$V_r = A_{ch} (0.65 f_{ctd} + \rho_{sh} f_{ywd}) \tag{2.18}$$

where V_r = shear capacity of a shear wall

A_{ch} = total cross sectional area of a shear wall

f_{ctd} = tensile strength of concrete

ρ_{sh} = steel ratio of shear wall

f_{ywd} = yield strength of steel

After calculating the shear demand (V_e) and capacity (V_r) of members, by comparing these values, the expected behavior mode of the member is determined as follows.

If $V_r > V_e$, member is DUCTILE.

If $V_r < V_e$, member is BRITTE.

2.1.4 Performance of Members

Expected seismic performance of the members is estimated by comparing their demands with capacities. This comparison is affected by some parameters which are actually related to the ductility of a member. If a member is more ductile, there is more tolerance to go beyond its elastic limit. Therefore, for ductile members, the assessment is carried out based on the ratio of moment demand to residual moment capacity, defines as “r” ratio that is obtained for each member depending on its ductility. This ratio basically shows the level of capacity exceedance.

$$r = \frac{M_E}{M_P - M_D} \quad \text{or} \quad r = \frac{M_E}{M_C} \tag{2.19}$$

for beams and

$$r = \frac{M_E}{M_K - M_D} \quad \text{or} \quad r = \frac{M_E}{M_A} \tag{2.20}$$

for columns and shear walls.

These ratios are compared with the limits given in the 2007 Turkish Earthquake Code to determine the expected damage states of the members. There are three limits for members which are determined based on the three performance levels; Immediate Occupancy, Life Safety and Collapse Prevention.

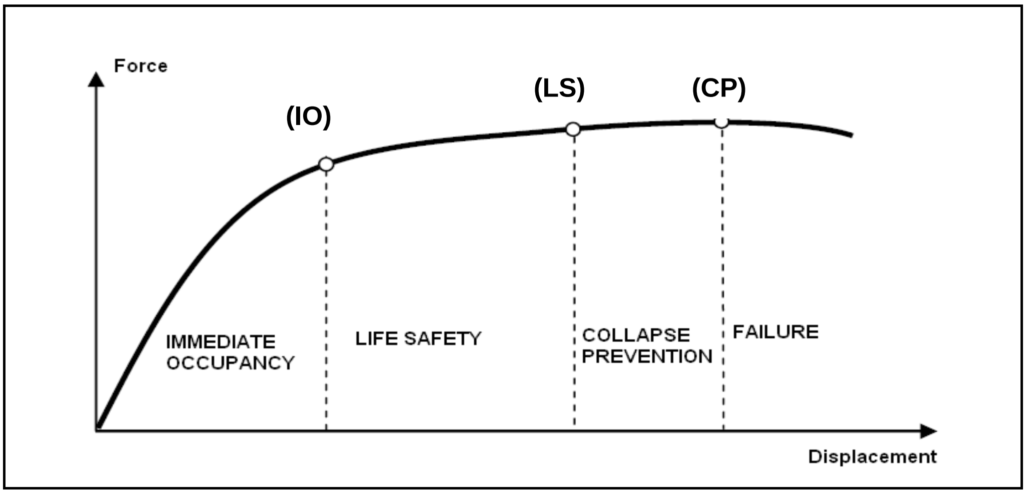

Figure 2.5 Performance Limits of a Member

The performance based limit r values for beams, columns and shear walls that are proposed in the 2007 Turkish Earthquake Code are presented in Tables 2.1, 2.2, and 2.3.

Table 2.1 r_{limit} Values for Reinforced Concrete Beams

Ductile Beams			Damage Limits		
$\frac{\rho-\rho'}{\rho_b}$	**Confinement**	$\frac{V_e}{b_w d f_{ctm}}$	**IO**	**LS**	**CP**
≤ 0.0	YES	≤ 0.65	3	7	10
≤ 0.0	YES	≥ 1.30	2.5	5	8
≥ 0.5	YES	≤ 0.65	3	5	7
≥ 0.5	YES	≥ 1.30	2.5	4	5
≤ 0,0	NO	≤ 0.65	2.5	4	6
≤ 0,0	NO	≥ 1.30	2	3	5
≥ 0.5	NO	≤ 0.65	2	3	5
≥ 0.5	NO	≥ 1.30	1.5	2.5	4
Brittle Beams			1	1	1

Table 2.2 r_{limit} Values for Reinforced Concrete Columns

Ductile Columns			Damage Limits		
$\frac{N_k}{A_c f_{cm}}$	**Confinement**	$\frac{V_e}{b_w d f_{ctm}}$	**IO**	**LS**	**CP**
≤ 0.1	YES	≤ 0.65	3	6	8
≤ 0.1	YES	≥ 1.30	2.5	5	6
≥ 0.4 and ≤ 0.7	YES	≤ 0.65	2	4	6
≥ 0.4 and ≤ 0.7	YES	≥ 1.30	1.5	2.5	3.5
≤ 0,1	NO	≤ 0.65	2	3.5	5
≤ 0,1	NO	≥ 1.30	1.5	2.5	3.5
≥ 0.4 and ≤ 0.7	NO	≤ 0.65	1.5	2	3
≥ 0.4 and ≤ 0.7	NO	≥ 1.30	1	1.5	2
≥ 0.7	-	-	1	1	1
Brittle Columns			1	1	1

Table 2.3 r_{limit} Values for Shear Walls

Ductile Shear Walls	**Damage Limits**		
Confinement	***IO***	***LS***	***CP***
YES	3	6	8
NO	2	4	6

Performance limits are related to five main parameters which are axial load level, confinement, shear force level, volumetric reinforcement ratio and ductility.

Axial load affects the deformation capacity of a member significantly. Figure 2.6 shows that as the axial load increases, deformation capacity of a member decreases. This directly affects the limit r values of a member.

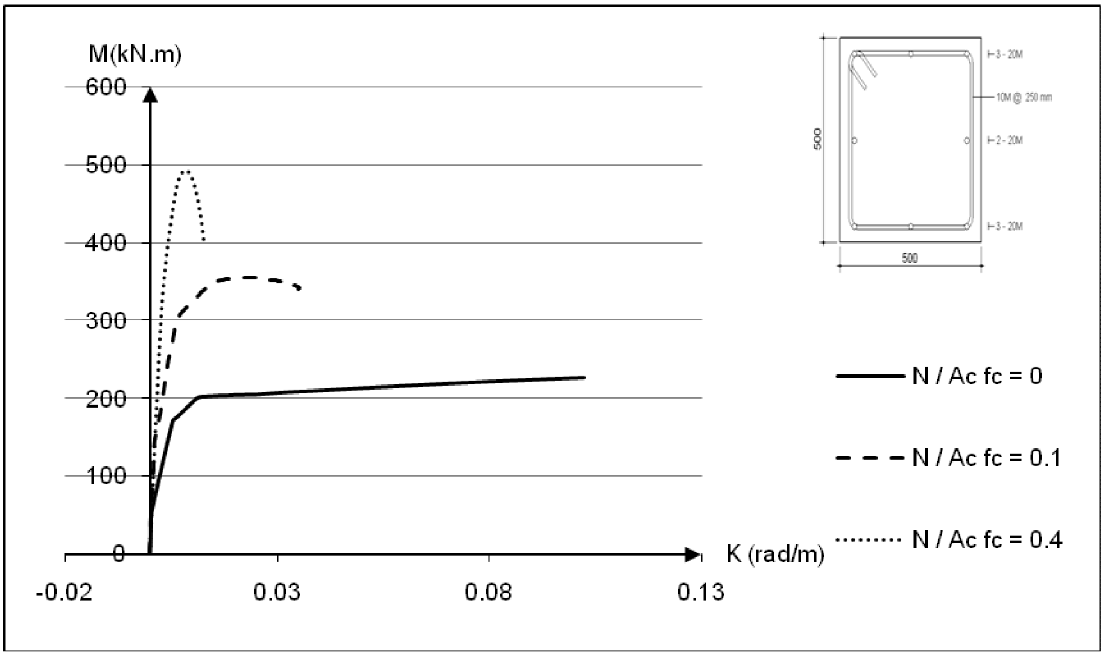

Figure 2.6 Moment Curvature Diagram of a Column for Different Axial Load Levels

In a concrete member, stirrups and ties cover the inner concrete and result in a lateral pressure which increases the strength and strain capacity of the member. This gives an enormous ductility capacity to that member. If at least minimum requirements of the 2007 Turkish Earthquake Code are satisfied for transverse reinforcement, the member is assumed as confined. The effect of confinement is shown in Figure 2.7.

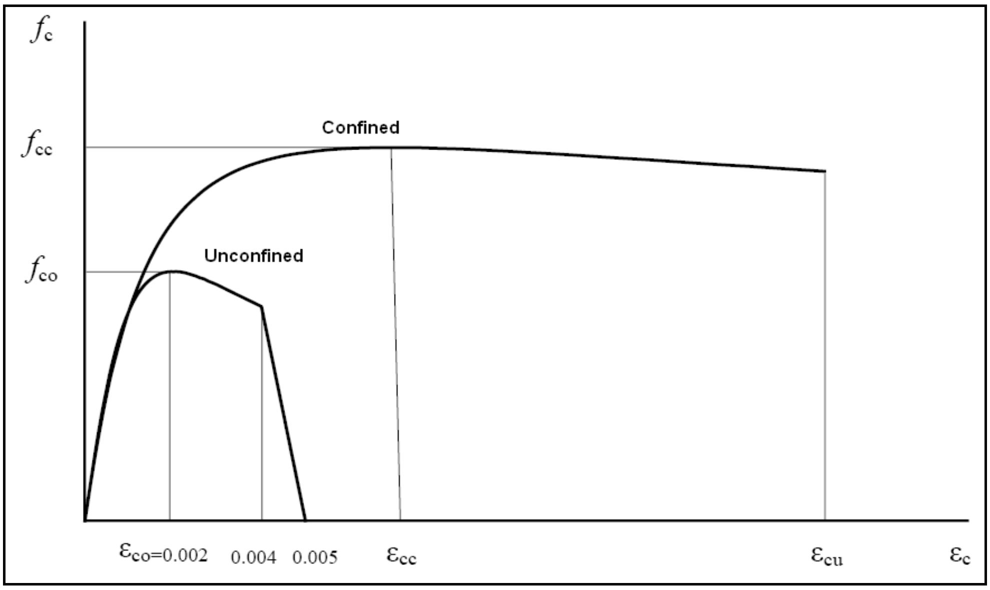

Figure 2.7 Confined and Unconfined Concrete Models (Mander Model) [10]

Brittle members cannot resist large moment, so when moment demand exceeds capacity, member will fail. Therefore, r limits are all 1 for brittle members.

Longitudinal reinforcement ratio helps to check whether a beam is under reinforced or not. If $\frac{\rho - \rho'}{\rho_b} < 1$, then section is under reinforced which means reinforcement in the tension zone yields. This causes ductility. As $\frac{\rho - \rho'}{\rho_b}$ ratio decreases, member becomes more ductile.

In other words, it is seen that each parameter is mainly related to ductility and according to the ductility level of members, r limits increase or decrease in the 2007 Turkish Earthquake Code.

2.1.5 Acceptance and Performance Check

After estimating the limit values for each member by using Tables 2.1, 2.2 and 2.3, r / r_{limit} is calculated for each member.

If $r / r_{limit} < 1$ then the corresponding end of the member is acceptable for the desired performance level

If $r / r_{limit} > 1$ then the corresponding end of the member is not acceptable for the desired performance level.

Each member is checked for its two ends. The worst case, highest r / r_{limit} ratio, is assumed as the r / r_{limit} of that member.

2.2 Nonlinear Assessment Procedure

Nonlinear assessment is the other procedure to estimate the performance level of an existing structure. In this procedure, deformation capacities of each member is calculated and imported to the computer program, nonlinear analysis is performed and plastic deformations are monitored. Each member is assessed according to its deformation compared with limits and finally global performance of a structure is obtained. This procedure is closer to the real situation, because this analysis considers redistribution of forces after the yielding of members. Schematic representation of this procedure is shown Figure 2.8.

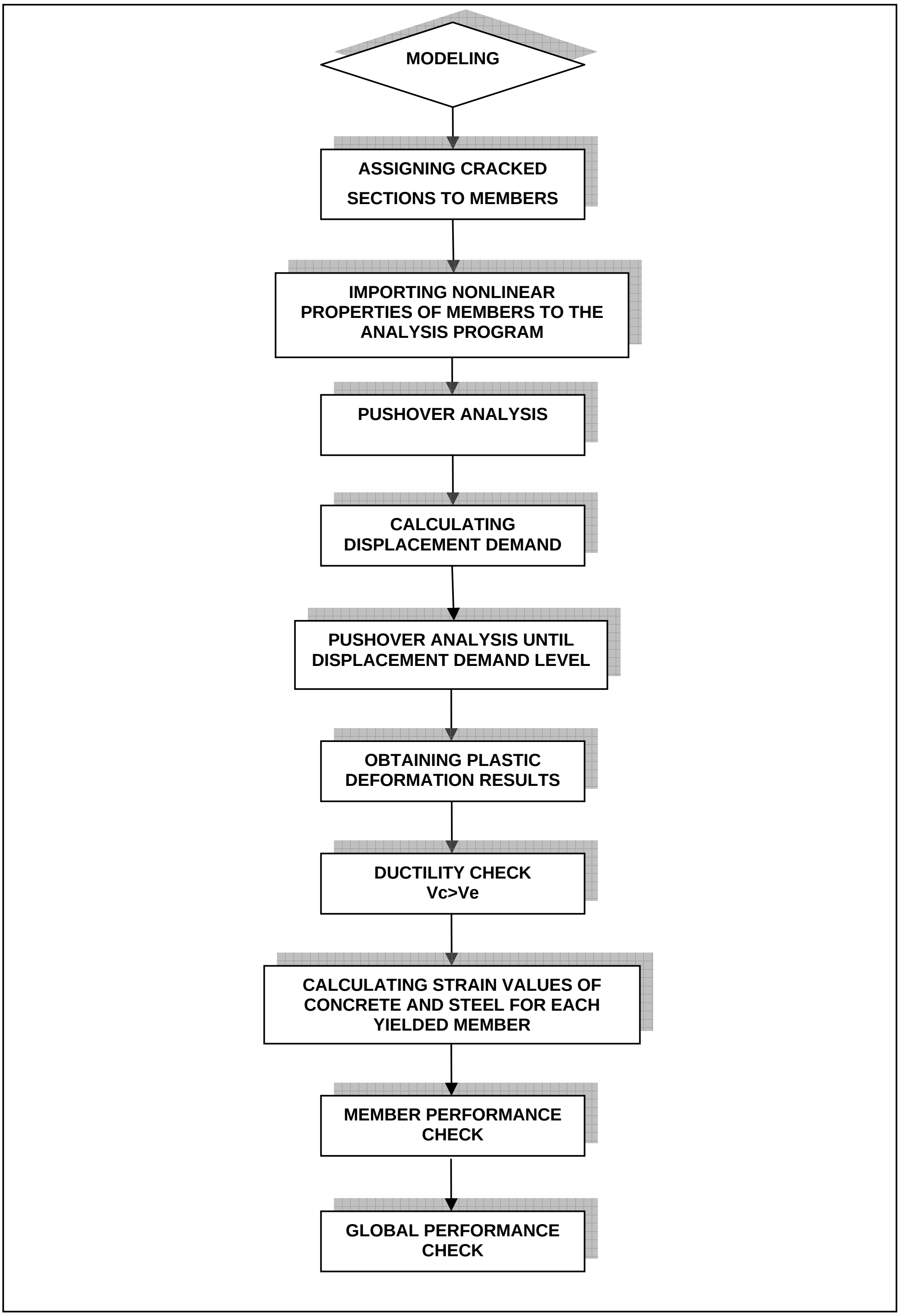

Figure 2.8 Nonlinear Assessment Steps

Similar to linear elastic procedure, this assessment procedure can applied to the buildings which

- are at most 25 m in height from ground level,
- have at most 8 stories,
- have torsional irregularity constant smaller than 1.4 ($\eta_{bi} < 1.4$)
- have at least 70% mass participation ratio for dominant mode.

2.2.1 Modeling

Modeling for nonlinear assessment is nearly the same as the modeling for linear assessment. 3-D model is prepared and each member is modeled according to the project drawings or building field survey results. Sections are assumed cracked for nonlinear analysis as well. Equation (2.1) is valid to estimate the cracked section properties.

Nonlinear parameters are calculated and assigned to each member in the analysis program. Moment rotation diagram is required for beams, columns and shear walls, while 3-D moment interaction diagram is required for columns and shear walls only.

To get an idealized moment rotation diagram, moment curvature diagram of a member should be obtained. Later, it should be bilinearized to estimate the yield and ultimate points. Figure 2.9 shows a bilinearized moment curvature diagram and yield point (M_y, K_y)

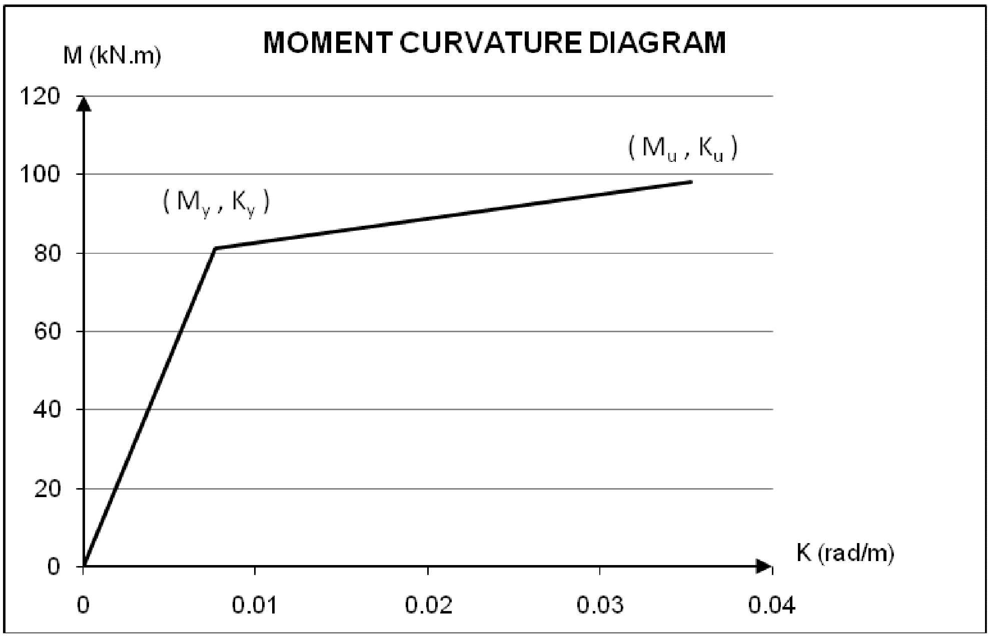

Figure 2.9 Moment Curvature Diagram of a Concrete Member

Yield moment (M_y), yield curvature (K_y), ultimate moment (M_u) and ultimate curvature (K_u) values are taken from the graph. Finally, yield and ultimate rotations can be computed by using parameters obtained from moment curvature graph and the equations below.

$$\theta_y = \frac{\varphi_y \, l_n}{6} \tag{2.21}$$

$$\theta_u = (\varphi_u - \varphi_y)\, L_p + \theta_y \tag{2.22}$$

where θ_y = yield rotation

θ_u = ultimate rotation

φ_y = yield curvature

φ_u = ultimate curvature

L_p = plastic hinge length of section

l_n = net length of the member

Plastic hinge length, L_p can be taken as half of the cross section depth, (L_p= h/2).

The axial load in a column or shear wall changes during nonlinear analysis. Each axial load level corresponds to a different yield point in the interaction diagram. Therefore, 3-D interaction diagram is necessary for columns and shear walls.

To obtain an idealized 3-D moment interaction diagram the following equation proposed by Parme et al (1966) can be used [8].

$$\left(\frac{M_{ux}}{M_{uxo}}\right)^{\left(\frac{\log(0.5)}{\log(\beta)}\right)} + \left(\frac{M_{uy}}{M_{uyo}}\right)^{\left(\frac{\log(0.5)}{\log(\beta)}\right)} = 1 \tag{2.23}$$

where M_{uxo} = uniaxial flexural strength about x-axis

M_{uyo} = uniaxial flexural strength about y-axis

M_{ux} = component of biaxial flexural strength on the x axis at required inclination

M_{uy} = component of biaxial flexural strength on the y axis at required inclination

β = parameter dictating the shape of interaction surface (can be taken as 0.6 or 0.7)

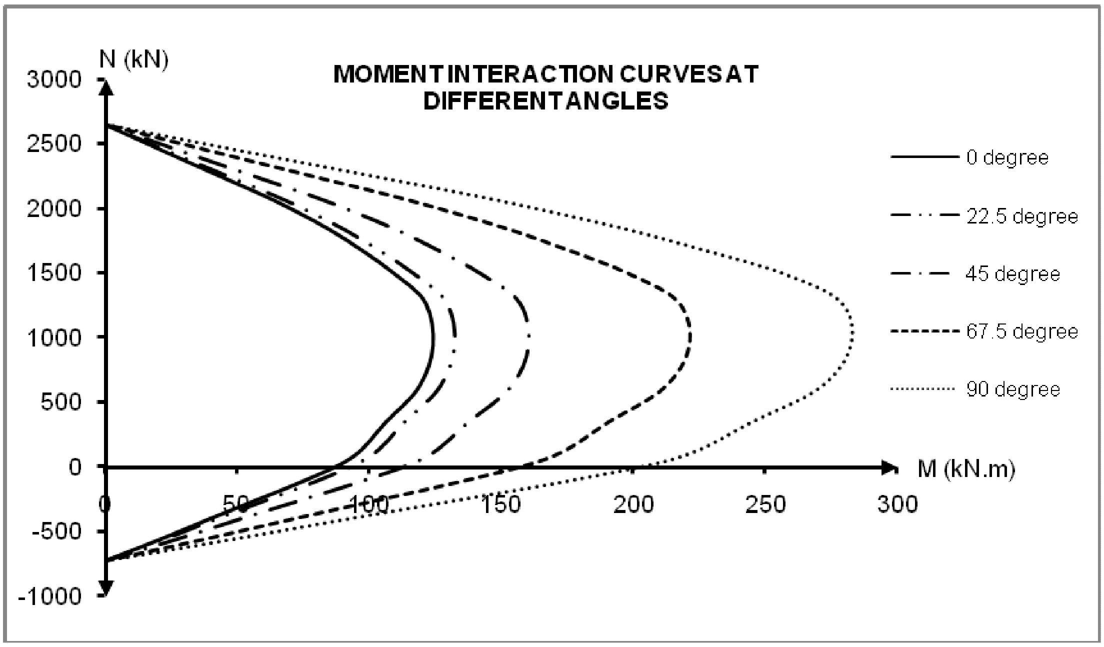

Figure 2.10 Moment Interaction Diagrams of a Column at Different Angles

SAP2000 calculates moment rotation and moment interaction parameters of the members itself. It uses default material models and hinge properties in its memory that cannot be modified by a user. However, to use this feature of the program, all properties of members should be imported to the system in detail to get satisfactory results [7].

2.2.2 Analysis

Nonlinear static analysis is performed in two steps. First, gravity loads (1G+nQ) are applied to the structure. Secondly, the model is pushed under a lateral earthquake load pattern to the desired direction. The structure behaves linearly until the first yield. After the first yield, the building goes into the nonlinear range. When a member reaches its capacity, a plastic hinge forms in that member and the building is pushed again until more member yield. When too many plastic hinges occur and the building behaves as a mechanism (unable to stand) the analysis ends.

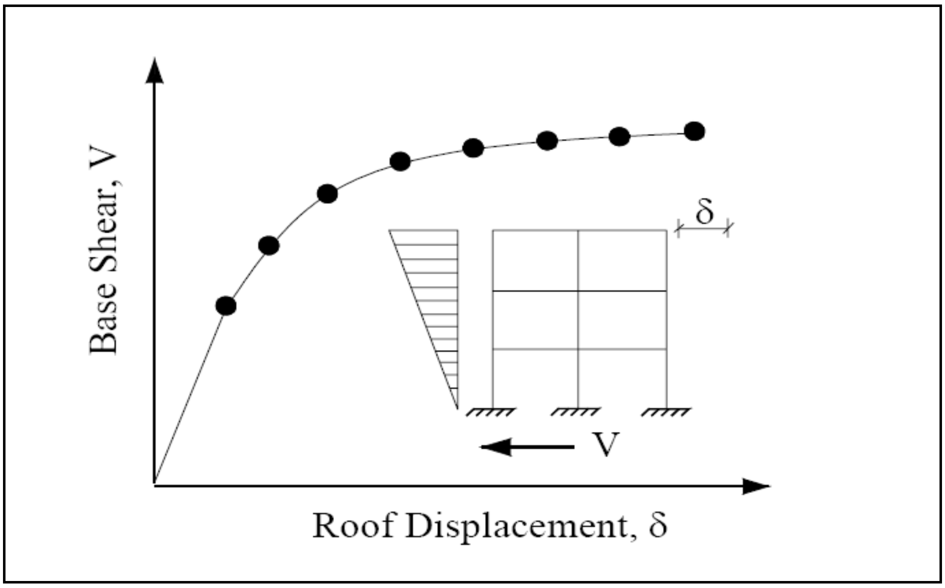

Figure 2.11 A Typical Pushover Curve [14]

After nonlinear analysis, a pushover curve is obtained which shows the base shear and roof displacement relationship (Figure 2.11).

2.2.3 Calculating Displacement Demand of the Building

During an earthquake, ground acceleration results in deformation of the building. When maximum deformation occurs, maximum forces take place in the members. In the nonlinear assessment procedure, based on pushover analysis, target displacement demand of the buildings is calculated under the desired earthquake loadings represented generally by the response spectra.

In the 2007 Turkish Earthquake Code, there is a procedure to calculate the displacement demand of the structure. According to this procedure, pushover curve is converted to the modal capacity diagram by using the equations given below.

$$a_1^{(i)} = \frac{V_{x1}^{(i)}}{M_{x1}} \tag{2.24}$$

$$d_1^{(i)} = \frac{u_{xN1}^{(i)}}{\Phi_{xN1}\Gamma_{x1}} \tag{2.25}$$

where $a_1^{(i)}$ = modal acceleration at the end of i^{th} step

$d_1^{(i)}$ = modal displacement at the end of i^{th} step

$V_{x1}^{(i)}$ = base shear in x direction after i^{th} step

M_{x1} = modal mass for first mode in x direction

$u_{xN1}^{(i)}$ = first mode roof displacement in x direction at the end of i^{th} step

Φ_{xN1} = first mode shape amplitude of the top storey in x direction

Γ_{x1} = participation factor of the first mode

Spectrum diagram is also converted to the modal capacity diagrams as follows.

$$S_{de} = \frac{S_{ae}}{(\omega^{(1)})^2} \tag{2.26}$$

where S_{de} = elastic spectral displacement of the corresponding period at first step of pushover analysis

S_{ae} = elastic spectral acceleration of the corresponding period at first step of pushover analysis

$\omega^{(1)}$ = frequency of the corresponding period

Inelastic spectral displacement is calculated by the equation below.

$$S_{di} = C_R \, S_{de} \tag{2.27}$$

where S_{di} = nonlinear spectral displacement of the corresponding mode

C_R = spectral displacement ratio of the corresponding mode

If the corresponding period (T) is greater than characteristic period of acceleration spectrum (T_B) ($T \geq T_B$), inelastic spectral displacement is equal to elastic spectral displacement or C_R = 1. (Figure 2.12)

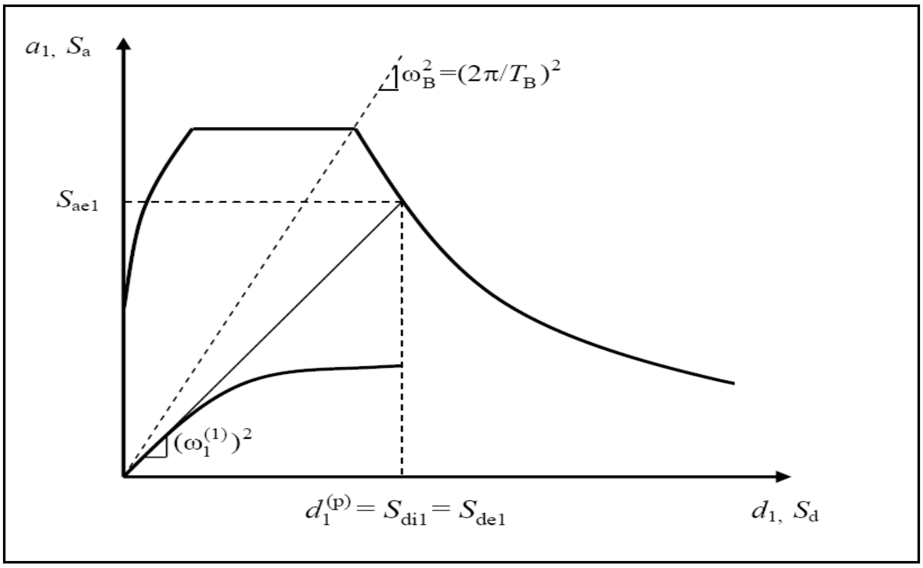

Figure 2.12 Calculating Displacement Demand When ($T \geq T_B$) [1]

If $T<T_B$, C_R is calculated by iterating the following equations.

$$C_{R1} = \frac{1 + (R_{y1} - 1)\, T_B / T_1^{(1)}}{R_{y1}} \geq 1 \tag{2.28}$$

$$R_{y1} = \frac{S_{ae1}}{a_{y1}} \tag{2.29}$$

where R_{y1} is resistance reduction factor.

Before iteration, modal capacity curve is bi-linearized as shown in Figure 2.13. Slope of the first line is taken equal to the square of the angular frequency of first mode, $(\omega_1^{(1)})^2$.

In the first iteration, assuming C_{R1} as 1, equivalent yield point coordinates (a^0_{y1}) determined by equal area rule (area under and over the capacity curves should be equal) as shown in Figure 2.14. Then, using Equation (2.28) and (2.29), new C_{R1} is calculated and by using equal are rule a_{y1}, R_{y1} and C_{R1} are determined again. If two consecutive results are nearly equal to each other, iteration is stopped.

As a result inelastic spectral displacement demand is calculated by the following equation.

$$S_{di1} = d_1^{(p)} \tag{2.30}$$

After calculating inelastic spectral displacement demand, it is converted to inelastic displacement demand.

$$u_{xN1}^{(i)} = \Phi_{xN1}\Gamma_{x1}d_1^{(p)} \tag{2.31}$$

where $u_{xN1}^{(i)}$ = target displacement

$d_1^{(p)}$ = inelastic spectral displacement (S_{di})

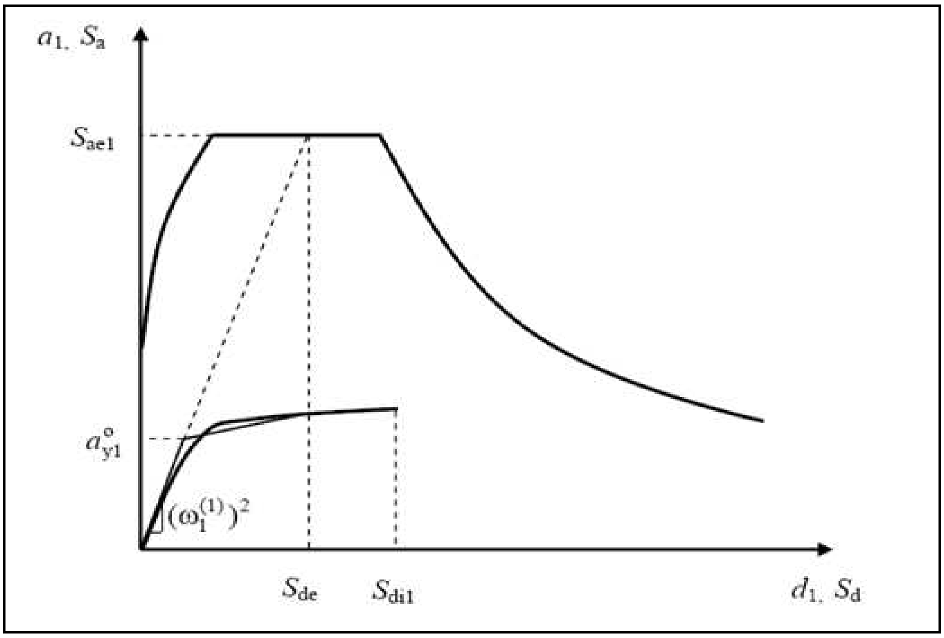

Figure 2.13 Bi-linearization of Modal Capacity Curve [1]

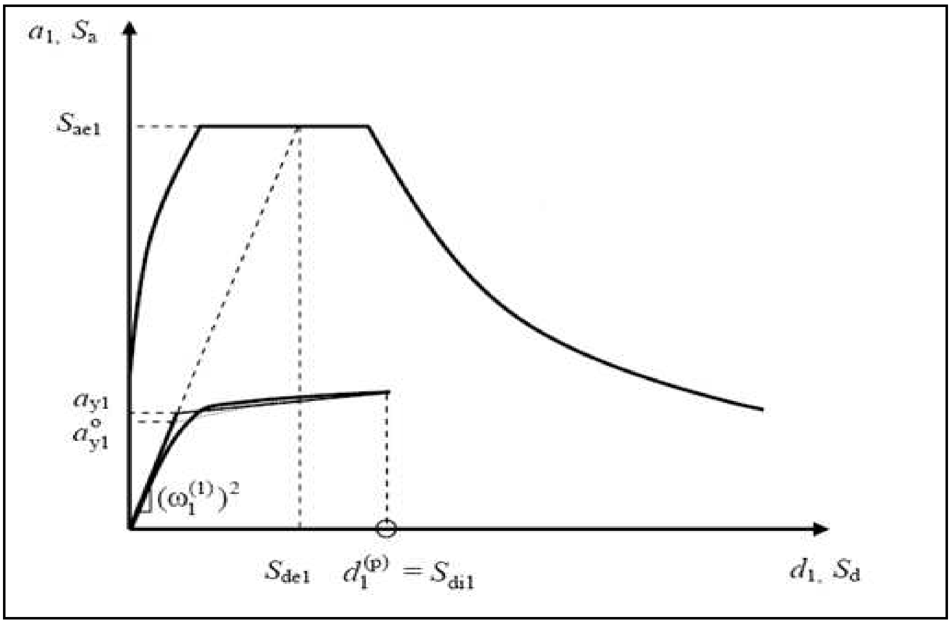

Figure 2.14 Calculating Displacement Demand When ($T < T_B$) [1]

2.2.4 Performance Level Check

Unlike other codes, the 2007 Turkish Earthquake Code checks the performance limits not in terms of rotation but in terms of strain. Therefore, recorded rotations for target displacement need to be converted into curvatures first, then to strains through cross sectional analysis.

Plastic curvatures are obtained from rotations by using Equations (2.32) and (2.33)

$$\varphi_p = \frac{\theta_p}{L_p} \tag{2.32}$$

$$\varphi_t = \varphi_p + \varphi_y \tag{2.33}$$

By carrying out cross sectional analyses, strains at the level of concrete and steel are computed for the total curvature calculated by Equation (2.33). These strains are compared with the performance based limiting values in the code.

According to the 2007 Turkish Earthquake Code, the concrete and steel strain limits are defined as follows.

For Immediate Occupancy Performance Level (IO)

$$(\varepsilon_{cg})_{MN} = 0.0035 \;\; ; \;\; (\varepsilon_s)_{MN} = 0.010 \tag{2.34}$$

where $(\varepsilon_{cg})_{MN}$ = outermost fiber strain of concrete

$(\varepsilon_s)_{MN}$ = steel strain

For Life Safety Performance Level (LS)

$$(\varepsilon_{cg})_{GV} = 0.0035 + 0.01\,(\rho_s / \rho_{sm}) \leq 0.0135 \;\; ; \;\; (\varepsilon_s)_{GV} = 0.040 \tag{2.35}$$

where $(\varepsilon_{cg})_{GV}$ = concrete clear cover strain

$(\varepsilon_s)_{GV}$ = steel strain

ρ_s = volumetric ratio of transverse reinforcement

ρ_{sm} = minimum design volumetric ratio of transverse reinforcement

For Collapse Prevention Performance Level (CP)

$$(\varepsilon_{cg})_{GC} = 0.004 + 0.014\,(\rho_s / \rho_{sm}) \leq 0.018 \;\; ; \;\; (\varepsilon_s)_{GV} = 0.060 \tag{2.36}$$

where $(\varepsilon_{cg})_{GC}$ = concrete clear cover strain

$(\varepsilon_s)_{GC}$ = steel strain

ρ_s = volumetric ratio of transverse reinforcement

ρ_{sm} = minimum design volumetric ratio of transverse reinforcement

2.2.5 Acceptance and Performance Check

$\varepsilon / \varepsilon_{limit}$ is calculated for each member for the assessment.

If $\varepsilon / \varepsilon_{limit} < 1$, then the corresponding end of the member is acceptable for the target performance level.

If $\varepsilon / \varepsilon_{limit} > 1$, then the corresponding end of the member is not acceptable for the target performance level.

Each member is checked for its two ends. The worst of two $\varepsilon / \varepsilon_{limit}$ ratios is assumed as the $\varepsilon / \varepsilon_{limit}$ of that member.

2.3 Estimating Performance of the Building

Both assessment procedures provide the performance levels of each member of a building. However, main aim of these assessments is to estimate the performance of the whole building. For this purpose, the 2007 Turkish Earthquake Code gives some criteria for Immediate Occupancy, Life Safety and Collapse Prevention Levels. According to code, performance limits are described as follows.

Immediate Occupancy (IO) Performance Level

At any storey, in the considered direction, at most 10% of the beams are allowed to go beyond *Immediate Occupancy Limit* and all other members should be below *Immediate Occupancy Limit.* After retrofitting brittle members if any, building performance is considered as *Immediate Occupancy Level*.

Life Safety (LS) Performance Level

After retrofitting brittle members if any, building is considered as Life Safety Performance Level when the following conditions are satisfied.

a) At any storey, in the considered direction, except secondary (which are not in the considered direction) beams, at most 30% of the beams is allowed to go beyond *Life Safety Performance Limit.*
b) At any storey, columns in *Collapse Prevention Level* should carry at most 20% of the total storey shear. At the top storey, columns in *Collapse Prevention Level* may carry 40% of the total storey shear.
c) All other members should be in *Immediate Occupancy Level* or *Life Safety Level.* However, at any storey, columns that exceed *Immediate Occupancy Limit* from top and bottom ends together should carry at most 30% of total storey shear.

Collapse Prevention (CP) Performance Level

a) At any storey, in the considered direction, except secondary (which are not in the considered direction) beams, at most 30% of the beams is allowed to go beyond *Collapse Prevention Limit*.
b) All other members should be in *Immediate Occupancy Level*, *Life Safety Level* or *Collapse Prevention Level.* However, at any storey, columns that

exceed *Immediate Occupancy Limit* from top and bottom ends together should carry at most 30% of total storey shear.

According to the 2007 Turkish Earthquake Code, buildings are assessed for different levels of earthquake forces that are determined according to their type and usage. The design acceleration spectra given in the code assumes that it represents an earthquake effect that has a probability of 10% in 50 years. This spectrum is multiplied by 0.5 and 1.5 to represent the earthquake effects with a 10% and 2% respectively in 50 years. As given in the Table 2.4, for both linear and nonlinear assessments, more than one analysis with different design acceleration spectra can be performed.

Table 2.4 Required Performance Levels for Buildings Considering Type and Usage

BUILDING TYPE AND USAGE	**Probability of occurrence of an earthquake**		
	50% in 50 years	*10% in 50 years*	*2% in 50 years*
Buildings to be utilized after earthquake	**-**	**IO**	**CP**
Intensively and long term occupied buildings	**-**	**IO**	**CP**
Intensively and short term occupied buildings	**IO**	**LS**	**-**
Buildings containing hazardous materials	**-**	**IO**	**CP**
Other buildings	**-**	**LS**	**-**

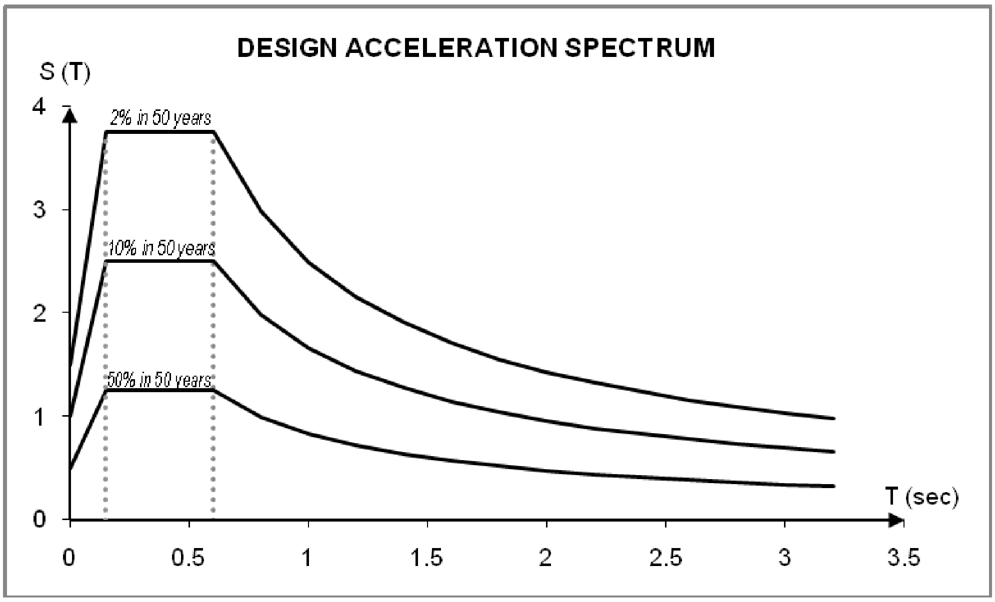

Figure 2.15 Design Acceleration Spectrum According to Probability of Occurrence of an Earthquake

In addition to the assessment based on member damage levels, the buildings are also checked according to their displacements using Table 2.5.

Table 2.5 Storey Displacement Limits

Storey Displacement Limits	Performance Level		
	Immediate Occupancy	Life Safety	Collapse Prevention
δ_{ji} / h_{ji}	0.01	0.03	0.04

where δ_{ji} = displacement between top and bottom ends of j^{th} column at i^{th} storey

h_{ji} = height of that column.

CHAPTER III

CASE STUDY 1: ASSESMENT OF A RESIDENTIAL BUILDING IN BAKIRKÖY, İSTANBUL

In this case study, a concrete residential building is analyzed in detail by performing both linear and nonlinear assessment according to the 2007 Turkish Earthquake Code. The building is located in Bakırköy district of İstanbul, had been identified as a vulnerable building by a study initiated under a project managed by the project implementation unit of Priministry. *Life Safety* is the target performance level for this case study. SAP2000 and some other utility programs (Response 2000, excel macros etc.) are used for this assessment.

The building has 5 storeys each of which is 2.9 m in height. It has a moment resisting frame system consisting of beams and columns. Framing of the building is irregular in plan where there are 8 axes in X-direction and 7 axes in Y-direction. Floor plan is same for each storey and has an area of 290.7 m^2. Slab thicknesses are 10 cm. A typical floor plan is shown in Figure 3.1. Also, storey masses, location of mass centers and mass moment of inertias are given in Table 3.1.

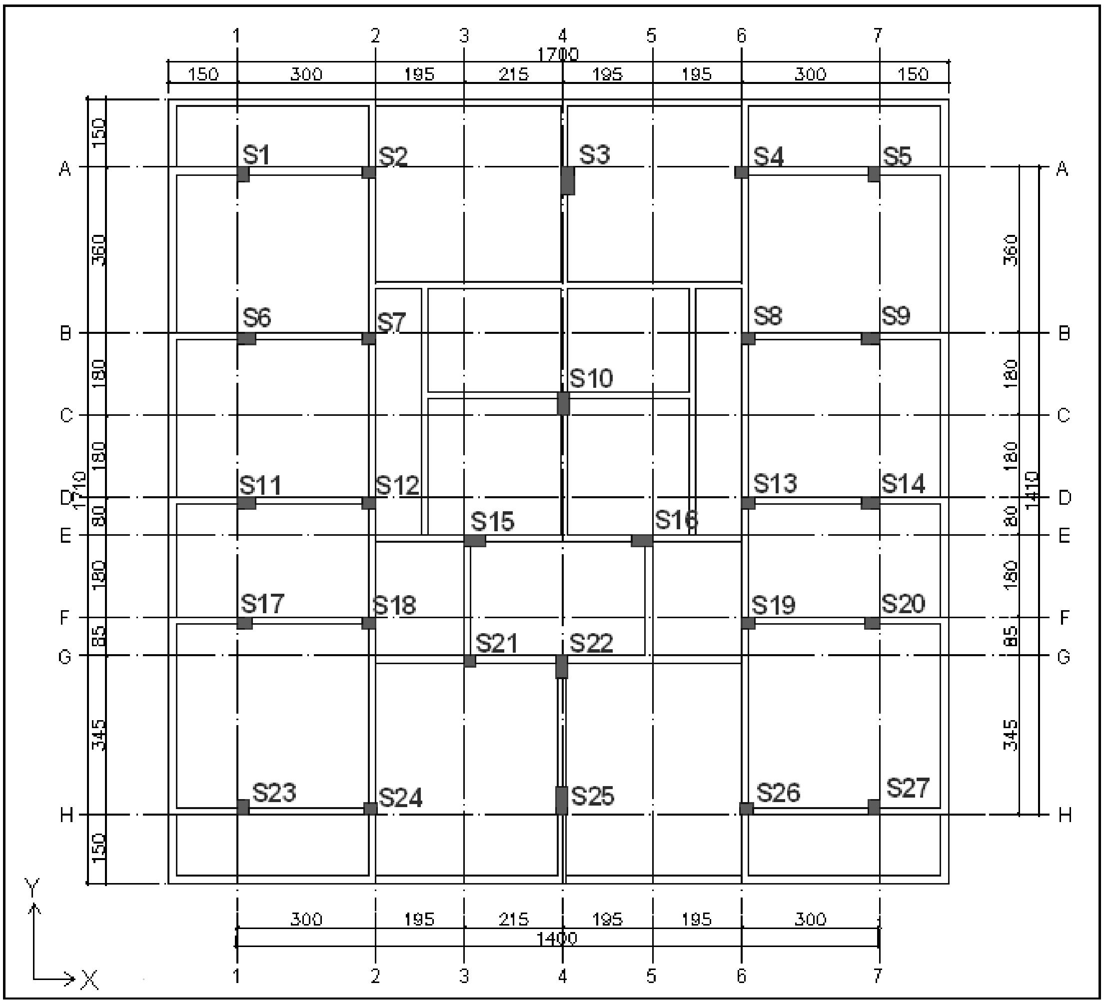

Figure 3.1 Typical Floor Plan

Table 3.1 Storey Masses, Mass Center Coordinates and Mass Moment of Inertias

Storey	Mass (t)	Mass Center		Mass Moment Of Inertia ($t.m^2$)
		X (m)	Y (m)	
1	189.27	8.5	8,4	9170.12
2	189.27	8.5	8.4	9170.12
3	189.27	8.5	8.4	9170.12
4	189.27	8.5	8.4	9170.12
5	117.79	8.5	8,4	5706.83

Cross sections of the columns, building properties, material properties and reinforcement details are given in Table 3.2 and Table 3.3 respectively. All beams have a 15x40 cm cross section.

Table 3.2 Cross Sections of the Columns

Frame	Cross Section	Frame	Cross Section	Frame	Cross Section	Frame	Cross Section	Frame	Cross Section
1S1	30x25	**2S1**	30x25	**3S1**	30x25	**4S1**	30x25	**5S1**	30x25
1S2	25x50	**2S2**	25x50	**3S2**	25x30	**4S2**	25x30	**5S2**	25x30
1S3	65x30	**2S3**	60x30	**3S3**	60x30	**4S3**	60x30	**5S3**	60x30
1S4	25x50	**2S4**	25x45	**3S4**	25x30	**4S4**	25x30	**5S4**	25x30
1S5	30x25	**2S5**	30x25	**3S5**	30x25	**4S5**	30x25	**5S5**	30x25
1S6	25x50	**2S6**	25x50	**3S6**	25x40	**4S6**	25x40	**5S6**	25x40
1S7	25x45	**2S7**	25x45	**3S7**	25x30	**4S7**	25x30	**5S7**	25x30
1S8	25x45	**2S8**	25x40	**3S8**	25x30	**4S8**	25x30	**5S8**	25x30
1S9	25x50	**2S9**	25x40	**3S9**	25x40	**4S9**	25x40	**5S9**	25x40
1S10	60x30	**2S10**	50x25	**3S10**	50x25	**4S10**	25x50	**5S10**	50x25
1S11	25x50	**2S11**	25x50	**3S11**	25x40	**4S11**	25x40	**5S11**	25x40
1S12	25x40	**2S12**	25x40	**3S12**	25x30	**4S12**	25x30	**5S12**	25x30
1S13	25x40	**2S13**	25x40	**3S13**	25x30	**4S13**	25x30	**5S13**	25x30
1S14	25x50	**2S14**	25x50	**3S14**	25x40	**4S14**	25x40	**5S14**	25x40
1S15	25x45	**2S15**	25x45	**3S15**	25x45	**4S15**	25x45	**5S15**	25x45
1S16	25x45	**2S16**	25x45	**3S16**	25x45	**4S16**	25x45	**5S16**	25x45
1S17	25x50	**2S17**	25x50	**3S17**	25x30	**4S17**	25x30	**5S17**	25x30
1S18	25x45	**2S18**	25x45	**3S18**	25x30	**4S18**	25x30	**5S18**	25x30
1S19	25x45	**2S19**	25x45	**3S19**	25x30	**4S19**	25x30	**5S19**	25x30
1S20	25x50	**2S20**	25x45	**3S20**	25x30	**4S20**	25x30	**5S20**	25x30
1S21	25x25	**2S21**	25x25	**3S21**	25x25	**4S21**	25x25	**5S21**	25x25
1S22	65x25	**2S22**	50x25	**3S22**	50x25	**4S22**	50x25	**5S22**	50x25
1S23	35x25	**2S23**	35x25	**3S23**	30x25	**4S23**	30x25	**5S23**	30x25
1S24	50x25	**2S24**	50x25	**3S24**	30x25	**4S24**	30x25	**5S24**	30x25
1S25	60x30	**2S25**	60x25	**3S25**	60x25	**4S25**	60x25	**5S25**	60x25
1S26	25x50	**2S26**	25x50	**3S26**	25x30	**4S26**	25x30	**5S26**	25x30
1S27	35x25	**2S27**	30x25	**3S27**	30x25	**4S27**	30x25	**5S27**	30x25

Table 3.3 Building and Material Properties, Reinforcement Details

BUILDING PROPERTIES	CONSTRUCTION DATE	1967
	# OF STOREYS	5
	EARTHQUAKE ZONE	1
	SOIL CLASS	Z3
MATERIAL PROPERTIES	CONCRETE STRENGTH (MPa)	11
	LONG. REINFORCEMENT STRENGTH (MPa)	300
	TRANS. REINFORCEMENT STRENGTH (MPa)	400
REINFORCEMENT DETAILS	% OF COLUMN LONG. REINFORCEMENT	0.9 %
	% OF BEAM TOP REINFORCEMENT	0.6 %
	% OF BEAM BOTTOM REINFORCEMENT	0.4 %
	COLUMN STIRRUPS	Ø6 / 20
	BEAM STIRRUPS	Ø6 / 25

The concrete compressive strength determined from the tests of core samples taken from the building is used to calculate tensile strength (f_{ctk}) and modulus of elasticity (E_c) by using the following formulae given in TS-500 [5].

$$f_{ctk} = 0.35\sqrt{f_{ck}} \tag{3.1}$$

$$f_{ctk} = 0.35\sqrt{11} = 1.161 \text{ MPa}$$

$$E_c = 3250\sqrt{f_{ck}} + 14000 \tag{3.2}$$

$$E_c = 3250\sqrt{11} + 14000 = 24800 \text{ MPa}$$

In the following sections, detailed application of the assessment procedures are illustrated for the column 1S18 and the beam K113 only for +X direction to show steps of the procedures for both linear and nonlinear assessments.

In the further analysis, positive moment (tension at the bottom) and tensile force is noted positive (+) while negative moment and compressive force is considered as negative (-).

3.1 Linear Assessment Procedure

The case study building was modeled and analyzed using SAP2000 according to the requirements of the code as explained in the previous chapter. In this section, assessment of the building based on the procedure given in the code for the linear elastic assessment is carried out. Each step of the procedure is briefly described and details of the steps are only provided for the selected members.

3.1.1 Modeling and Analysis

The case study building is modeled in SAP2000. 3-D view of the model is shown in Figure 3.2.

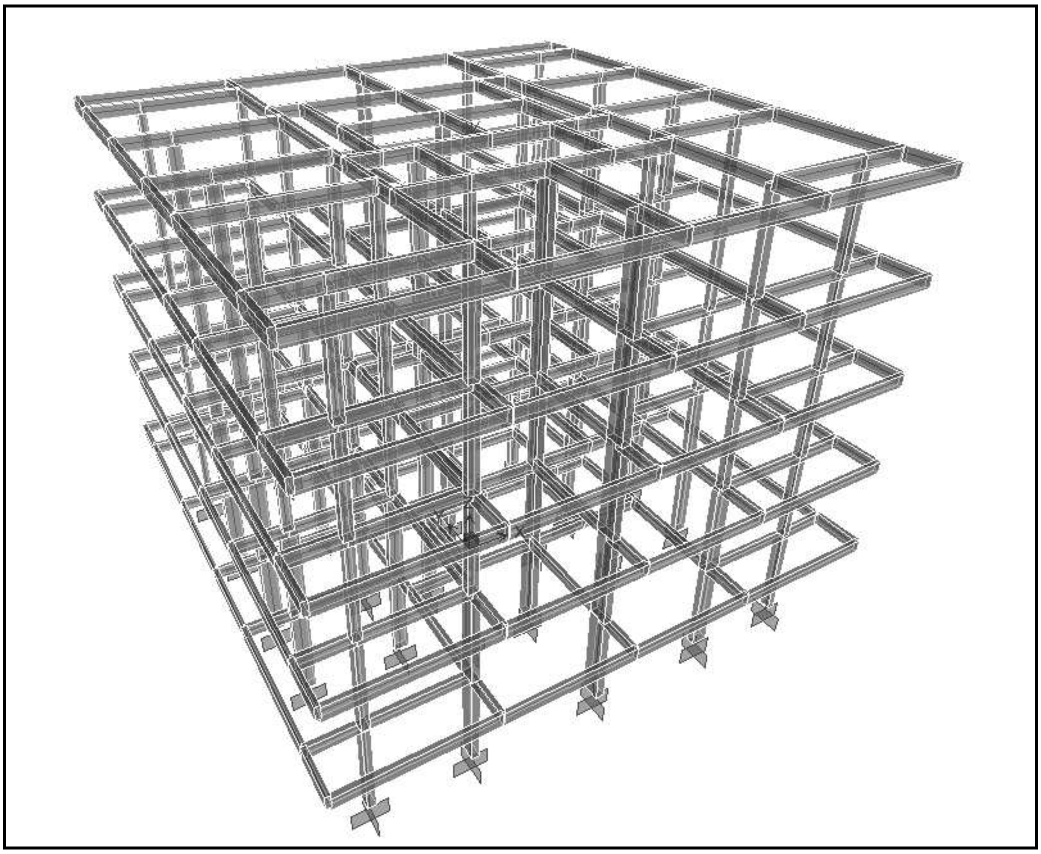

Figure 3.2 3-D Model of the Building in SAP2000

Rigidity of each member is reduced by using Equation (2.1). First, a gravity load analysis is performed and then rigidity of each member is reduced. This analysis results in terms of the axial load are used to calculate the cracked rigidity for the column 1S18 as shown in the following steps.

N_d = -221.89 kN, A_c = 250 x 450 = 112500 $\Rightarrow$ $N_d / (A_c\, f_{cm}) = 0.18$

Since the axial load ratio is 0.18 and the reduction coefficients are only given in the code for the axial load ratios of 0.4 and 0.1, linear iteration is required to calculate cracked rigidity $(EI)_c$. Therefore, the amount of reduction is calculated as

(0.8-0.4) / (0.4-0.1) x (0.18 – 0.10) + 0.4 = 0.51

As a result, for the column 1S18 N_d = -221.89 kN ➔ $(EI)_c = 0.51\ (EI)_0$ meaning that uncracked rigidity is reduced by multiplying it with 0.51.

The design spectrum representing the earthquake loads to be considered is calculated using Equations 2.2 and 2.3 as follows.

Earthquake Zone = 1 ➔ $A_0 = 0.40$

Soil Type = Z3 ➔ $T_A = 0.15$, $T_B = 0.60$

For this building, the period was calculated from dynamic analysis performed in SAP2000 as 1.27 sec and 1.52 sec for X and Y directions respectively.

The design spectrum is then used to calculate the base shear force in the two principal directions of the building. The spectrum curve and corresponding S(T) values for T_x and T_y are calculated as 1.37 and 1.19, respectively as shown in Figure 3.4.

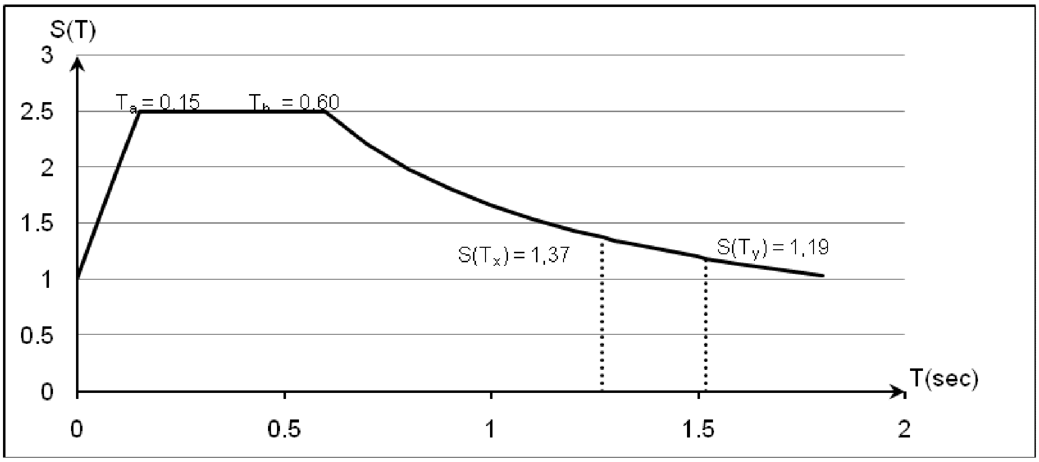

Figure 3.3 Estimating S(T) Values from Response Spectrum Diagram

Finally, total base shear force for X and Y directions are,

V_x = 0.85 x 8582.29 x 0.4 x 1 x 1.37 / 1 = 3997.64 kN

V_y = 0.85 x 8582.29 x 0.4 x 1 x 1.19 / 1 = 3443.21 kN

These forces are distributed to each storey proportional to their masses and heights as given in Tables 3.4 and 3.5.

Table 3.4 Earthquake Loads Distributed to Each Storey (+X Direction)

# of Storey	w_i	H_i	$w_i H_i$	$\frac{w_i H_i}{\sum w_i H_i}$	V_i
1	1856.70	2.9	5384.44	0.08	293.46
2	1856.70	5.8	10768,88	0.15	586.92
3	1856.70	8.7	16153.32	0.23	880.38
4	1856.70	11.6	21537.76	0.30	1173.83
5	1155.48	14,5	16754.46	0.24	1063.05

Table 3.5 Earthquake Loads Distributed to Each Storey (+Y Direction)

# of Storey	w_i	H_i	$w_i H_i$	$\frac{w_i H_i}{\sum w_i H_i}$	V_i
1	1856.70	2,9	5384.44	0.08	252.75
2	1856.70	5.8	10768.88	0.15	505.52
3	1856.70	8.7	16153.32	0.23	758.28
4	1856.70	11.6	21537.76	0.30	1011.04
5	1155.48	14.5	16754.46	0.24	915.62

The building is analyzed for 5 load combinations including gravity (1G+0.3Q) load, +X earthquake (1Ex) load, -X earthquake (-1Ex) load, +Y earthquake (1Ey) load and -Y Earthquake (-1Ey) load. The internal forces calculated for K113 and 1S18 from gravity and +X earthquake load combinations are presented in Tables 3.6 and 3.7, respectively.

Table 3.6 Analysis Results of Beam K113

Member	Load Combination	N(kN)	V(kN)	M (kN.m)
K113 (i end)	1G+0,3Q	-	-14.102	-6.02
K113 (j end)		-	-6.019	-12.22
K113 (i end)	1Ex	-	228.633	341.46
K113 (j end)		-	228.633	-344.44

Table 3.7 Analysis Results of Column 1S18

Member	Load Combination	N(kN)	V(kN)	M (kN.m)
1S18 (top)	1G+0,3Q	-221.89	-1.301	-2.45
1S18 (bottom)		-221.89	-1.301	1.33
1S18 (top)	1Ex	-185.11	-205.57	-129.61
1S18 (bottom)		-185.11	-205.57	466.52

3.1.2 Calculation of Member Capacities

Calculation of Beam Capacity

As can be seen in Table 3.3, the top and bottom reinforcement ratios of the beam K113 are 0.6 percent and 0.4 percent respectively. The positive and negative moment-curvature diagrams for this beam are shown in Figure 3.4.

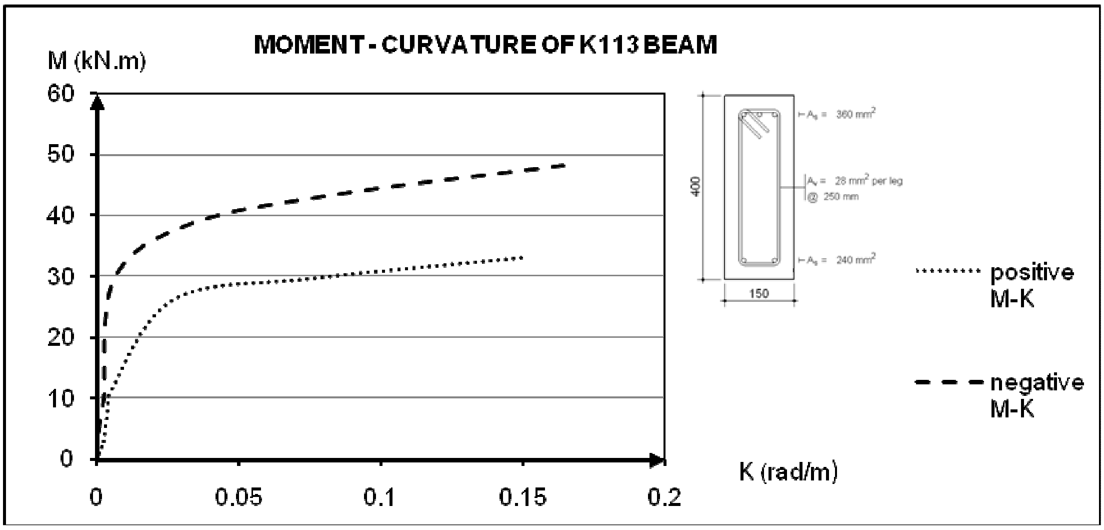

Figure 3.4 Positive and Negative Moment Curvature Graphs of K113 Beam

The positive and negative moment capacities, as can be extracted from Figure 3.4, are

$+M_p$ = 33.06 kN.m , $-M_p$ = -48.15 kN.m

For +X direction the residual moment capacity is calculated by using Equation (2.5) as

For end i,

M_p = 33.06 kN.m, M_d = -6.02 kN.m

M_c = 33.06 - (-6.02) = 39.08 kN.m

For end j,

M_p = -48.15 kN.m, M_d = -12.22 kN.m

M_c = -48.15 - (-12.22) = -35.93 kN.m

Calculation of Column Capacity

The interaction diagram of 1S18 is drawn first as shown in Figure 3.5. It is worth noting that longitudinal reinforcement ratio is 0.9 percent as given in Table 3.3.

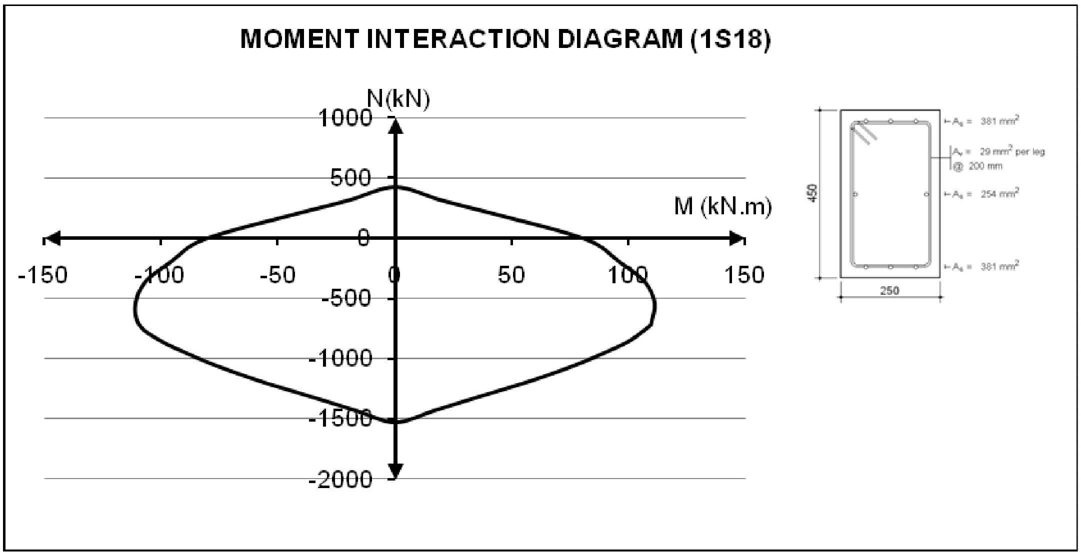

Figure 3.5 Moment Interaction Diagram of 1S18 Column

The capacity of this column is calculated based on the internal forces obtained from the analysis according to the procedure explained earlier. For the upper and lower ends of column 1S18 the axial loads and bending moments obtained from the analysis are as follows,

Upper end,

N_D = -221.89 kN, M_D = -2.45 kN.m

N_E = -185.11 kN, M_E = -129.61 kN.m

$N_D + N_E$ = -407 kN , $M_D + M_E$ = -132.06 kN.m

Lower end,

N_D = -221.89 kN, M_D = 1.33 kN.m

N_E = -185.11 kN, M_E = 466.52 kN.m

$N_D + N_E$ = -407 kN , $M_D + M_E$ = 467.85 kN.m

These values are marked in the interaction diagram as illustrated in Figure 3.6.

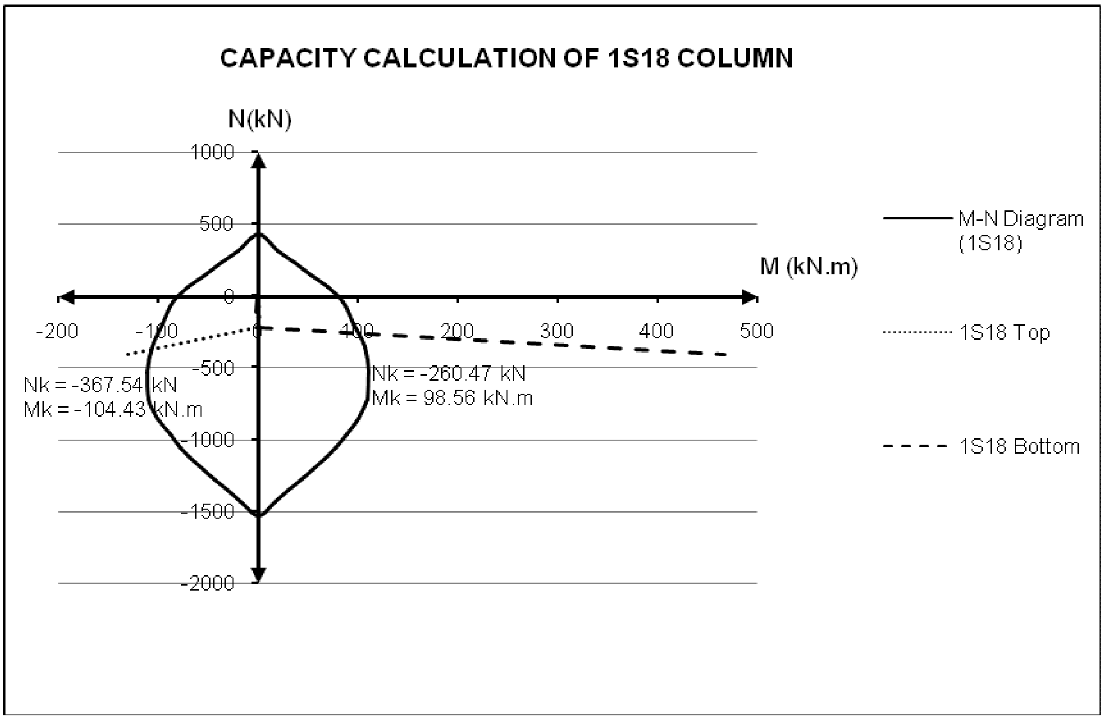

Figure 3.6 Capacity Calculation of Column 1S18

From Figure 3.6, it is seen that the intersection of the lines (N_D, M_D) - ($N_D + N_E$, $M_D + M_E$) with the interaction curve gives (N_K, M_K) the capacity of column 1S18 for the top and bottom ends. These capacities are determined as

$N_{K\ (top)}$ = -367.54 kN, $M_{K\ (top)}$ = -104.43 kN.m

$N_{K\ (bottom)}$ = -260.47 kN, $M_{K\ (bottom)}$ = 98.56 kN.m

3.1.3 Ductility Check

This check is necessary to understand whether the members are ductile or not.

Ductility Check for the Beam

Shear demand of the beam is calculated using Equation (2.6).

For K113, the shear forces for the simply supported beam are calculated as, V_{dy} is -16.67 kN and 16.67 kN for i and j ends, respectively (Figure 3.8).

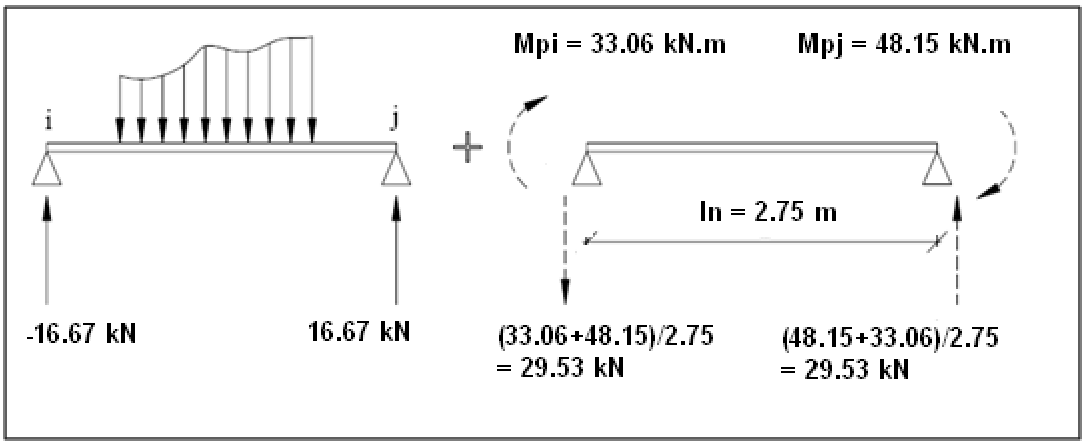

Figure 3.7 Calculation of V_e for Beam K113

The flexural shear capacity for ends i and j is calculates as,

$V_{e(i)}$ = -16.67 + 29.53 = 12.86 kN

$V_{e(j)}$ = 16.67 + 29.53 = 46.20 kN

The shear capacity is calculated by Equation (2.7), (2.8) and (2.9) as follows.

V_{cr} = (0.65 x 1.161 x 150 x 370) / 1000

V_{cr} = 41.88 kN

$V_w = ((2 \times \pi \times 6^2 / 4) \times 400 \times 370 / 250) / 1000$

$V_w = 33.46$ kN

$V_r = 0.8 \times 41.88 + 33.46 = 66.96$ kN.m

Since $V_r > V_{e\,(i)}$ and $V_r > V_{e\,(j)}$, the beam is ductile.

Ductility Check for the Column

To determine whether the column will fail under ductile or brittle behavior, its ductility is checked by using Equation (2.10).

CBCR value for the column is computed from Equation (2.9) as,

(104.03 + 95.08) / (33.06 + 48.15) = 2.45 > 1.20 (OK)

Since the columns are stronger than the beams, hinges are expected to occur in beams first and thus the column capacities need to be modified. Therefore, by using Equation (2.10) and (2.11)

$$M_{ü} = \frac{129.61}{129.61 + 344.63}(33.06 + 48.15) = 22.19 \text{ kN.m}$$

CBCR cannot be applied to the lower end of 1S18 since it is a ground storey column.

$M_a = M_{pa} = 98.56$ kN.m

The flexural shear capacity is calculated using Equation (2.8) as.

$V_e = (22.19 + 98.56) / 2.75 = 43.91$ kN

The nominal shear capacity is calculated by using Equations (2.7), (2.8) and (2.9) as

$V_{cr} = (0.65 \times 1.161 \times 250 \times 420) \times (1+ 0.07 \times (221.89 \times 1000) / (250 \times 450)) / 1000$

$V_{cr} = 90.18$ kN

$V_w = ((2 \times \pi \times 6^2 / 4) \times 400 \times 420 / 200) / 1000$

$V_w = 47.48$ kN

$V_r = 0.8 \times 90.18 + 47.48 = 119.62$ kN.

Since $V_r > V_e$, column 1S18 is ductile.

3.1.4 Performance of Members

After estimating the capacity and checking the ductility of members, r and r_{limit} values of the members can be calculated.

Performance Assessment for Beam K113

Equation (2.16) is used to compute the r value of beams.

For end i and j of K113 the demand capacity ratios are,

$r_i = M_e / M_c = 341.46 / (33{,}06-(-9{,}02)) = 8.74$

$r_j = M_e / M_c = -344.44 / (-48{,}15-(-12{,}22)) = 9.59$

Based on the detailing and cross sectional properties such as volumetric reinforcement ratio, shear force and confinement of the beam the r_{limit} values are determined as follows.

<u>For end i of beam K113</u>

$$\frac{\rho - \rho'}{\rho_b} = 0.23$$

Confinement ➔ No

$$\frac{V_e}{b_w d f_{ctm}} = 12.86 \times 1000 / (150 \times 400 \times 1.161) = 0.185$$

For Life Safety Performance Level, r_{limit} = 4 from the corresponding table.

For end j of beam K113

$$\frac{\rho - \rho'}{\rho_b} = -0.23$$

Confinement ➔ No

$$\frac{V_e}{b_w d f_{ctm}} = 46.20 \times 1000 / (150 \times 400 \times 1.161) = 0.663$$

For Life Safety Performance Level, r_{limit} is calculated by two interpolations.

4 - (4-3) / (0.5-0) x (0.23-0) = 3.54

3.54 + (3-4) / (1.30-0.65) x (0.663 – 0.65) = 3.52

Finally the ratios of r’s for each end of the beam are;

$(r / r_{limit})_i = 8.74 / 4 = 2.19$

$(r / r_{limit})_j = 9.59 / 3.52 = 2.72$

This means that since $(r / r_{limit}) > 1$ for both ends, K113 does not satisfy Life Safety Performance Level.

Performance Assessment for Column 1S18

r value of the column is calculated for both ends as follows.

<u>Upper end of 1S18</u>

r = -129.61 / (-104.40-(-2.45)) = 1.27

<u>Lower end of 1S18</u>

r = 466.52 / (98.56– (1.33)) = 4.80

Since r_{limit} of columns depends on axial load level, shear force and confinement, these properties for column 1S18 are used to determine it.

<u>For upper end of 1S18</u>

$N_k / (A_c\ f_c)$ = 367.54 x 1000 / (250 x 450 x 11) = 0.30

Confinement ➔ No

$$\frac{V_e}{b_w d f_{ctm}} = 43.91 \times 1000 / (250 \times 420 \times 1.161) = 0.36$$

r_{limit} = 3.5 - (3.5 – 2) / (0.4-0.1) x (0.30 – 0.10) = 2.5

Finally r / r_{limit} for upper end of 1S18 = 1.27 / 2.5 = 0.51

<u>For lower end of 1S18</u>

$N_k / (A_c\ f_c)$ = 260.47 x 1000 / (250 x 450 x 11) = 0.21

Confinement ➔ No

$\frac{V_e}{b_w d f_{ctm}}$ = 43.91 x1000 / (250 x 420 x 1.161) = 0.36

r_{limit} = 3.5 - (3.5 – 2) / (0.4-0.1) x (0.21 – 0.10) = 2.95

Finally r / r_{limit} for lower end of 1S18 = 4.80 / 2.95 = 1.63

These results indicate that although (r / r_{limit}) < 1 for upper end, 1S18 does not satisfy Life Safety Performance Level since r / r_{limit} for lower end of 1S18 is 1.63.

The performance of each column and beam in the case study building were evaluated applying the steps illustrated for the example column and the beam. Figures A.1-A.4 and A.5-A.8 of Appendix A show the r / r_{limit} values for the columns and beams at each storey, respectively. These results are summarized in Tables 3.8 - 3.11 and Tables 3.12 - 3.15, respectively.

Table 3.8 Summary of Linear Assessment Results for Columns (+X Direction)

+X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($r/r_{limit}<1$)	Number of Columns	2	7	3	12	27
	Column Percentage	**7.41%**	**28.00%**	**11.11%**	**44.44%**	**100.00%**
	Shear Force Percentage	**4.58%**	**38.30%**	**16.42%**	**50.24%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($r/r_{limit}>1$)	Number of Columns	25	18	24	15	0
	Column Percentage	**92.59%**	**72.00%**	**88.89%**	**55.56%**	**0.00%**
	Shear Force Percentage	**95.42%**	**61.70%**	**83.58%**	**49.76%**	**0.00%**

Table 3.9 Summary of Linear Assessment Results for Columns (-X Direction)

- X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($r/r_{limit}<1$)	Number of Columns	2	7	4	10	27
	Column Percentage	**7.41%**	**28.00%**	**14.81%**	**37.04%**	**100.00%**
	Shear Force Percentage	**4.41%**	**36.92%**	**21.92%**	**43.04%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($r/r_{limit}>1$)	Number of Columns	25	18	23	17	0
	Column Percentage	**92.59%**	**72.00%**	**85.19%**	**62.96%**	**0.00%**
	Shear Force Percentage	**95.59%**	**63.08%**	**78.08%**	**56.96%**	**0.00%**

Table 3.10 Summary of Linear Assessment Results for Columns (+Y Direction)

+Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit<1)	Number of Columns	5	12	12	16	23
	Column Percentage	**18.52%**	**44.44%**	**44.44%**	**59.26%**	**85.19%**
	Shear Force Percentage	**3.98%**	**46.55%**	**51.38%**	**68.25%**	**83.20%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit>1)	Number of Columns	22	15	15	11	4
	Column Percentage	**81.48%**	**55.56%**	**55.56%**	**40.74%**	**14.81%**
	Shear Force Percentage	**96.02%**	**53.45%**	**48.62%**	**31.75%**	**16.80%**

Table 3.11 Summary of Linear Assessment Results for Columns (-Y Direction)

-Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit<1)	Number of Columns	5	11	13	15	23
	Column Percentage	**18.52%**	**40.74%**	**48.15%**	**55.56%**	**85.19%**
	Shear Force Percentage	**2.78%**	**37.66%**	**60.20%**	**61.29%**	**82.16%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit>1)	Number of Columns	22	16	14	12	4
	Column Percentage	**81.48%**	**59,26%**	**51.85%**	**44.44%**	**14.81%**
	Shear Force Percentage	**97.22%**	**62.34%**	**39.80%**	**38.71%**	**17.84%**

Table 3.12 Summary of Linear Assessment Results for Beams (+X Direction).

+X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit<1)	Number of Beams	3	3	3	7	18
	Beam Percentage	**16.67%**	**16.67%**	**16.67%**	**38.89%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit<1)	Number of Beams	15	15	15	11	0
	Beam Percentage	**83.33%**	**83.33%**	**83.33%**	**61.11%**	**0.00%**

Table 3.13 Summary of Linear Assessment Results for Beams (-X Direction)

-X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit<1)	Number of Beams	2	2	2	7	18
	Beam Percentage	**11.11%**	**11.11%**	**11.11%**	**38.89%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit<1)	Number of Beams	16	16	16	11	0
	Beam Percentage	**88.89%**	**88.89%**	**88.89%**	**61.11%**	**0.00%**

Table 3.14 Summary of Linear Assessment Results for Beams (+Y Direction)

+Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit<1)	Number of Beams	0	0	0	6	12
	Beam Percentage	**0.00%**	**0.00%**	**0.00%**	**46.15%**	**92.31%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit<1)	Number of Beams	13	13	13	7	1
	Beam Percentage	**100.00%**	**100.00%**	**100.00%**	**53.85%**	**7.69%**

Table 3.15 Summary of Linear Assessment Results for Beams (-Y Direction).

-Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit<1)	Number of Beams	0	0	0	3	11
	Beam Percentage	**0.00%**	**0.00%**	**0.00%**	**23.08%**	**84.62%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit<1)	Number of Beams	13	13	13	10	2
	Beam Percentage	**100.00%**	**100.00%**	**100.00%**	**76.92%**	**15.38%**

Examination of these tables reveals that this building does not satisfy Life Safety Performance Level of the code for linear elastic assessment because more than 30% of the beams go beyond *Life Safety* performance limit for each direction and more than 20% of shear force is carried by the columns that exceeds the required performance limit.

3.2 Nonlinear Assessment

In order to investigate the differences between the two assessment procedures of the code, the case study building was assessed using nonlinear assessment procedure based on pushover analysis.

3.2.1 Modeling and Calculation of Member Capacities

Steps involved in modeling for nonlinear analysis is the same as linear elastic analysis. Again, SAP2000 program is used to construct the 3-D model of the building based on cracked section properties. Unlike linear elastic analysis, since the plastic rotations in members are important in this assessment, moment-rotation relationships and moment-axial load interactions are calculated and imported to the program.

The moment-rotation relationships for column 1S18 and beam K113 were constructed from their moment-curvature graphs that are obtained from sectional analyses. The moment-rotation relationship of beam K113 presented in Figure 3.9, is obtained from its moment-curvature diagram (Figure 3.8) as follows.

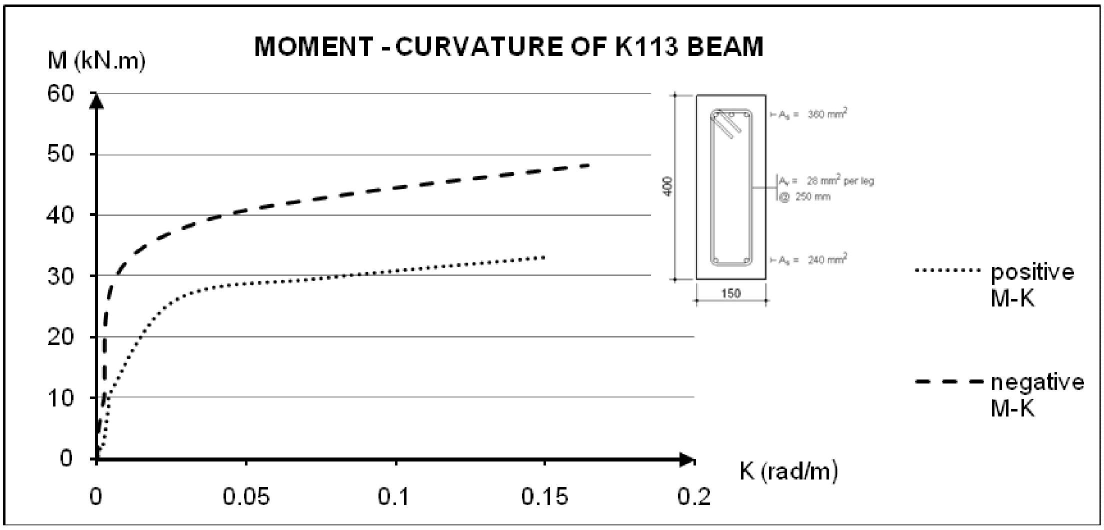

Figure 3.8 Moment Curvature Diagram of K113 Beam

From Figure 3.8, following parameters are obtained.

For positive bending,

M_y = 26.23 kN.m, K_y = 0.0269 rad/m

M_u = 33.06 kN.m, K_u = 0.1493 rad/m

For positive bending,

M_y = 29.53 kN.m, K_y = 0.0058 rad/m

M_u = 48.15 kN.m, K_u = 0.1642 rad/m

The length and plastic hinge length for this beam are l_n = 2.75 m and L_p = 0.40/ 2 = 0.20 m respectively.

Therefore, yield and ultimate rotation values are computed using Equation (2.18) and (2.19),

<u>For positive bending</u>

θ_y = (0.0269 x 2.75) / 6 = 0.0123 rad

θ_u = (0.1493 – 0.0269) x 0.20 + 0.0123 = 0.0368 rad

<u>For negative bending</u>

θ_y = (0.0058 x 2.75) / 6 = 0.0027 rad

θ_u = (0.1642 – 0.0058) x 0.20 + 0.0027 = 0.0344 rad

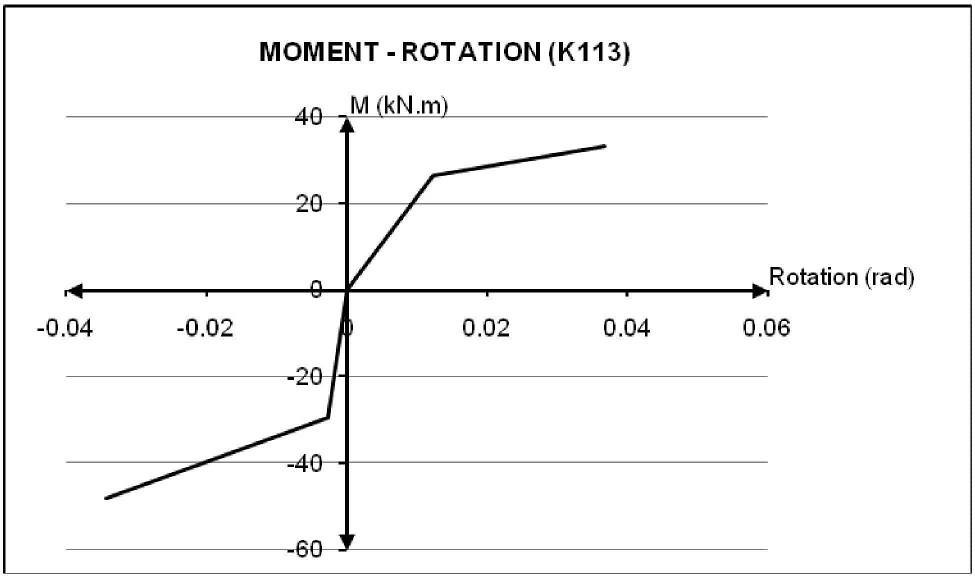

Figure 3.9 Positive and Negative Moment Rotation Diagram of K113 Beam

The moment-rotation relationship (Figure 3.11) for column 1S8 is obtained from the moment-curvature diagram presented in Figure 3.10.

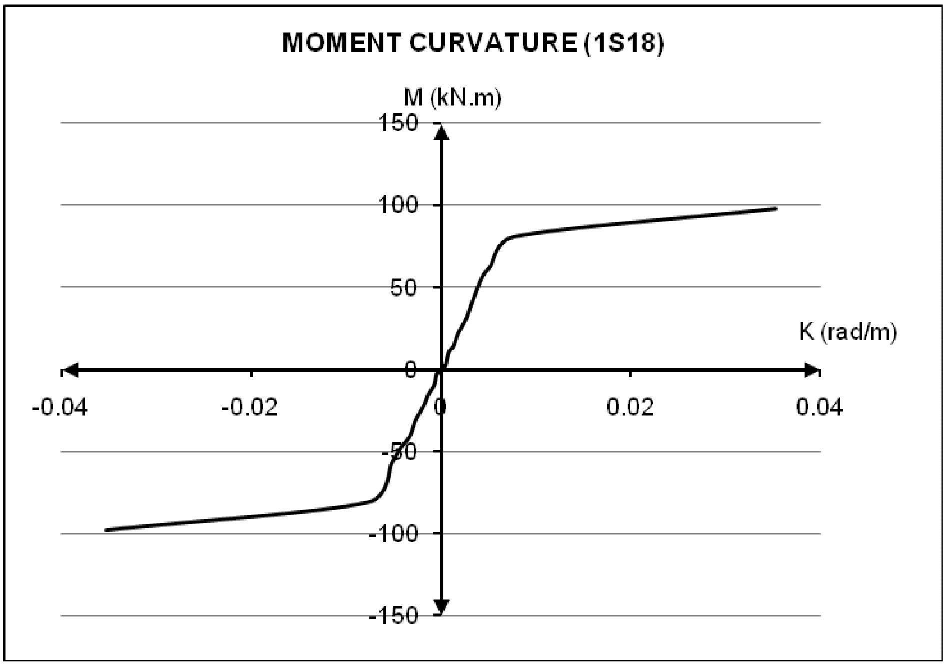

Figure 3.10 Moment Curvature Diagram of 1S18 Column

The properties of the moment-curvature diagram are obtained as

M_y = 81.01 kN.m, K_y = 0.0077 rad/m,

M_u = 98.03 kN.m and K_u = 0.0353 rad/m

for both upper and lower ends of 1S18.

The length and plastic hinge length of the columns are l_n = 2.75 m , L_p = 0.45 / 2 = 0.225 m.

Therefore, the yield and ultimate rotations are computed as

θ_y = (0.0077 x 2.75) / 6 = 0.0035 rad

θ_u = (0.0353 – 0.0077) x 0.225 + 0.0035 = 0.0097 rad

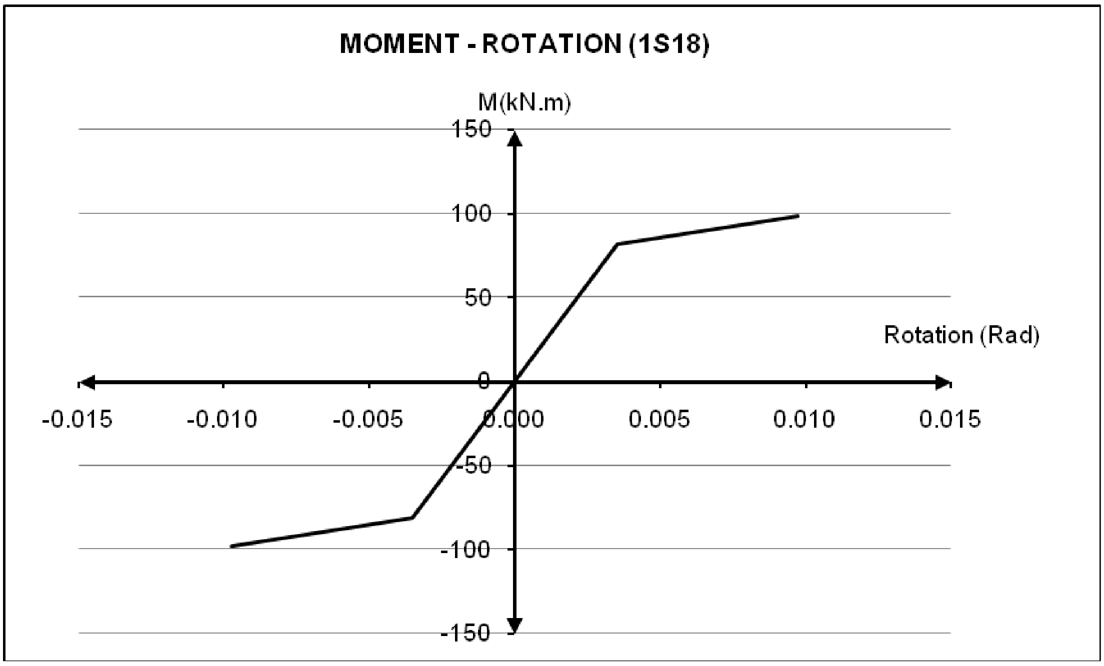

Figure 3.11 Moment Rotation Diagram of 1S18 Column

The 3-D interaction diagram of column 1S18 is determined by using Equation 2.23 and taking β =0.7. The capacities in X and Y directions are calculated as M_{ux0} = 79.20 kN.m and M_{uy0} = 40.25 kN.m.

The discrete values of the interaction diagram are given in Table 3.16.

Table 3.16 3-D Moment Interaction Data of 1S18

P (kN.m)	M_{ux0}	M_{uy0}	M at Ψ=0°	M at Ψ=22.5°	M at Ψ=45 °	M at Ψ=67.5 °	M at Ψ=90 °
424.59	0.00	0.00	0.00	0.00	0.00	0.00	0.00
317.07	19.36	10.03	19.36	16.24	12.52	10.61	10.03
230.76	36.66	18.95	36.66	30.73	23.67	20.04	18.95
147.86	53.10	27.35	53.10	44.46	34.21	28.94	27.35
63.41	69.68	35.78	69.68	58.28	44.78	37.87	35.78
-42.59	85.59	44.13	85.59	71.71	55.19	46.70	44.13
-197.51	95.99	50.09	95.99	80.81	62.48	52.98	50.09
-354.99	106.52	56.09	106.52	90.00	69.84	59.31	56.09
-536.31	111.05	58.68	111.05	93.96	73.01	62.04	58.68
-721.63	109.51	56.98	109.51	92.10	71.13	60.28	56.98
-853.58	100.28	51.39	100.28	83.82	64.36	54.40	51.39
-961.75	88.55	45.62	88.55	74.17	57.06	48.28	45.62
-1054.05	77.30	39.91	77.30	64.79	49.89	42.23	39.91
-1134.12	65.98	34.17	65.98	55.37	42.68	36.14	34.17
-1207.71	54.72	28.33	54.72	45.91	35.39	29.97	28.33
-1276.55	42.95	22.17	42.95	35.98	27.70	23.45	22.17
-1349.39	30.56	15.75	30.56	25.58	19.69	16.66	15.75
-1432.27	16.80	8.63	16.80	14.04	10.79	9.13	8.63
-1531.10	0.00	0.00	0.00	0.00	0.00	0.00	0.00

3.2.2 Analysis

After modeling the building and preparing nonlinear properties of the members, pushover analysis based on the lateral load pattern described in the code is performed. As it is mentioned, two analyses are conducted. First, the model is analyzed under gravity loads (1G+0.3Q). Later, it is pushed by an earthquake load pattern until the model behaves as mechanism. After the analysis, pushover curves are obtained from the computer program as shown in Figure 3.12.

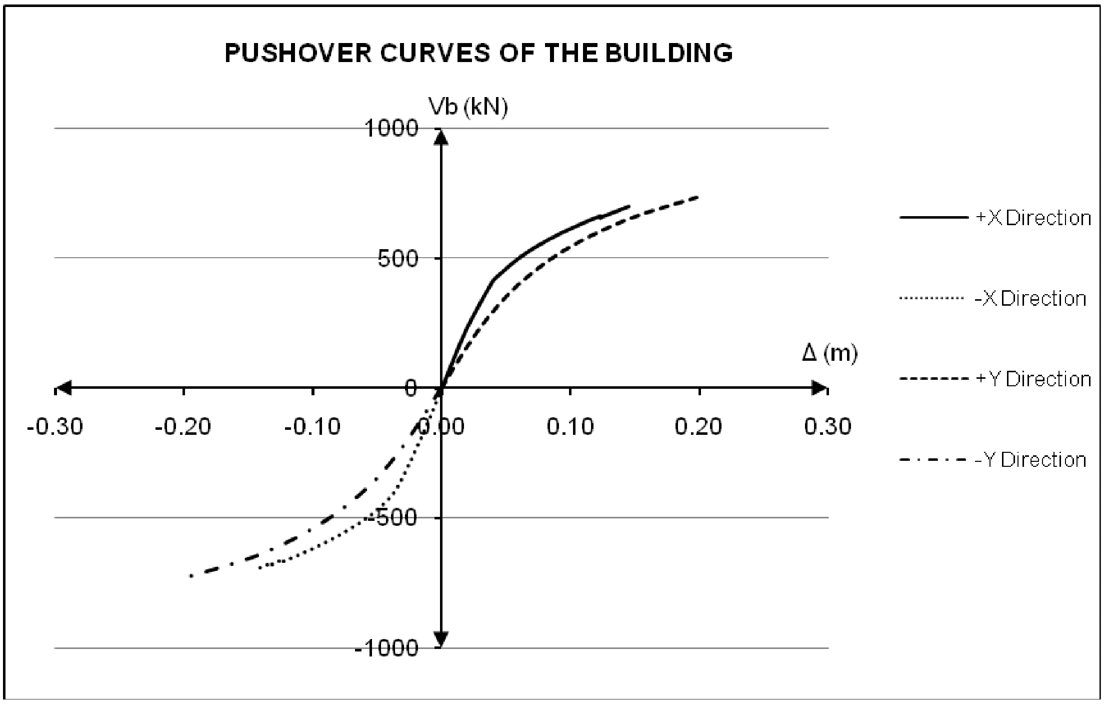

Figure 3.12 Pushover (Capacity) Curves of the Model for Each Direction

From the capacity curves, performance points are calculated for each direction under the effect of the code design spectrum. The plastic rotations at each end of the members are extracted from the results of the pushover analysis corresponding to the performance point. These rotation values are converted to strains which are compared with the limit values given in the code.

The capacity curve and the capacity diagram obtained using Equations (2.21). (2.22) and (2.23) are displayed in Figure 3.53 for +X direction.

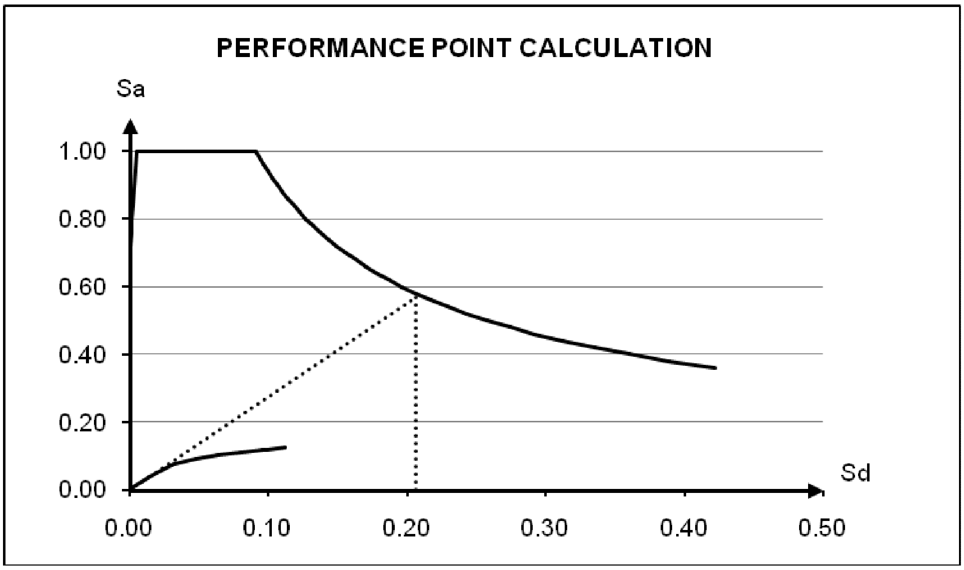

Figure 3.13 Performance point calculation of the building for +X direction

Period is greater than characteristic period of acceleration spectrum (T_B) therefore, inelastic spectral displacement is equal to elastic spectral displacement.

$S_{di} = S_{de} = 0.206$ m

$u_{xN1} = 0.0546 \times 23.67 \times 0.206 = 0.26$ m

Although the calculated target displacement is 0.26 m. Figure 3.53 shows that the ultimate displacement capacity of the building is smaller than the target displacement demand. Therefore, the building is assumed to reach the maximum displacement capacity under the presumed earthquake effect. Finally, the target displacement is

$S_{di1} = S_{de1} = 0.112$ m

Using Equation (2.31)

u_{xN1} = 0.0546 x 23.67 x 0.112= 0.145 m

The plastic rotations and corresponding strains are calculated corresponding to the roof displacement of 0.145 m.

3.2.3 Performance of Members

The plastic rotation values at the target displacement for each end of beam K113 are $\theta_{p(i)}$ = 0.006924 rad and $\theta_{p(j)}$ = -0.00942 rad. The results show that hinging occurs only at lower end of column 1S18. the upper end remains elastic. The plastic rotation at lower end is θ_p = 0.006689 rad.

The corresponding strain values are obtained by converting these values to curvatures by using Equations (2.32) and (2.33). The plastic and total curvature values at each end of the beam and the column are calculated as follows.

<u>end i of K113</u>

φ_p = 0.006924 / 0.20 = 0.03462 rad/m

φ_t = 0.03462 + 0.0269 = 0.06152 rad/m

<u>end j of K113</u>

φ_p = - 0.00942 / 0.20 = - 0.0471 rad/m

φ_t = - 0.0471 + (-0.0058) = - 0.0529 rad/m

Lower end of 1S18

φ_p = 0.006689 / 0.225 = 0.02973 rad/m

φ_t = 0.02973+ 0.0077 = 0.03743 rad/m

From these curvatures, cross sectional analyses were carried out and the strains corresponding to the total curvatures were calculated from Equation (3.3).

$$K = \frac{\varepsilon}{c} \tag{3.3}$$

where K = curvature of the cross section at specified point

ε = strain at the uppermost fiber of the cross section

c = compression zone in the cross section

The total curvature values and corresponding strains for the beam and column ends are

end i of K113

φ_t = 0.06152 rad/m ➔ ε = 0.00223

end j of K113

φ_t = -0.0529 rad/m ➔ ε = 0.00187

Lower end of 1S18

φ_t = 0.03743 rad/m ➔ ε = 0.00701

The limit values for strain are obtained for life safety performance level from the code and compared with the strains of members as follows. Note that all members are assumed to be unconfined.

From Equation (2.35). ε_{limit} is computed as 0.0035 for concrete strain as shown below.

$\rho_s / \rho_{sm} \approx 0$ ➔ $(\varepsilon_{cg})_{GV} = 0.0035$ and $(\varepsilon_s)_{GV} = 0.040$

For end i of K113

$\varepsilon / \varepsilon_{limit} = 0.00223 / 0.0035 = 0.64 < 1$ (OK)

For end j of K113

$\varepsilon / \varepsilon_{limit} = 0.00187 / 0.0035 = 0.53 < 1$ (OK)

For bottom of 1S18

$\varepsilon / \varepsilon_{limit} = 0.00701 / 0.0035 = 2.00 > 1$ (NOT OK)

These results show that both ends of beam K113 satisfy Life Safety Performance requirements of the code for nonlinear assessment procedure while 1S18 does not.

The performance assessment for all columns and beams are given in Figures A.9-A.12 and A.13-A.16. A summary of these assessments is provided in Tables 3.17-3.20 and 3.21-3.24.

Table 3.17 Summary of Nonlinear Assessment Results for Columns (+X Direction)

+X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Columns	9	20	23	18	23
	Column Percentage	**33.33%**	**74.07%**	**85.19%**	**66.67%**	**85.19%**
	Shear Force Percentage	**39.14%**	**63.94%**	**78.55%**	**52.19%**	**76.23%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ >1)	Number of Columns	18	7	4	9	4
	Column Percentage	**66.67%**	**25.93%**	**14.81%**	**33.33%**	**14.81%**
	Shear Force Percentage	**60.86%**	**36.06%**	**21.45%**	**47.81%**	**23.77%**

Table 3.18 Summary of Nonlinear Assessment Results for Columns (-X Direction)

-X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Columns	9	23	24	19	23
	Column Percentage	**33.33%**	**85.19%**	**88.89%**	**70.37%**	**85.19%**
	Shear Force Percentage	**43.64%**	**82.53%**	**85.43%**	**59.40%**	**69.12%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ >1)	Number of Columns	18	4	3	8	4
	Column Percentage	**66.67%**	**14.81%**	**11.11%**	**29.63%**	**14.81%**
	Shear Force Percentage	**56.36%**	**17.47%**	**14.57%**	**40.60%**	**30.88%**

Table 3.19 Summary of Nonlinear Assessment Results for Columns (+Y Direction)

+Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Columns	17	27	27	24	22
	Column Percentage	**62.96%**	**100.00%**	**100.00%**	**88.89%**	**81.48%**
	Shear Force Percentage	**65.66%**	**100.00%**	**100.00%**	**86.87%**	**73.44%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ >1)	Number of Columns	10	0	0	3	5
	Column Percentage	**37.04%**	**0.00%**	**0.00%**	**11.11%**	**18.52%**
	Shear Force Percentage	**34.34%**	**0.00%**	**0.00%**	**13.13%**	**26.56%**

Table 3.20 Summary of Nonlinear Assessment Results for Columns (-Y Direction)

-Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Columns	14	27	27	25	22
	Column Percentage	**51.85%**	**100.00%**	**100.00%**	**92.59%**	**81.48%**
	Shear Force Percentage	**34.03%**	**100.00%**	**100.00%**	**84.97%**	**60.52%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ >1)	Number of Columns	13	0	0	2	5
	Column Percentage	**48.15%**	**0.00%**	**0.00%**	**7.41%**	**18.52%**
	Shear Force Percentage	**65.97%**	**0.00%**	**0.00%**	**15.03%**	**39.48%**

Table 3.21 Summary of Nonlinear Assessment Results for Beams (+X Direction)

+X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Beams	18	18	18	18	18
	Beam Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Beams	0	0	0	0	0
	Beam Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 3.22 Summary of Nonlinear Assessment Results for Beams (-X Direction)

-X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Beams	18	18	18	18	18
	Beam Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Beams	0	0	0	0	0
	Beam Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 3.23 Summary of Nonlinear Assessment Results for Beams (+Y Direction)

+Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Beams	13	13	13	13	13
	Beam Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Beams	0	0	0	0	0
	Beam Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 3.24 Summary of Nonlinear Assessment Results for Beams (-Y Direction)

-Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$<1)	Number of Beams	13	13	13	13	13
	Beam Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Beams	0	0	0	0	0
	Beam Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Although all of the beams appear to satisfy the performance objective, due to significant number of columns not satisfying the performance criteria, this building does not satisfy the Life Safety Performance Level of the code for nonlinear assessment.

For this reason the building is recommended to be strengthened.

CHAPTER IV

CASE STUDY 2: ASSESSMENT OF RETROFITTED BUILDING

Linear and nonlinear assessment results for the case study building showed that this building does not satisfy Life Safety Performance Level. In this chapter, the building is retrofitted theoretically by adding shear walls to the system. Insertion of shear walls is one of the most common retrofit options. However, some conditions should be satisfied by the architectural plan of the building. Shear walls are generally placed in the bays where infill walls are present and run continuously from foundation to top floor. Also these infill walls should not involve door or window openings. Locations of shear walls need to be determined such that they are symmetrical with respect to the both principal axes of the building in order to minimize torsion effects during an earthquake.

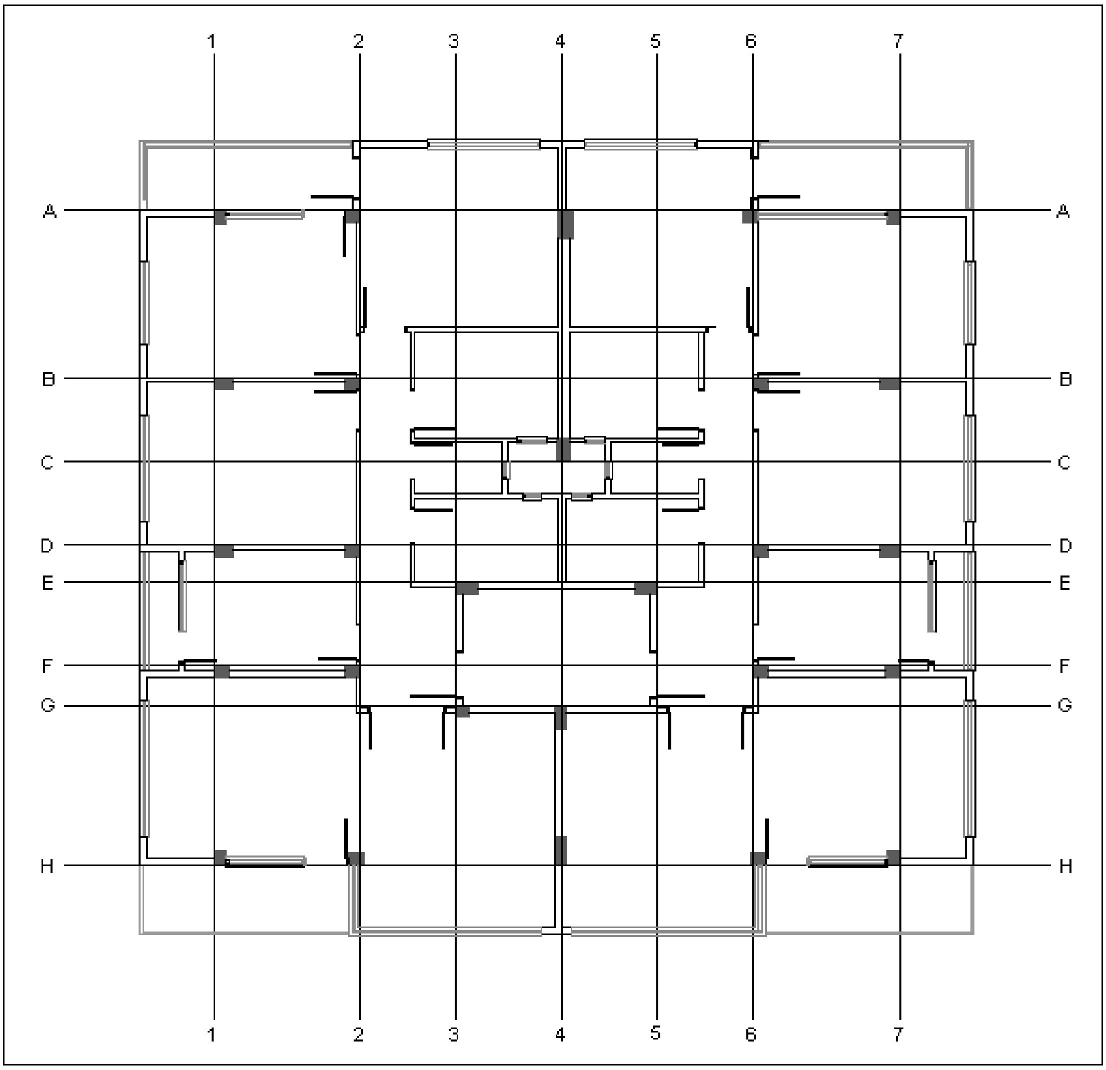

Figure 4.1 Architectural Plan of the Building

In view of these principals, the building is retrofitted with 4 shear walls in the X direction and 2 shear walls in the Y direction. The location of the shear walls have been determined considering the architectural plan shown in Figure 4.1. They are placed symmetrically with respect to both axes. The thickness of walls is 25 cm. Properties of the new shear walls are given in Table 4.1 and new frame system of the structure is shown in Figure 4.2.

Table 4.1 Shear Wall Properties

Shear Walls		b_w (cm)	h (cm)	Wall End Zone Reinforcement	Wall Web Reinforcement
X Direction	P1	25	300	4 x 2 / ø25	12 x 2 / ø20
	P2	25	300	4 x 2 / ø25	12 x 2 / ø20
	P3	25	300	4 x 2 / ø25	12 x 2 / ø20
	P4	25	300	4 x 2 / ø25	12 x 2 / ø20
Y Direction	P5	25	540	8 x 2 / ø25	16 x 2 / ø20
	P6	25	345	4 x 2 / ø25	13 x 2 / ø20

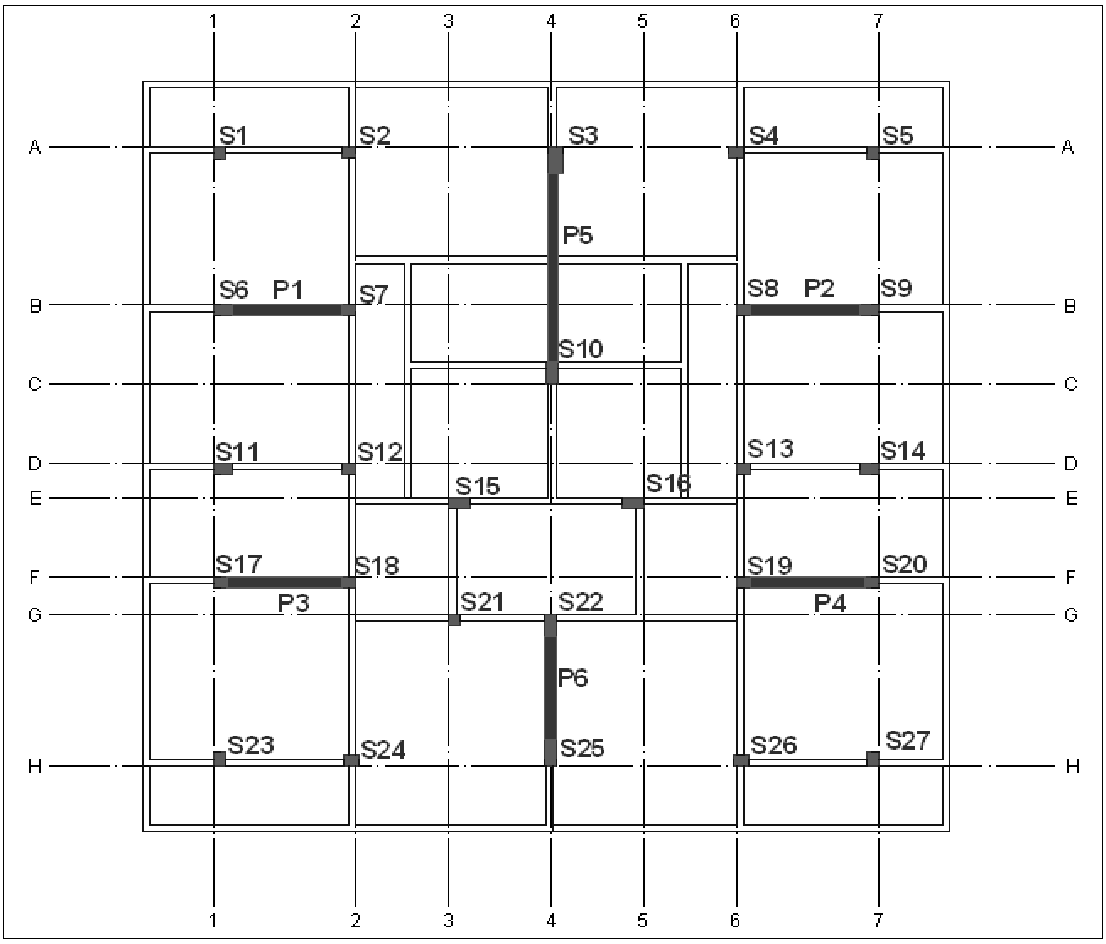

Figure 4.2 New Floor Plan with Additional Shear Walls

3-D models for both linear and nonlinear analysis are modified after retrofit. Retrofitted model is prepared by adding the shear walls to the building. Columns adjacent to shear walls are removed from model. Also, properties of beams between two adjacent shear walls are modified because beams between shear walls behave as a part of them.

4.1 Linear Assessment Procedure

Retrofitted building is analyzed for both gravity and earthquake loads separately and demands are compared with capacity of each member.

Shear walls affect the earthquake load since the building becomes more rigid after retrofit. Therefore, dynamic analysis is performed and new periods are obtained as 0.40 sec and 0.32 sec for X and Y directions, respectively. Figure 4.3 shows S(T) values for the corresponding periods.

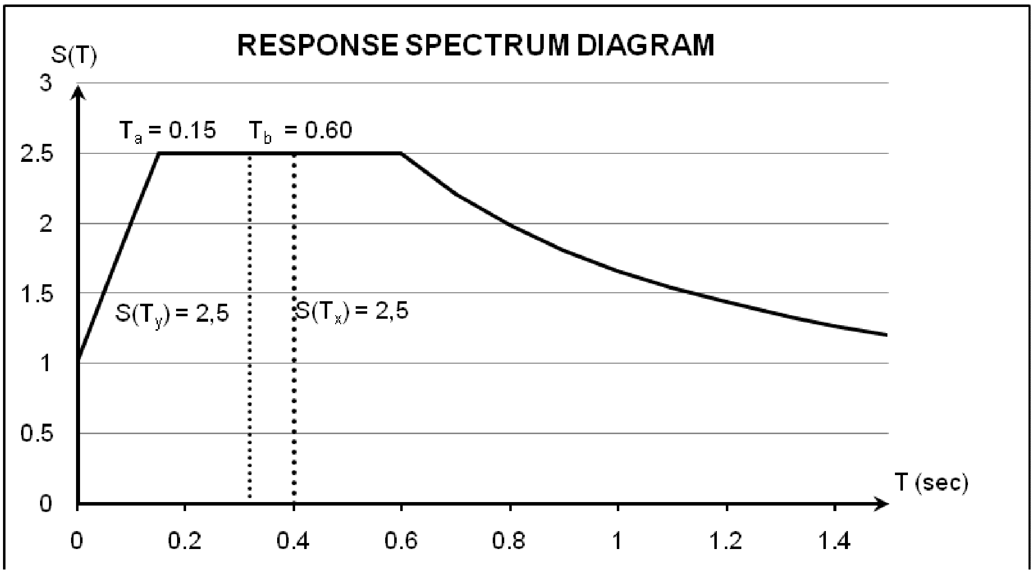

Figure 4.3 Estimating S(T) Values from Response Spectrum Diagram

New base shear force is computed as 8582.3 kN for both axes. Shear force distribution for each storey is shown in Tables 4.2.

Table 4.2 Earthquake Loads Distributed to Each Storey (X and Y Direction)

# of Storey	**w_i**	**H_i**	**$w_i H_i$**	$\frac{w_i H_i}{\sum w_i H_i}$	**V_i**
1	1856.70	2.9	5384.44	0.08	630.01
2	1856.70	5.8	10768.88	0.15	1260.02
3	1856.70	8.7	16153.32	0.23	1890.03
4	1856.70	11.6	21537.76	0.31	2520.04
5	1155.48	14.5	16754.46	0.24	2282.20

Ductility of each member is checked for the further steps of the assessment. For retrofitted building, ductility of shear walls are also controlled by using Equations (2.17) and (2.18).

For 1P2 shear wall, analysis results are shown in the following table.

Table 4.3 Analysis Result of 1P2 Shear Wall

	M_d (kN.m)	**V_d (kN)**
1P2 (top)	11840.6	1775.5
1P2 (bottom)	16969.4	1775.5

Critical section is the bottom of the shear wall. Ductility is checked for this section.

$$V_e = 1.5 \frac{8174.2}{16969.4} 1775.5 = 1282.9 \, kN$$

$$V_r = 250 \text{ x } 3000 \text{ x } (0.65 \text{ x } 0.35 \text{ x } \sqrt{30} + 0.0153 \text{ x } 365) /1000 = 5040.80 \text{ kN}$$

This result shows that. 1P2 is ductile according to the 2007 Turkish Earthquake Code.

Building is analyzed for 4 directions and none of the column demand exceeded its capacity in linear assessment. Therefore, retrofitted shear walls and beams were assessed. The results are given in Figures B.1-B.8 and summarized in Tables 4.4-4.11.

Table 4.4 Summary of Linear Assessment Results for Shear Walls (+X Direction)

+X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/r_{limit} <1)	Number of Shear Walls	4	4	4	4	4
	Shear Wall Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
	Shear Force Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/r_{limit} >1)	Number of Shear Walls	0	0	0	0	0
	Shear Wall Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**
	Shear Force Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 4.5 Summary of Linear Assessment Results for Shear Walls (-X Direction)

-X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/r_{limit} <1)	Number of Shear Walls	4	4	4	4	4
	Shear Wall Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
	Shear Force Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/r_{limit} >1)	Number of Shear Walls	0	0	0	0	0
	Shear Wall Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**
	Shear Force Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 4.6 Summary of Linear Assessment Results for Shear Walls (+Y Direction)

+Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/r_{limit} <1)	Number of Shear Walls	2	2	2	2	2
	Shear Wall Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
	Shear Force Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/r_{limit} >1)	Number of Shear Walls	0	0	0	0	0
	Shear Wall Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**
	Shear Force Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 4.7 Summary of Linear Assessment Results for Shear Walls (-Y Direction)

-Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/r_{limit} <1)	Number of Shear Walls	2	2	2	2	2
	Shear Wall Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
	Shear Force Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/r_{limit} >1)	Number of Shear Walls	0	0	0	0	0
	Shear Wall Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**
	Shear Force Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 4.8 Summary of Linear Assessment Results for Beams (+X Direction)

+X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/r_{limit} <1)	Number of Beams	18	18	18	18	18
	Beam Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/r_{limit} <1)	Number of Beams	0	0	0	0	0
	Beam Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 4.9 Summary of Linear Assessment Results for Beams (-X Direction)

-X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/r_{limit} <1)	Number of Beams	18	18	18	18	18
	Beam Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/r_{limit} <1)	Number of Beams	0	0	0	0	0
	Beam Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 4.10 Summary of Linear Assessment Results for Beams (+Y Direction)

+Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/r_{limit} <1)	Number of Beams	13	13	13	13	13
	Beam Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/r_{limit} <1)	Number of Beams	0	0	0	0	0
	Beam Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 4.11 Summary of Linear Assessment Results for Beams (-Y Direction)

-Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/r_{limit} <1)	Number of Beams	13	13	13	13	13
	Beam Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/r_{limit} <1)	Number of Beams	0	0	0	0	0
	Beam Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Results show that shear wall retrofit option is a good solution, because all the members satisfy Life Safety Performance Level for linear assessment.

4.2 Nonlinear Assessment Procedure

Model for nonlinear analysis is updated by adding the nonlinear parameters of shear walls. After pushing the building up to its limit state, the following pushover curves are obtained for both directions.

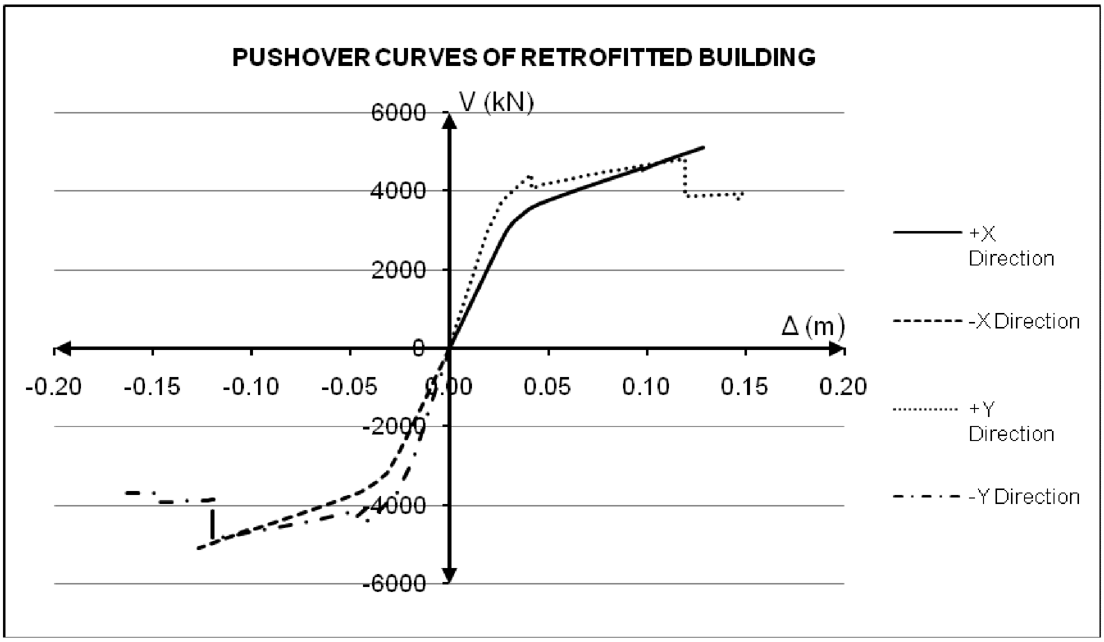

Figure 4.4 Pushover (Capacity) Curves of the Retrofitted Model for Each Direction

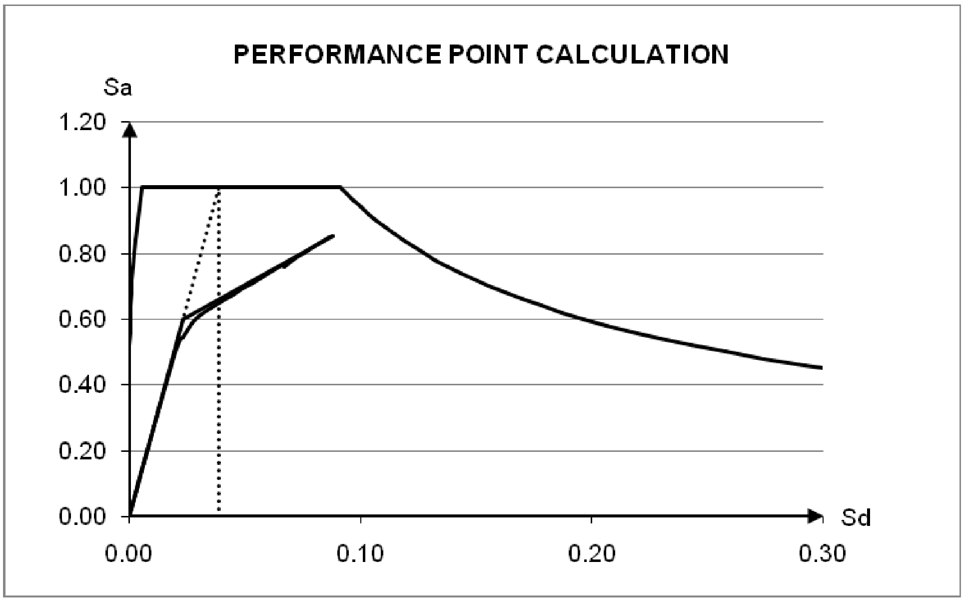

Figure 4.5 Performance Point Calculation for +X Direction

According to these curves, displacement demands are calculated and nonlinear analysis is performed. Plastic rotations are recorded and they are compared with the limits. Summary of the nonlinear analysis results are shown in the following tables. Detailed results are given in the Figures B.9-B.18.

Table 4.12 Summary of Nonlinear Assessment Results for Shear Walls (+X Direction)

+X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Shear Walls	4	4	4	4	4
	Shear Wall Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
	Shear Force Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ >1)	Number of Shear Walls	0	0	0	0	0
	Shear Wall Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**
	Shear Force Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 4.13 Summary of Nonlinear Assessment Results for Shear Walls (-X Direction)

-X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Shear Walls	4	4	4	4	4
	Shear Wall Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
	Shear Force Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ >1)	Number of Shear Walls	0	0	0	0	0
	Shear Wall Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**
	Shear Force Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 4.14 Summary of Nonlinear Assessment Results for Shear Walls (+Y Direction)

+Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Shear Walls	2	2	2	2	2
	Shear Wall Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
	Shear Force Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ >1)	Number of Shear Walls	0	0	0	0	0
	Shear Wall Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**
	Shear Force Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 4.15 Summary of Nonlinear Assessment Results for Shear Walls (-Y Direction)

+Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Shear Walls	2	2	2	2	2
	Shear Wall Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
	Shear Force Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ >1)	Number of Shear Walls	0	0	0	0	0
	Shear Wall Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**
	Shear Force Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 4.16 Summary of Nonlinear Assessment Results for Columns (+X Direction)

+X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Columns	27	27	27	27	27
	Column Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
	Shear Force Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ >1)	Number of Columns	0	0	0	0	0
	Column Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**
	Shear Force Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 4.17 Summary of Nonlinear Assessment Results for Columns (-X Direction)

-X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Columns	27	27	27	27	27
	Column Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
	Shear Force Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ >1)	Number of Columns	0	0	0	0	0
	Column Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**
	Shear Force Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 4.18 Summary of Nonlinear Assessment Results for Columns (+Y Direction)

+Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Columns	27	27	27	27	27
	Column Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
	Shear Force Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ >1)	Number of Columns	0	0	0	0	0
	Column Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**
	Shear Force Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 4.19 Summary of Nonlinear Assessment Results for Columns (-Y Direction)

-Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Columns	27	27	27	27	27
	Column Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
	Shear Force Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ >1)	Number of Columns	0	0	0	0	0
	Column Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**
	Shear Force Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 4.20 Summary of Nonlinear Assessment Results for Beams (+X Direction)

+X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Beams	18	18	18	18	18
	Beam Percentage	**100%**	**100%**	**100%**	**100%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Beams	0	0	0	0	0
	Beam Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 4.21 Summary of Nonlinear Assessment Results for Beams (-X Direction)

-X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Beams	18	18	18	18	18
	Beam Percentage	**100%**	**100%**	**100%**	**100%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Beams	0	0	0	0	0
	Beam Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 4.22 Summary of Nonlinear Assessment Results for Beams (+Y Direction)

+Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Beams	13	13	13	13	13
	Beam Percentage	**100%**	**100%**	**100%**	**100%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Beams	0	0	0	0	0
	Beam Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 4.23 Summary of Nonlinear Assessment Results for Beams (-Y Direction)

-Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Beams	13	13	13	13	13
	Beam Percentage	**100%**	**100%**	**100%**	**100%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL ($\varepsilon/\varepsilon_{limit}$ <1)	Number of Beams	0	0	0	0	0
	Beam Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

It can be concluded that retrofitted building satisfy the life safety performance level for both linear and nonlinear assessment.

CHAPTER V

CASE STUDY 3: ASSESSMENT OF RE-DESIGNED BUILDING

Up until now, a residential building is analyzed according to linear and nonlinear procedures given in the 2007 Turkish Earthquake Code. Then, it is retrofitted by adding shear walls to the system. In this chapter, the original building is re-designed according to the new code without adding or removing any member. New design is also assessed by linear and nonlinear procedures.

5.1 Design Criteria

Preliminary Design

Preliminary design for columns is made by using the minimum cross section area equation of the code. According to the code, axial load divided by multiplication of cross section and concrete strength should be at most 0.5. Therefore, cross section of each column is expanded up to a certain level. Also, minimum dimension is taken as 30 cm. For beams, width is selected as 25 cm and height is taken as 40 cm.

Table 5.1 Cross Sections of the Columns for Re-designed Building

Frame	Cross Section	Frame	Cross Section	Frame	Cross Section	Frame	Cross Section	Frame	Cross Section
1S1	40x30	**2S1**	40x30	**3S1**	40x30	**4S1**	40x30	**5S1**	40x30
1S2	30x60	**2S2**	30x60	**3S2**	30x40	**4S2**	30x40	**5S2**	30x40
1S3	70x30	**2S3**	70x30	**3S3**	70x30	**4S3**	70x30	**5S3**	70x30
1S4	30x60	**2S4**	30x50	**3S4**	30x40	**4S4**	30x40	**5S4**	30x40
1S5	40x30	**2S5**	40x30	**3S5**	40x30	**4S5**	40x30	**5S5**	40x30
1S6	30x60	**2S6**	30x60	**3S6**	30x50	**4S6**	30x50	**5S6**	30x50
1S7	30x50	**2S7**	30x50	**3S7**	30x40	**4S7**	30x40	**5S7**	30x40
1S8	30x50	**2S8**	30x50	**3S8**	30x40	**4S8**	30x40	**5S8**	30x40
1S9	30x60	**2S9**	30x50	**3S9**	30x50	**4S9**	30x50	**5S9**	30x50
1S10	70x30	**2S10**	60x30	**3S10**	60x30	**4S10**	30x60	**5S10**	60x30
1S11	30x60	**2S11**	30x60	**3S11**	30x50	**4S11**	30x50	**5S11**	30x50
1S12	30x50	**2S12**	30x50	**3S12**	30x40	**4S12**	30x40	**5S12**	30x40
1S13	30x50	**2S13**	30x50	**3S13**	30x40	**4S13**	30x40	**5S13**	30x40
1S14	30x60	**2S14**	30x60	**3S14**	30x50	**4S14**	30x50	**5S14**	30x50
1S15	30x50	**2S15**	30x50	**3S15**	30x50	**4S15**	30x50	**5S15**	30x50
1S16	30x50	**2S16**	30x50	**3S16**	30x50	**4S16**	30x50	**5S16**	30x50
1S17	30x60	**2S17**	30x60	**3S17**	30x40	**4S17**	30x40	**5S17**	30x40
1S18	30x50	**2S18**	30x50	**3S18**	30x40	**4S18**	30x40	**5S18**	30x40
1S19	30x50	**2S19**	30x50	**3S19**	30x40	**4S19**	30x40	**5S19**	30x40
1S20	30x60	**2S20**	30x50	**3S20**	30x40	**4S20**	30x40	**5S20**	30x40
1S21	30x30	**2S21**	30x30	**3S21**	30x30	**4S21**	30x30	**5S21**	30x30
1S22	70x30	**2S22**	60x30	**3S22**	60x30	**4S22**	60x30	**5S22**	60x30
1S23	40x30	**2S23**	40x30	**3S23**	40x30	**4S23**	40x30	**5S23**	40x30
1S24	60x30	**2S24**	60x30	**3S24**	40x30	**4S24**	40x30	**5S24**	40x30
1S25	70x30	**2S25**	70x30	**3S30**	70x30	**4S25**	70x30	**5S25**	70x30
1S26	30x60	**2S26**	30x60	**3S26**	30x40	**4S26**	30x40	**5S26**	30x40
1S27	40x30	**2S27**	40x30	**3S27**	40x30	**4S27**	40x30	**5S27**	40x30

Final Design

Building is designed according to the gravity and earthquake loads. Following load combinations are used during analyses. [6]

1.4G + 1.6Q

1G+1Q+1E (for each direction)

Dynamic analysis is performed and periods for re-designed building are calculated as 0.80 and 0.95 sec for X and Y directions respectively.

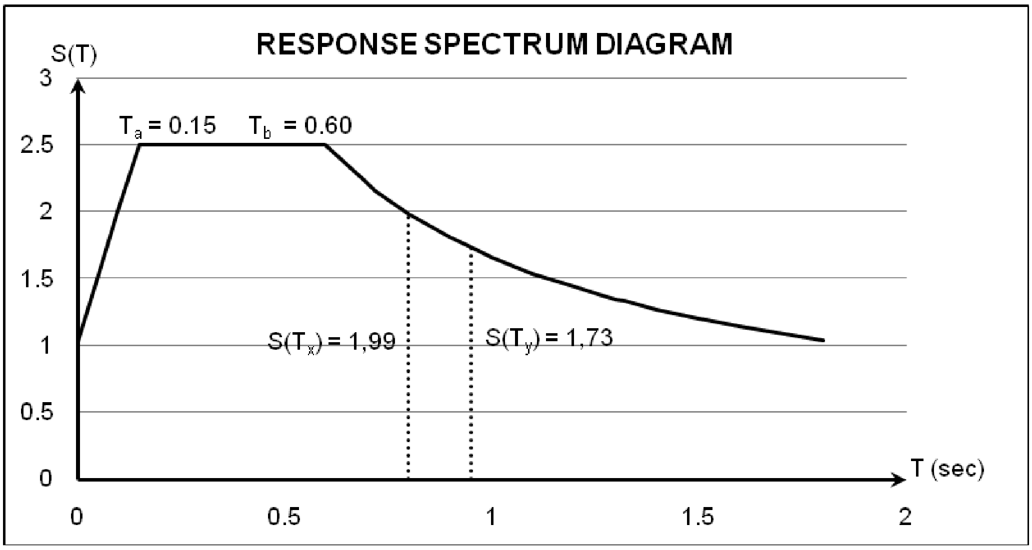

Figure 5.1 Estimating S(T) Values from Response Spectrum Diagram

R is taken as 4 for normal ductility level and importance factor is taken as 1 while calculating the earthquake load for re-designed building.

After the analysis, reinforcements are detailed according to the worst loads. Floor plan is same as the existing building shown in Figure 2.1 cross section of the members and reinforcement ratios are improved and new properties are shown in Tables 5.1 and 5.2.

Table 5.2 Reinforcement Ratios of the Columns for Re-designed Building

Cross Sections	Long. Reinforcement Ratio	Cross Sections	Long. Reinforcement Ratio
30x30	2.05%	40x30	1.37%
30x35	2.74%	50x30	1.83%
30x40	3.19%	60x30	1.37%
30x50	2.74%	70x30	2.74%

5.2 Linear Assessment Procedure

Procedures given in the previous case studies are applied successively to this building. From Figure 5.1, base shear forces for X and Y direction are as follows.

V_x = 0.85 x 8582.29 x 0.4 x 1 x 1.99 / 1 = 5806.78 kN

V_y = 0.85 x 8582.29 x 0.4 x 1 x 1.73 / 1 = 5938.94 kN

Capacities are calculated and compared with the analysis results. Shear demands and capacities are controlled for ductility check. “r” ratio of all members are calculated and compared with the limits given in the code.

Detailed results are presented in Appendix C. A summary of these results are given in Tables 5.3-5.11.

Table 5.3 Summary of Linear Assessment Results for Columns (+X Direction)

+X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit<1)	Number of Columns	27	27	27	27	27
	Column Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
	Shear Force Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit>1)	Number of Columns	0	0	0	0	0
	Column Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**
	Shear Force Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 5.4 Summary of Linear Assessment Results for Columns (-X Direction)

-X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit<1)	Number of Columns	27	27	27	27	27
	Column Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
	Shear Force Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit>1)	Number of Columns	0	0	0	0	0
	Column Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**
	Shear Force Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 5.5 Summary of Linear Assessment Results for Columns (+Y Direction)

+Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit<1)	Number of Columns	27	27	27	27	27
	Column Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
	Shear Force Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit>1)	Number of Columns	0	0	0	0	0
	Column Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**
	Shear Force Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 5.6 Summary of Linear Assessment Results for Columns (-Y Direction)

-Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit<1)	Number of Columns	27	27	27	27	27
	Column Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
	Shear Force Percentage	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit>1)	Number of Columns	0	0	0	0	0
	Column Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**
	Shear Force Percentage	**0.00%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

5.3 Nonlinear Assessment Procedure

Same procedures given in the previous chapters are followed for nonlinear assessment. Nonlinear parameters of the members are calculated and imported to the model. Performance point is calculated and building is pushed up to this level. Strains are estimated and compared with the limits given in the code.

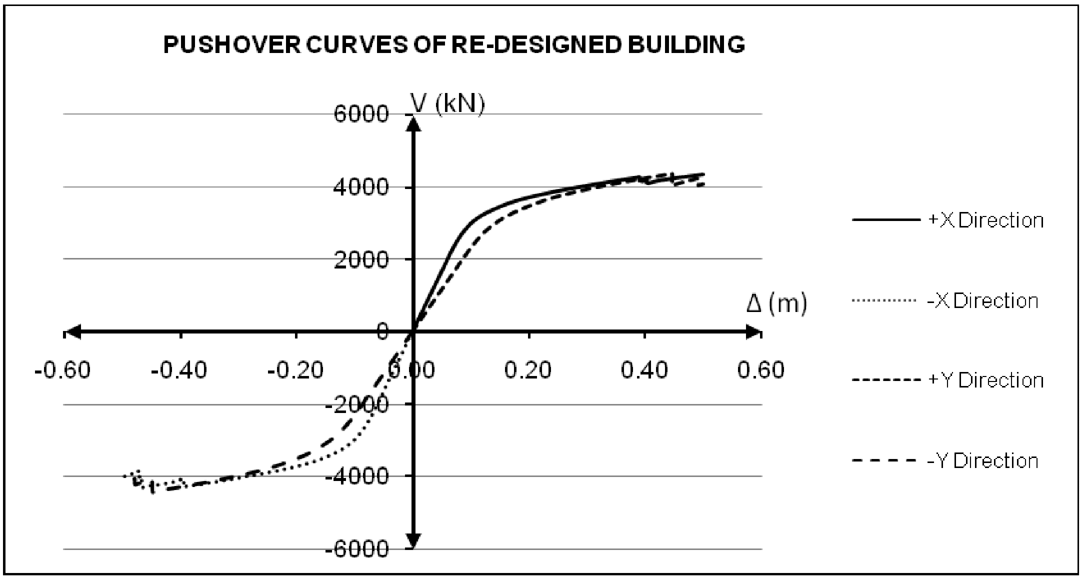

Figure 5.2 Pushover Curves of the Re-Designed Model for Each Direction

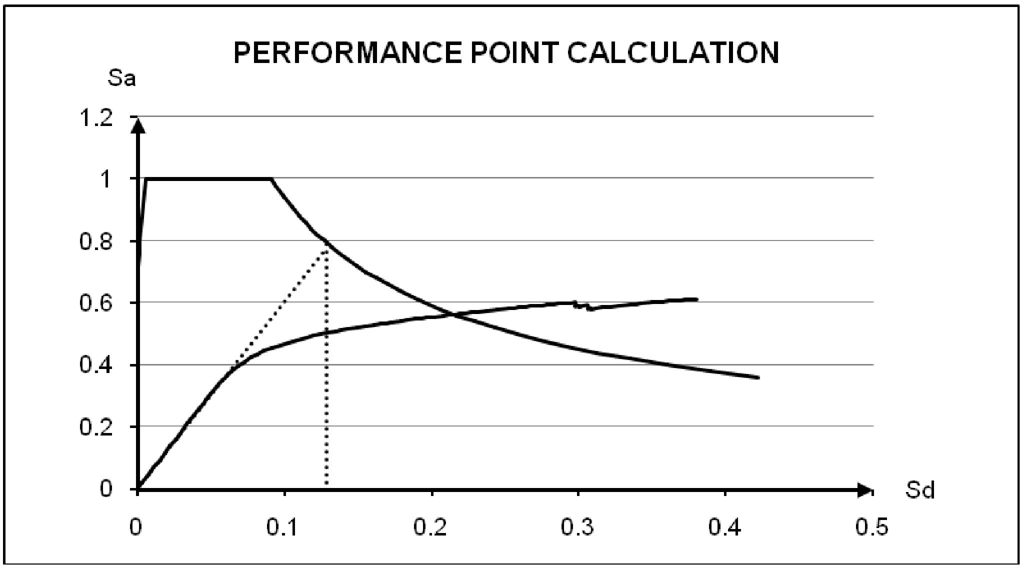

Figure 5.3 Performance Point Calculation of the Re-designed Building for +X Direction

From Figure 5.3, it can be concluded that elastic spectral displacement is 0.129 m and performance point is

u_{xN1} = 0.0494 x 26.62 x 0.129 = 0.17 m

Building is pushed up to the performance point and strains of each member is recorded. As a result, nonlinear assessment gives the same results with the linear assessment. All beams and columns satisfy the Life Safety Performance Level which can be seen in Appendix C.

It can be concluded that re-designed building satisfies the required performance level for a residential building.

CHAPTER VI

DISCUSSION OF RESULTS AND CONCLUSIONS

The case study residential building located in İstanbul has been assessed by the linear and nonlinear procedures described in the current seismic code of Turkey. Both procedures suggested that the building did not have adequate capacity to satisfy the life safety performance criteria of the code. Therefore, the building is retrofitted by adding shear walls in both principal directions. Assessment of the retrofitted building according to the code revealed satisfactory results. In order to investigate the consistency between design and assessment criteria of the code, the original building system and architecture was used to re-design its members. Their sizes and detailing were determined according to the code and the performance of the new design was assessed. It has been found that the new member designs led to a satisfactory performance.

The fundamental mode periods obtained for each case are given in Table 6.1. The displacement profiles along the height are compared in Figures 6.1 and 6.2 for each model. Note that these profiles correspond to the linear analyses results. Figures 6.3 and 6.4 show the pushover curves obtained from the nonlinear static analyses.

Table 6.1 Periods of the Models

	EXISTING	**RETROFITTED**	**RE-DESIGN**
T_x	1.27	0.40	0.80
T_y	1.52	0.33	0.95

T_y is greater than T_x for existing and re-designed model. However, it is vice versa for retrofitted model since the total rigidity of shear walls in Y direction is more than X direction.

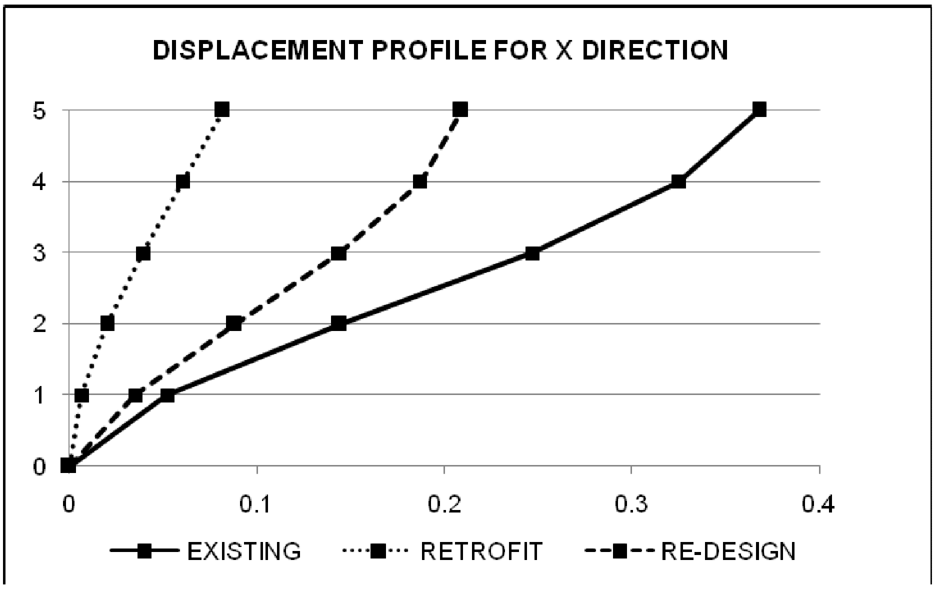

Figure 6.1 Displacement Profile for +X Direction

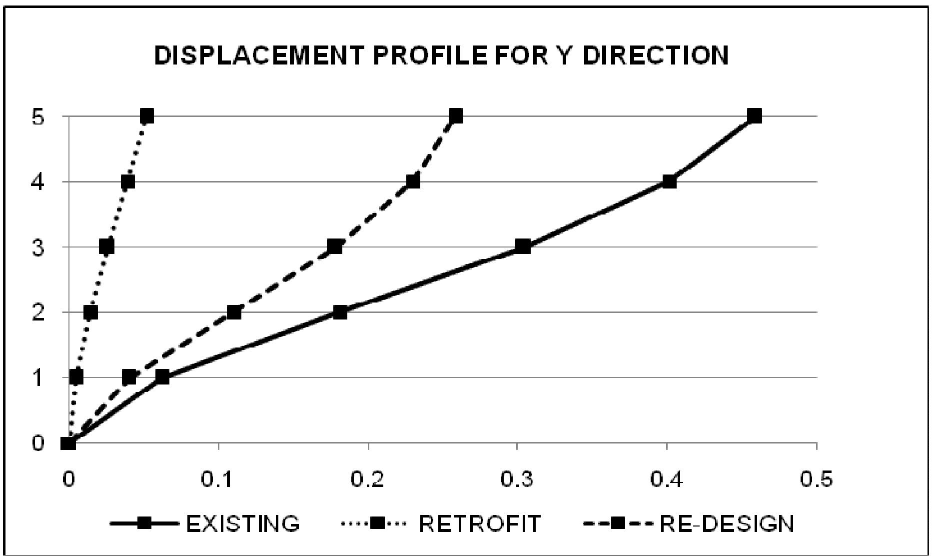

Figure 6.2 Displacement Profile for +Y Direction

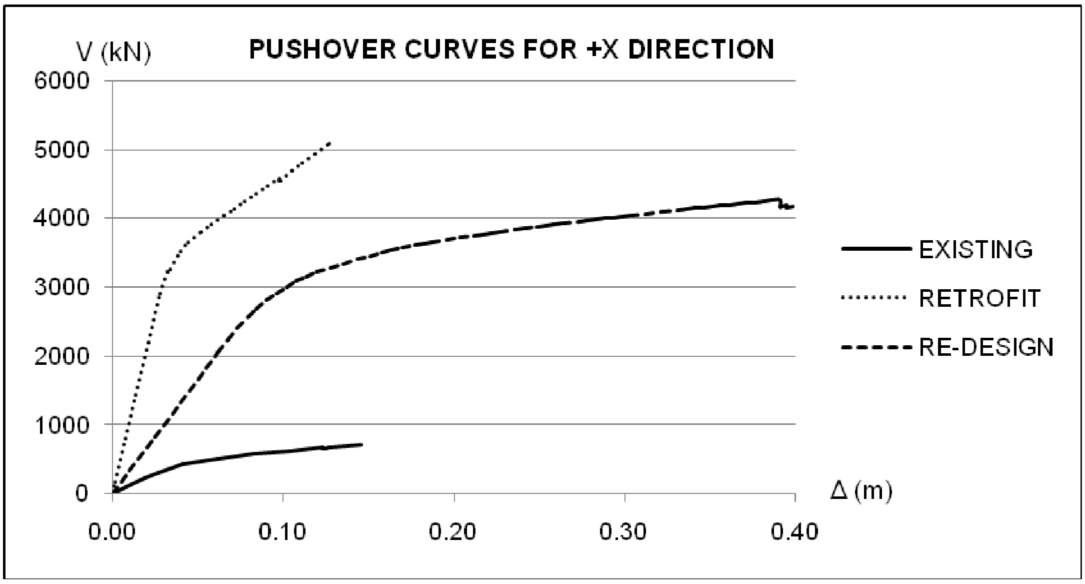

Figure 6.3 Pushover Curves for +X Direction

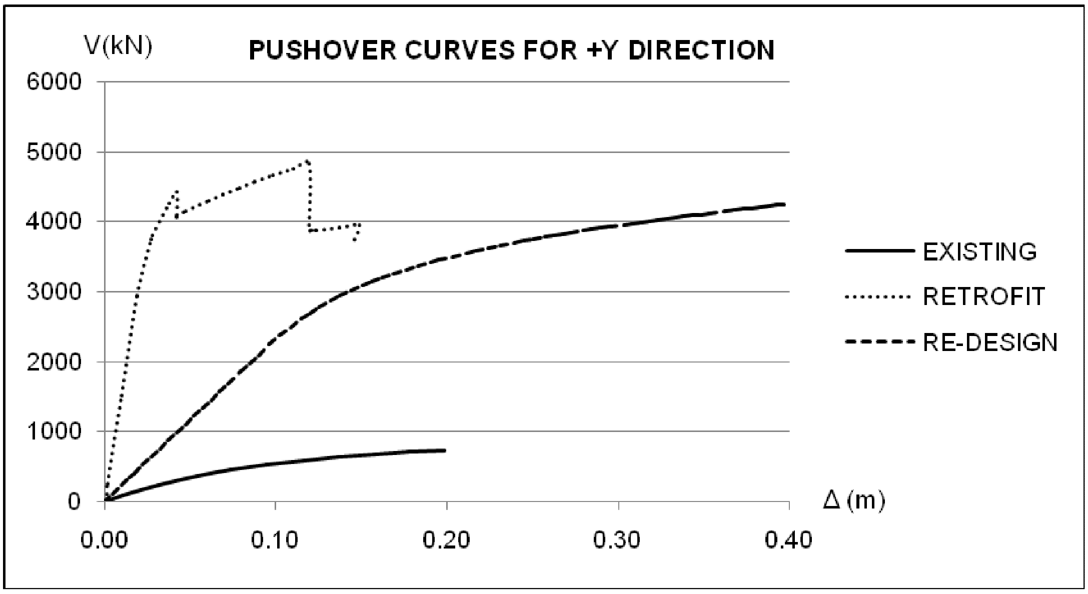

Figure 6.4 Pushover Curves for +Y Direction

Reduction factors (R) of each case is calculated for +X and +Y directions in the following figures and results are given in Table 6.2.

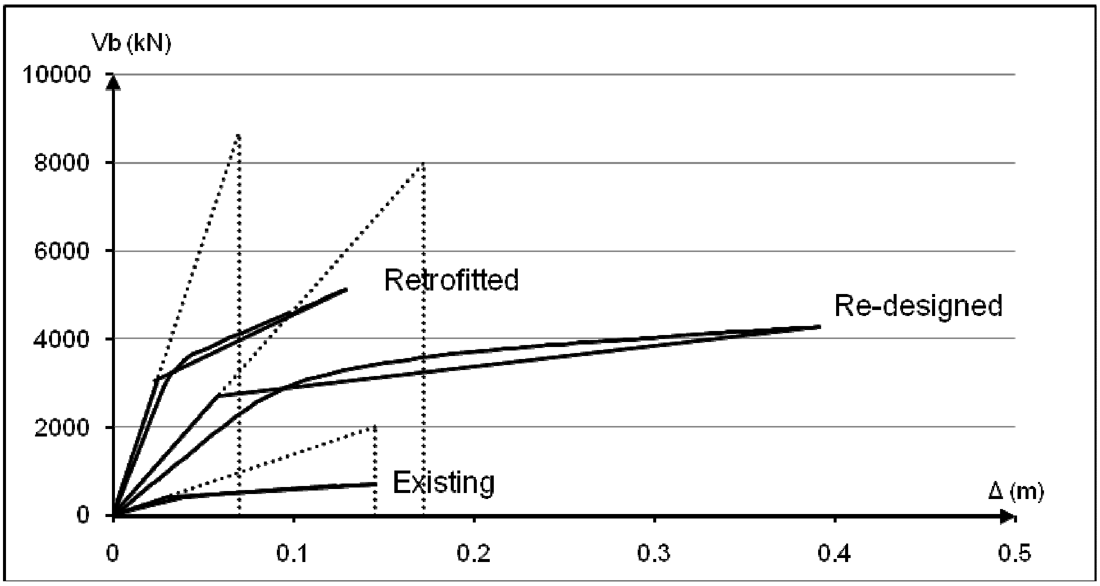

Figure 6.5 Bi-linearized Capacity Curves of Each Case for +X Direction

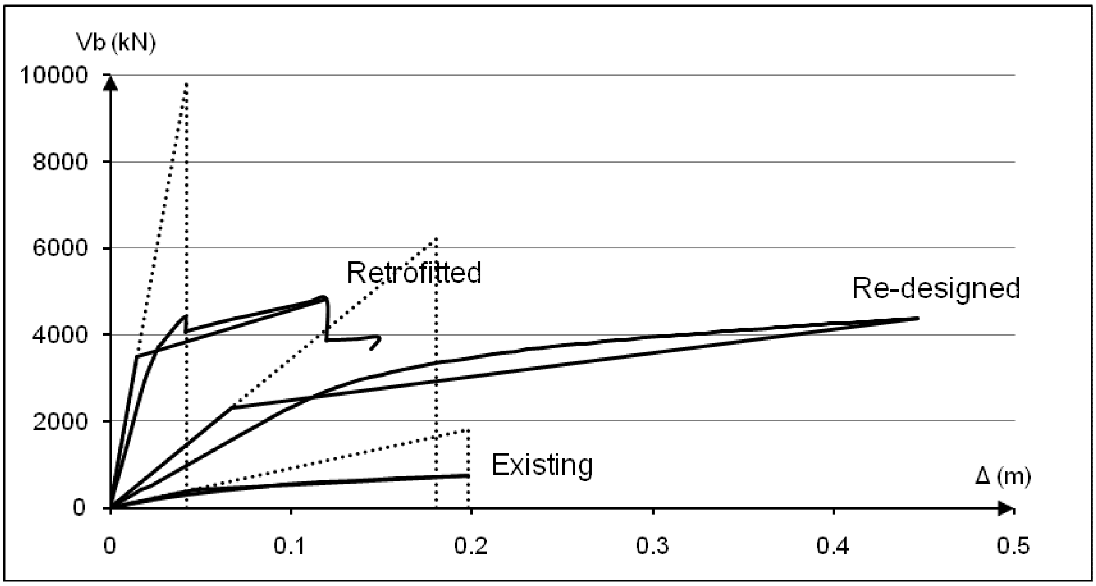

Figure 6.6 Bi-linearized Capacity Curves of Each Case for +X Direction

Table 6.2 Reduction (R) Factors for Each Case

Case Study	+X	+Y
Existing	4.76	4.29
Retrofitted	2.86	2.82
Re-designed	2.96	2.70

These three building models have also been assessed according to the FEMA 356 limits. For retrofitted and re-designed models. none of the members exceeded the limits of FEMA, so only the results of existing building are summarized in Tables 6.3-6.6. [3]

Table 6.3 Summary of Nonlinear Assessment (FEMA 356) Results for Columns (+X Direction)

+X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit<1)	Number of Columns	10	27	26	27	27
	Column Percentage	**37.04%**	**100.00%**	**96.15%**	**100.00%**	**100.00%**
	Shear Force Percentage	**41.10%**	**100.00%**	**95.60%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit>1)	Number of Columns	17	0	1	0	0
	Column Percentage	**62.96%**	**0.00%**	**3.85%**	**0.00%**	**0.00%**
	Shear Force Percentage	**58.90%**	**0.00%**	**4.40%**	**0.00%**	**0.00%**

Table 6.4 Summary of Nonlinear Assessment (FEMA 356) Results for Columns (-X Direction)

-X Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit<1)	Number of Columns	10	27	26	27	27
	Column Percentage	**37.04%**	**100.00%**	**96.15%**	**100.00%**	**100.00%**
	Shear Force Percentage	**50.38%**	**100.00%**	**96.78%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit>1)	Number of Columns	17	0	1	0	0
	Column Percentage	**62.96%**	**0.00%**	**3.85%**	**0.00%**	**0.00%**
	Shear Force Percentage	**49.62%**	**0.00%**	**3.22%**	**0.00%**	**0.00%**

Table 6.5 Summary of Nonlinear Assessment (FEMA 356) Results for Columns (+Y Direction)

+Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit<1)	Number of Columns	21	27	27	27	27
	Column Percentage	**77.78%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
	Shear Force Percentage	**77.95%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit>1)	Number of Columns	6	0	0	0	0
	Column Percentage	**22.22%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**
	Shear Force Percentage	**22.05%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

Table 6.6 Summary of Nonlinear Assessment (FEMA 356) Results for Columns (-Y Direction)

-Y Direction		1. Storey	2. Storey	3. Storey	4. Storey	5. Storey
MEMBERS SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit<1)	Number of Columns	22	27	27	27	27
	Column Percentage	**81.48%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
	Shear Force Percentage	**54.12%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**
MEMBERS NOT SATISFYING LIFE SAFETY PERFORMANCE LEVEL (r/rlimit>1)	Number of Columns	5	0	0	0	0
	Column Percentage	**18.52%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**
	Shear Force Percentage	**45.88%**	**0.00%**	**0.00%**	**0.00%**	**0.00%**

6.1 Comparison of Assessment Procedures

Seismic performance assessments of the existing building model are compared in Table 6.7. Since the retrofitted and re-designed building members are satisfied for all assessment procedures, they are not taken into consideration. As it is seen, FEMA 356. TEC 2007 linear and TEC 2007 nonlinear assessment procedures suggest similar results for the overall building performances. However, TEC 2007 linear procedure appears to be more conservative than others. The most unconservative results are observed from FEMA 356 procedure. To investigate the reasons for these differences, the acceptance criteria for the three procedures have been compared. The reason for comparing the acceptance criteria is that since the model and analysis phase is very similar the major difference in the assessment arises due to performance based limit values.

In order to express the plastic rotation limits given in FEMA 356 in terms of corresponding strain limits, section analyses results obtained for each member have been used. Therefore, the effect of axial loads. member detailing and cross sectional properties have been used to obtain the corresponding strain limits of the plastic rotation values given in FEMA 356.

Table 6.7 Columns not Satisfying Life Safety Performance Level for Existing Building

# of Storey	Directions	r / r_{limit} (TEC 2007)		$\varepsilon / \varepsilon_{limit}$ (TEC 2007)		$\varepsilon / \varepsilon_{limit}$ (FEMA 356)	
		Number of Columns	Column Percentage	Number of Columns	Column Percentage	Number of Columns	Column Percentage
1	+X	25	92.59%	18	66.67%	17	62.96%
	-X	25	92.59%	18	66.67%	17	62.96%
	+Y	22	81.48%	10	37.04%	6	22.22%
	-Y	22	81.48%	13	48.15%	5	18.52%
2	+X	18	66.67%	7	25.93%	-	0.00%
	-X	18	66.67%	4	14.81%	-	0.00%
	+Y	15	55.56%	-	0.00%	-	0.00%
	-Y	16	59.26%	-	0.00%	-	0.00%
3	+X	24	88.89%	4	14.81%	1	3.70%
	-X	23	85.19%	3	11.11%	1	3.70%
	+Y	15	55.56%	-	0.00%	-	0.00%
	-Y	14	51.85%	-	0.00%	-	0.00%
4	+X	15	55.56%	9	33.33%	-	0.00%
	-X	17	62.96%	8	29.63%	-	0.00%
	+Y	11	40.74%	3	11.11%	-	0.00%
	-Y	12	44.44%	2	7.41%	-	0.00%
5	+X	-	0.00%	4	14.81%	-	0.00%
	-X	-	0.00%	4	14.81%	-	0.00%
	+Y	4	14.81%	5	18.52%	-	0.00%
	-Y	4	14.81%	5	18.52%	-	0.00%

Figures 6.7 - 6.14 provide comparative results of first and second story columns only for r/r_{limit} (TEC 2007). $\varepsilon/\varepsilon_{limit}$ (TEC 2007). $\varepsilon/\varepsilon_{limit}$ (FEMA 356) and θ/θ_{limit} (FEMA 356) for the original building employed. As indicated earlier, all members of this building are assumed to be unconfined. As it is expected the plastic rotation and strain limit ratios are very similar for FEMA 356. Besides, the ratios of demand to the limit values (i.e. demand to capacity ratios) for TEC 2007 nonlinear and TEC linear procedures are larger than the FEMA 356 values indicating that both procedures of

TEC 2007 are more conservative than FEMA 356. At upper floors due to changes in the axial load TEC 2007 linear becomes more conservative. FEMA 356 gives the largest limits for this case. It is also observed that nonlinear procedure is more conservative in the weak direction (X-direction) corresponding to larger period whereas the linear procedure is more conservative in the strong direction (Y-direction).

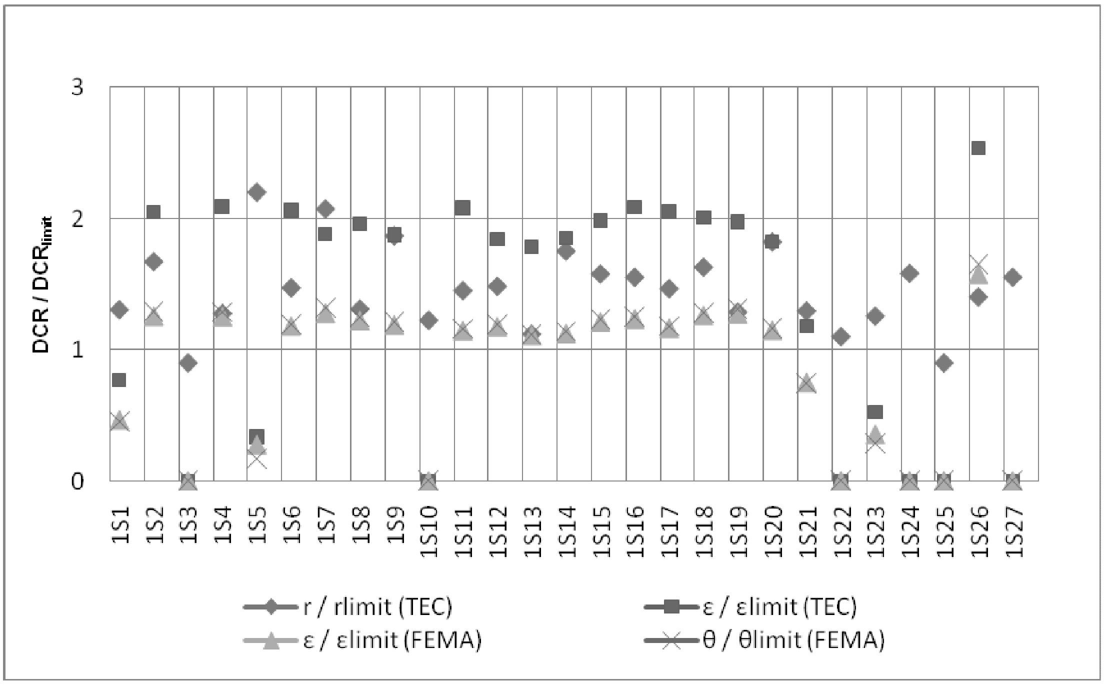

Figure 6.7 Performance Results of 1st Storey Columns for Existing Building (+X Direction)

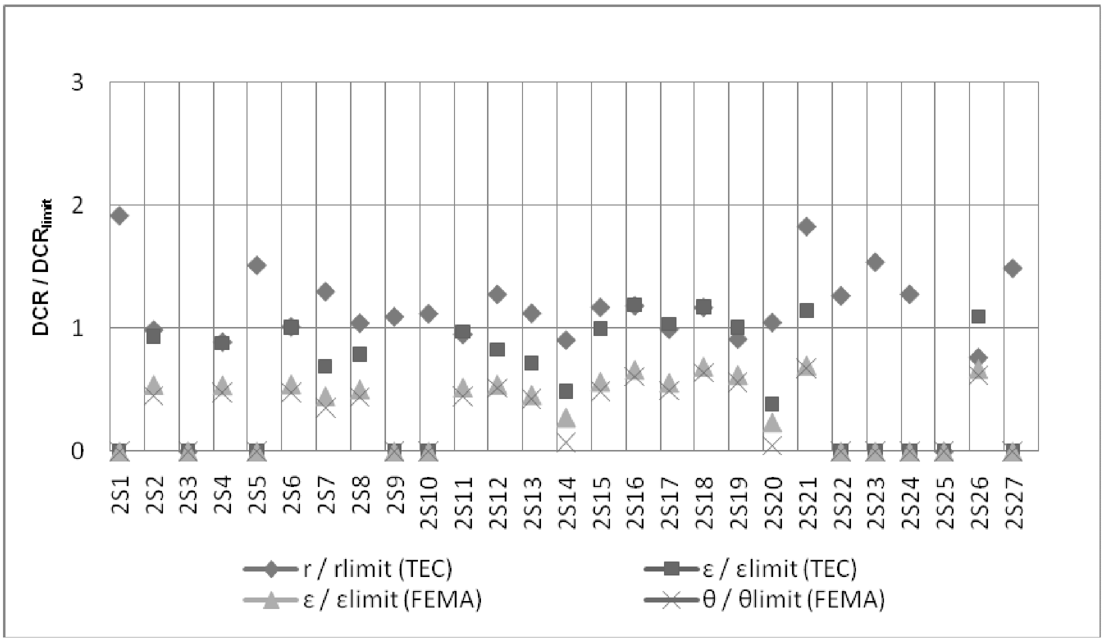

Figure 6.8 Performance Results of 2nd Storey Columns for Existing Building (+X Direction)

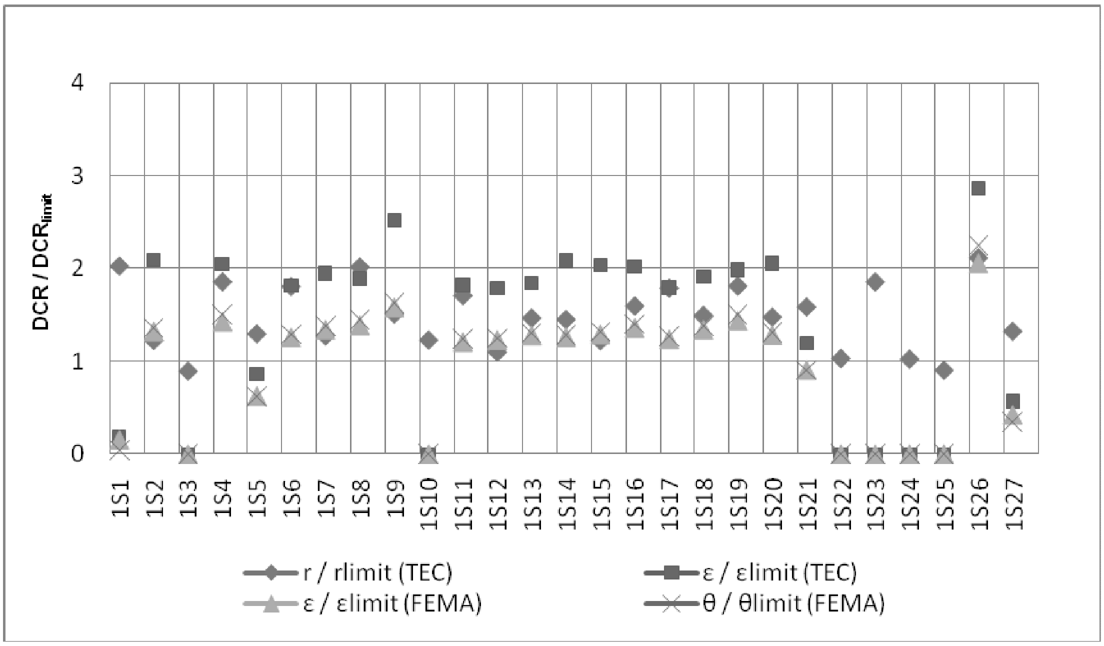

Figure 6.9 Performance Results of 1st storey Columns for Existing Building (-X Direction)

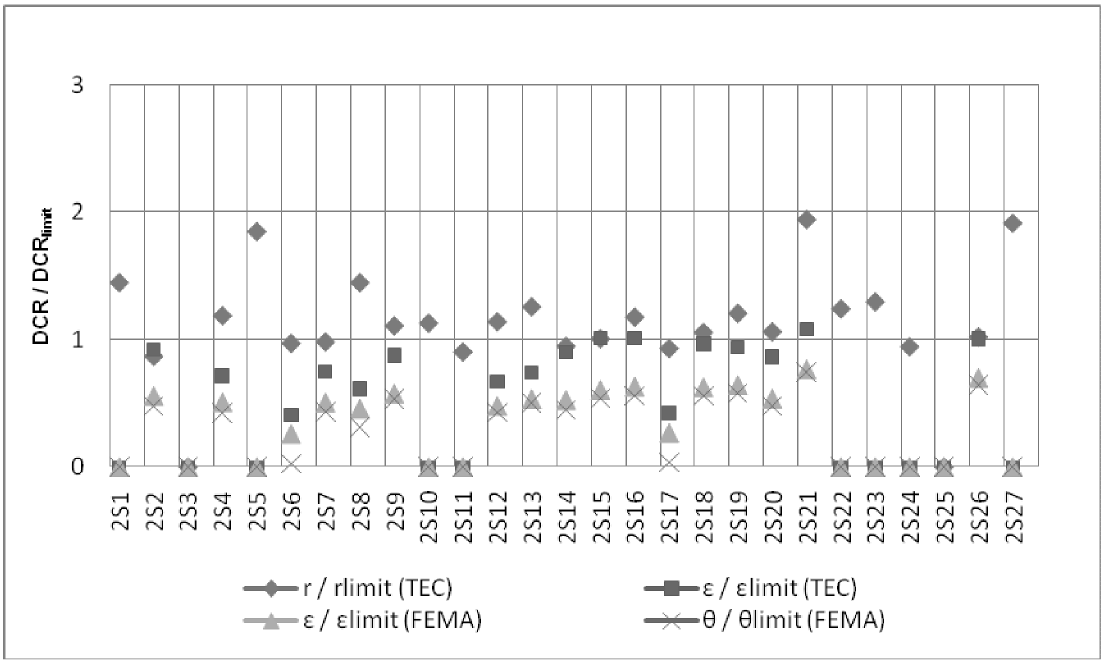

Figure 6.10 Performance Results of 2nd Storey Columns for Existing Building (-X Direction)

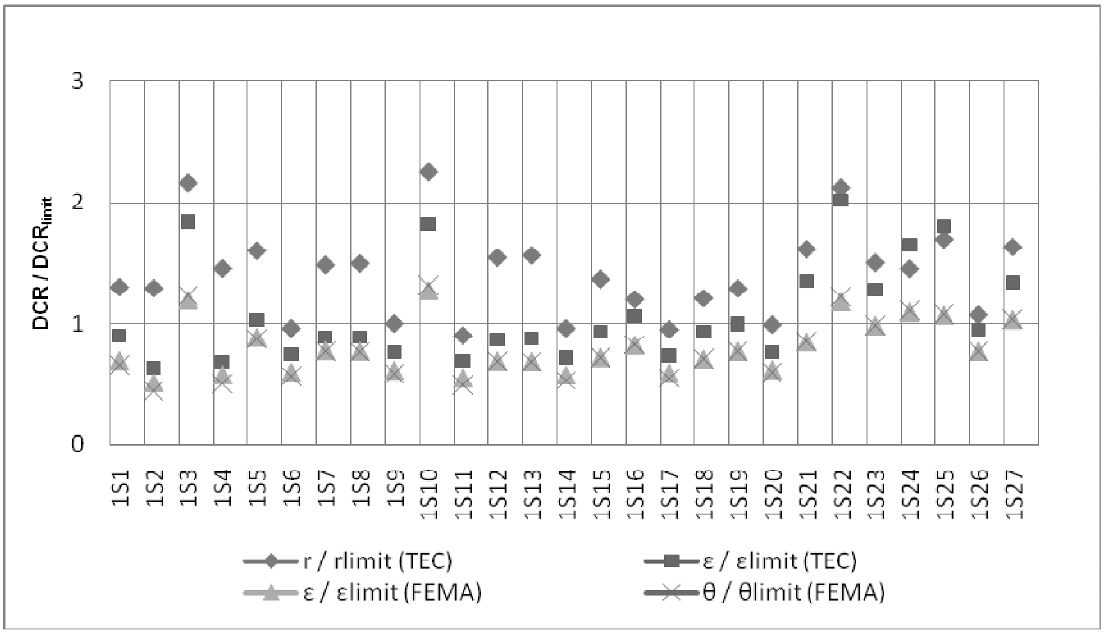

Figure 6.11 Performance results of 1st Storey Columns for Existing Building (+Y Direction)

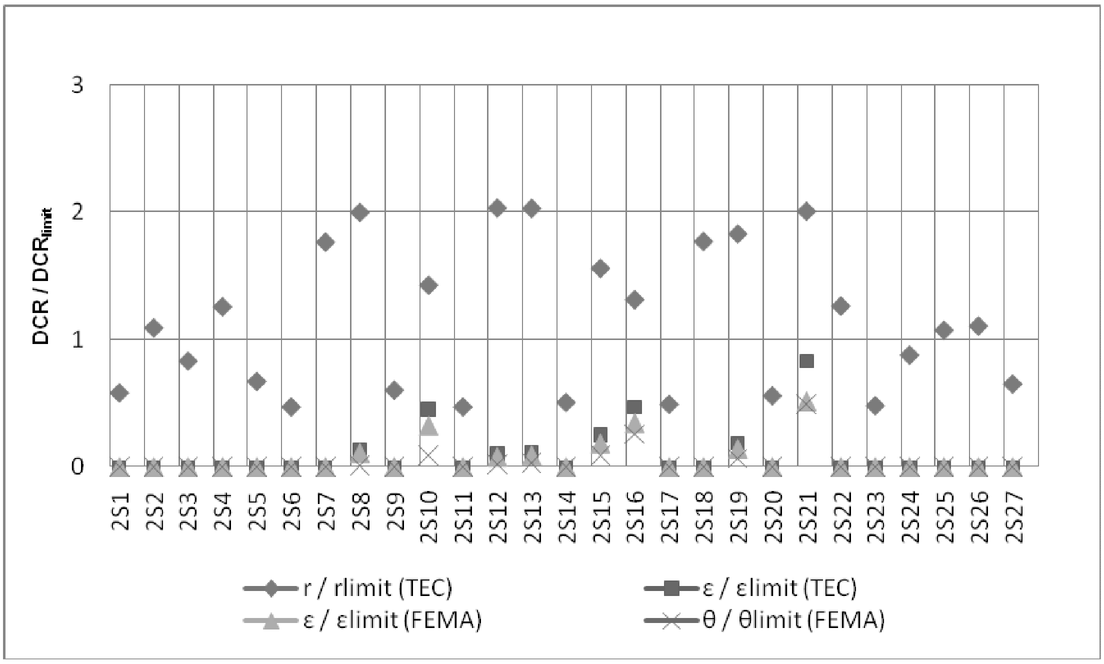

Figure 6.12 Performance Results of 2nd Storey Columns for Existing Building (+Y Direction)

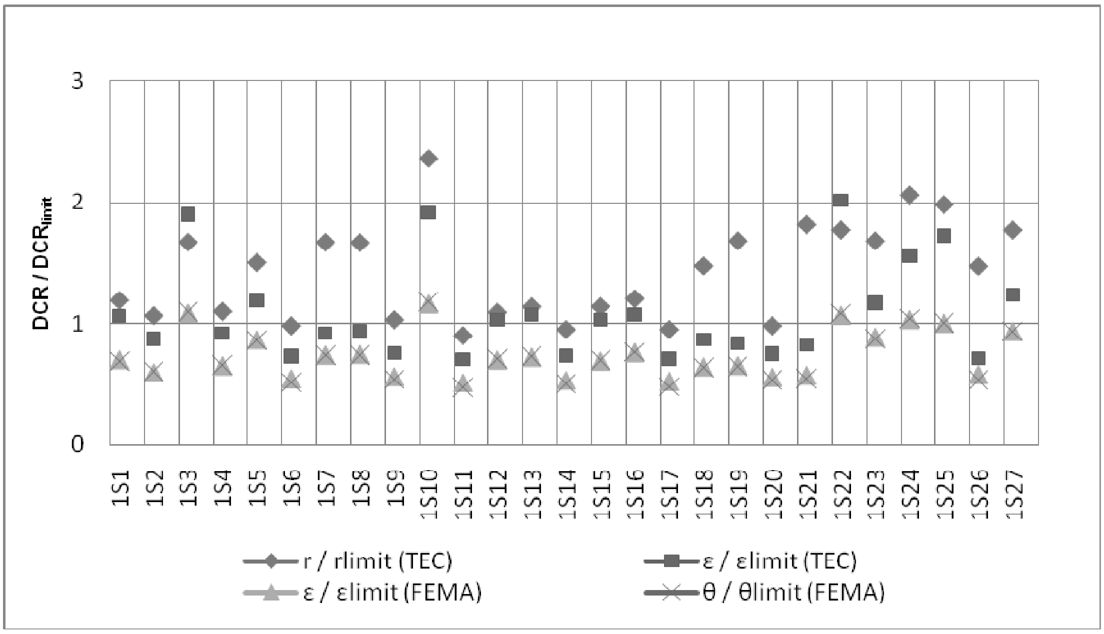

Figure 6.13 Performance Results of 1st Storey Columns for Existing Building (-Y Direction)

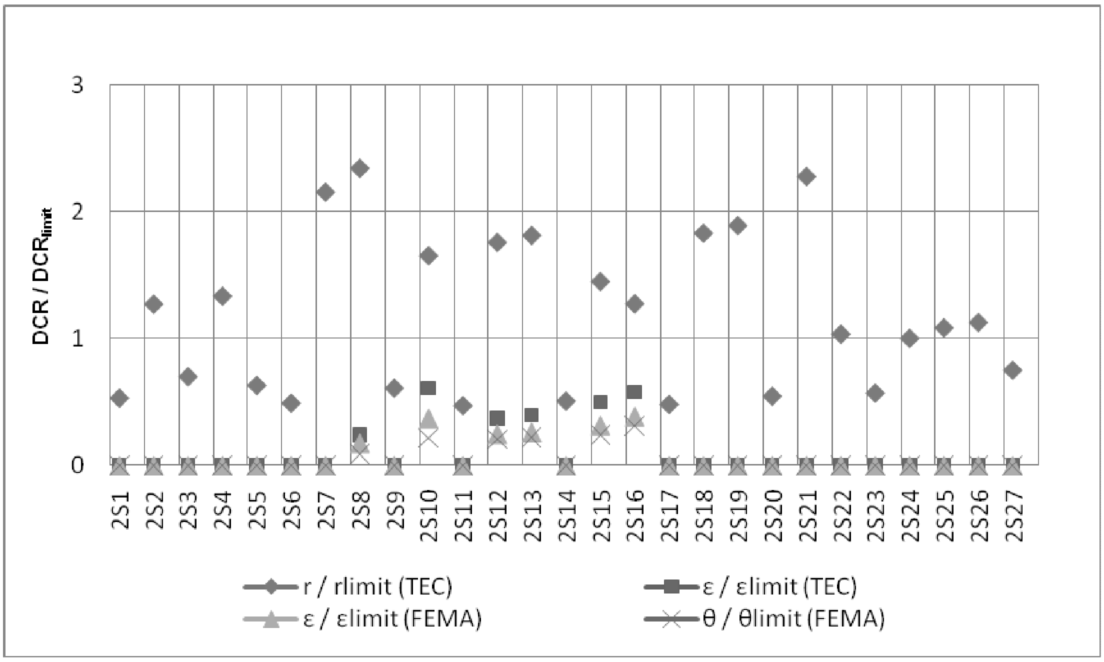

Figure 6.14 Performance Results of 2nd Storey Columns for Existing Building (-Y Direction)

For retrofitted building, since the performance is governed by behavior of the shear walls, the focus has been devoted to the wall limits. Tables 4.4 - 4.7 show linear and Tables 4.12 - 4.15 show nonlinear assessment results for shear walls. None of the members exceed the Life Safety Performance Level and linear assessment procedure of TEC 2007 gives the most conservative result for retrofitted building.

For re-designed building, Figures 6.15-6.22 shows the comparative results for the first two columns. It can be concluded that properly designed and detailed building is becomes more ductile and this causes a better consistency between linear and nonlinear assessment.

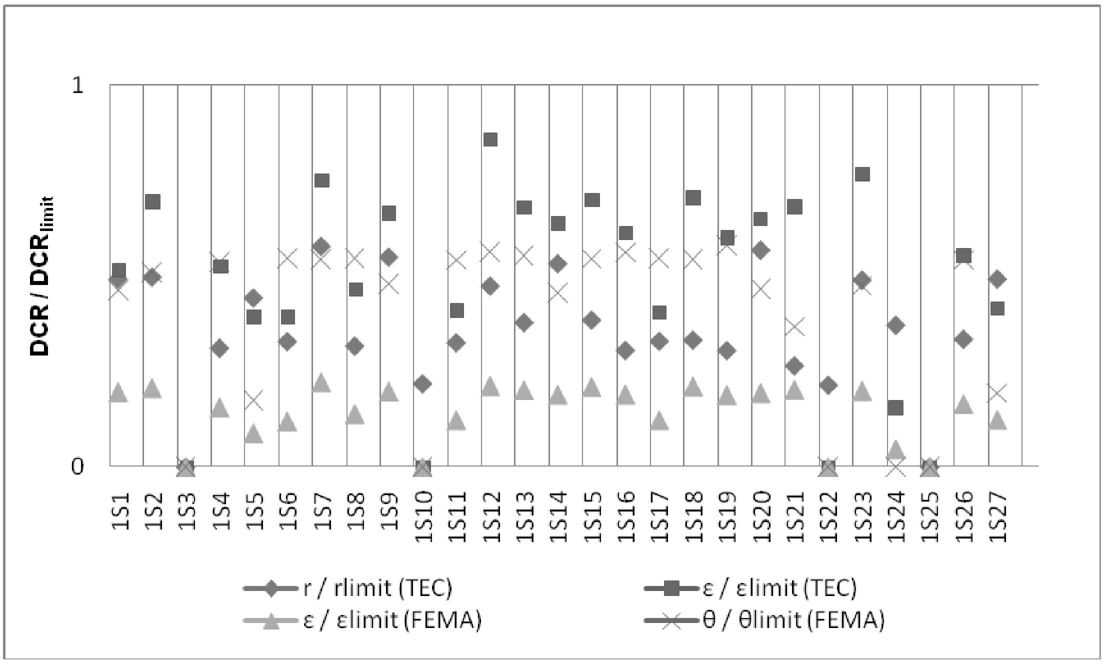

Figure 6.15 Performance Results of 1st Storey Columns for Re-designed Building (+X Direction)

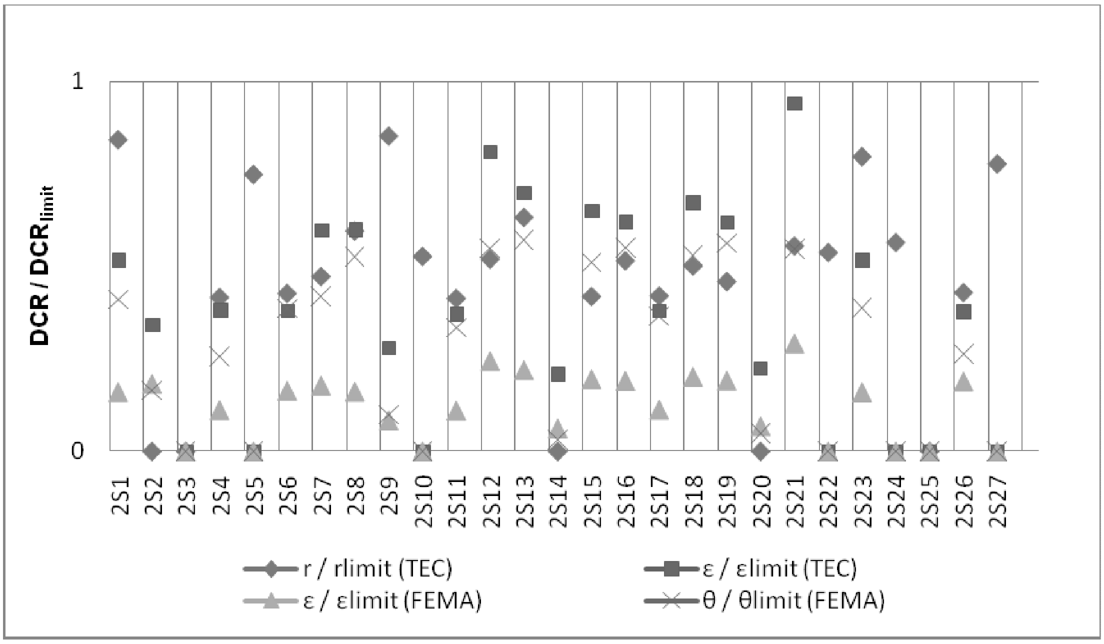

Figure 6.16 Performance Results of 2nd Storey Columns for Re-designed Building (+X Direction)

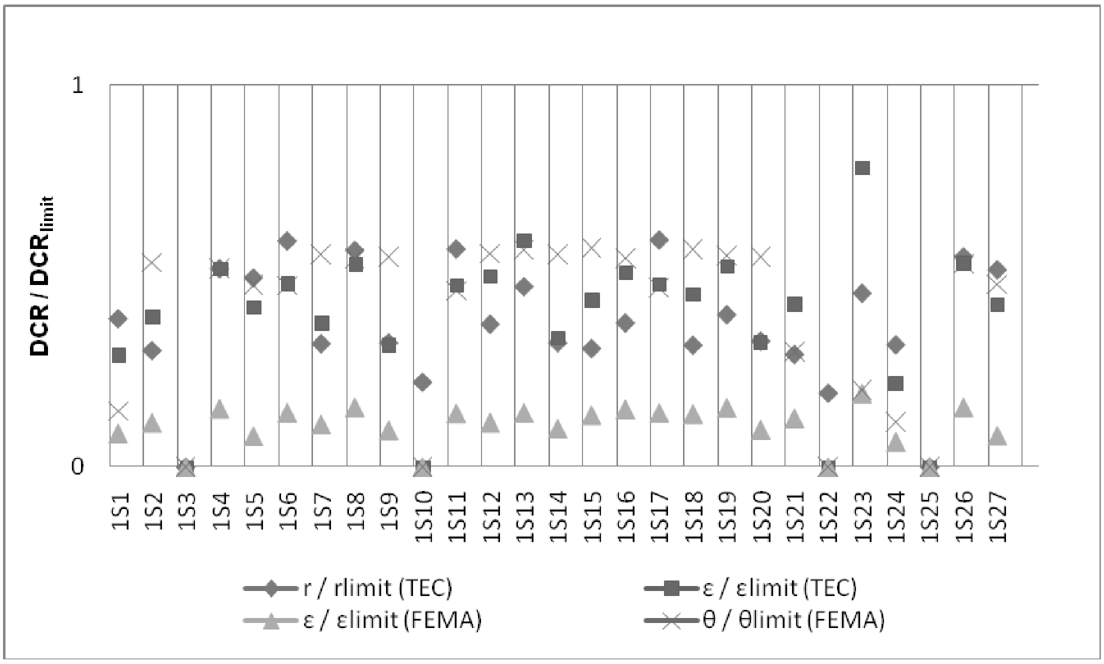

Figure 6.17 Performance Results of 1st Storey Columns for Re-designed Building (-X Direction)

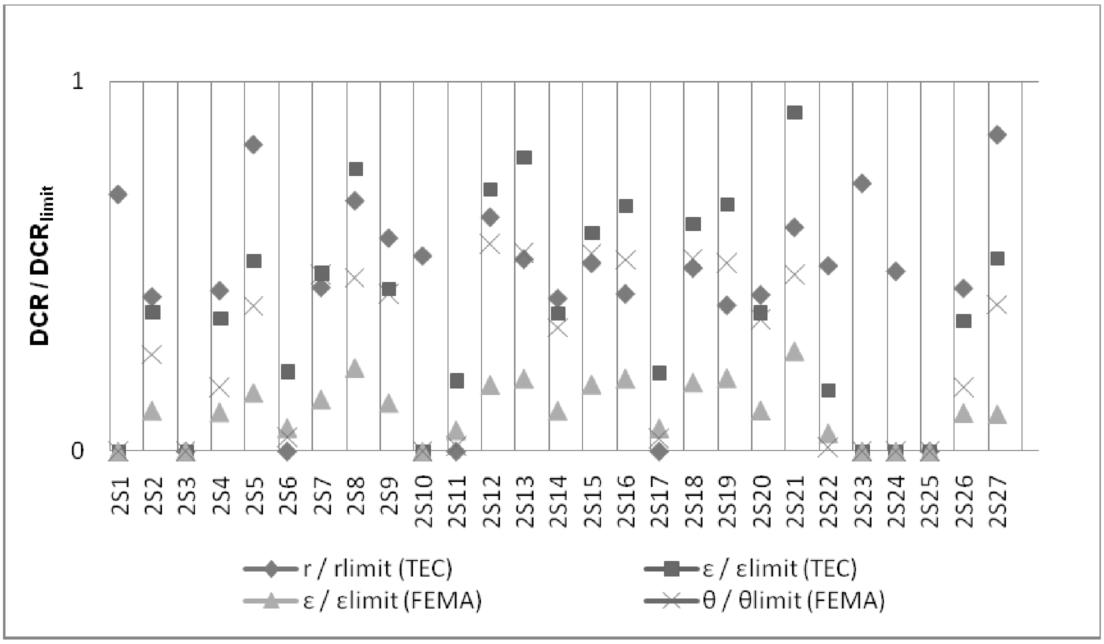

Figure 6.18 Performance Results of 2nd Storey Columns for Re-designed Building (-X Direction)

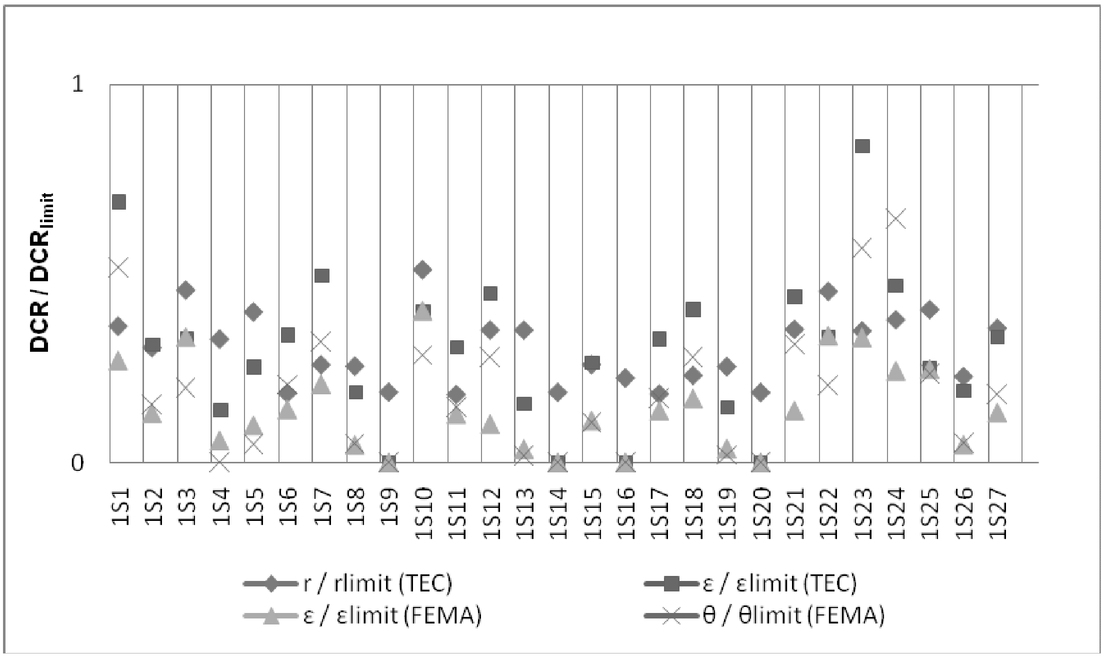

Figure 6.19 Performance Results of 1st Storey Columns for Re-designed Building (+Y Direction)

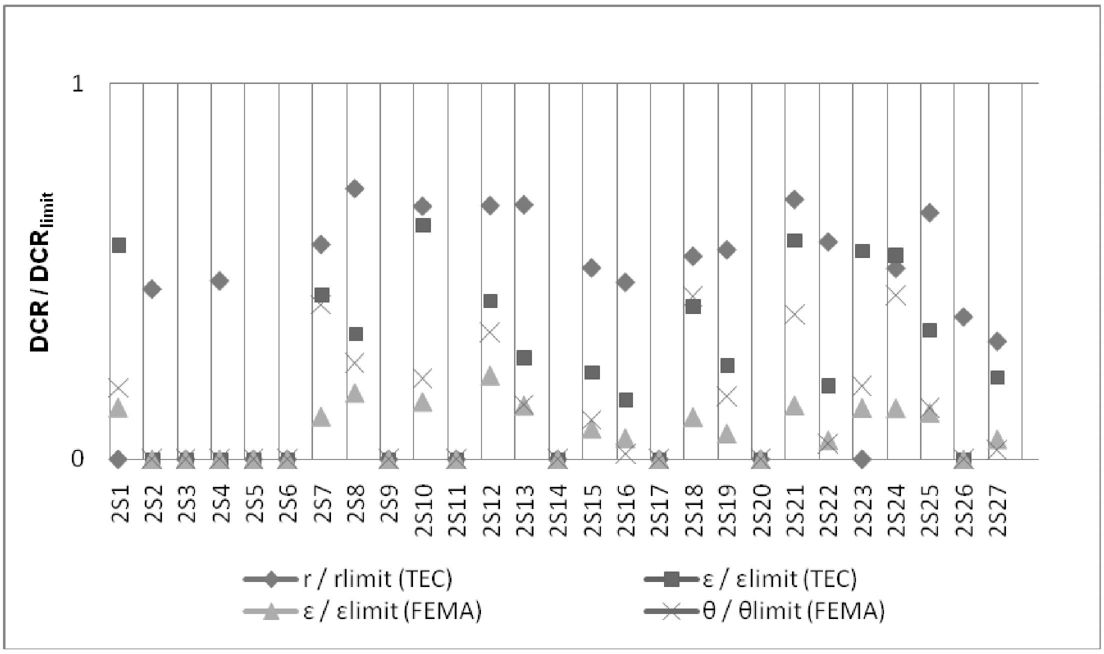

Figure 6.20 Performance Results of 2nd Storey Columns for Re-designed Building (+Y Direction)

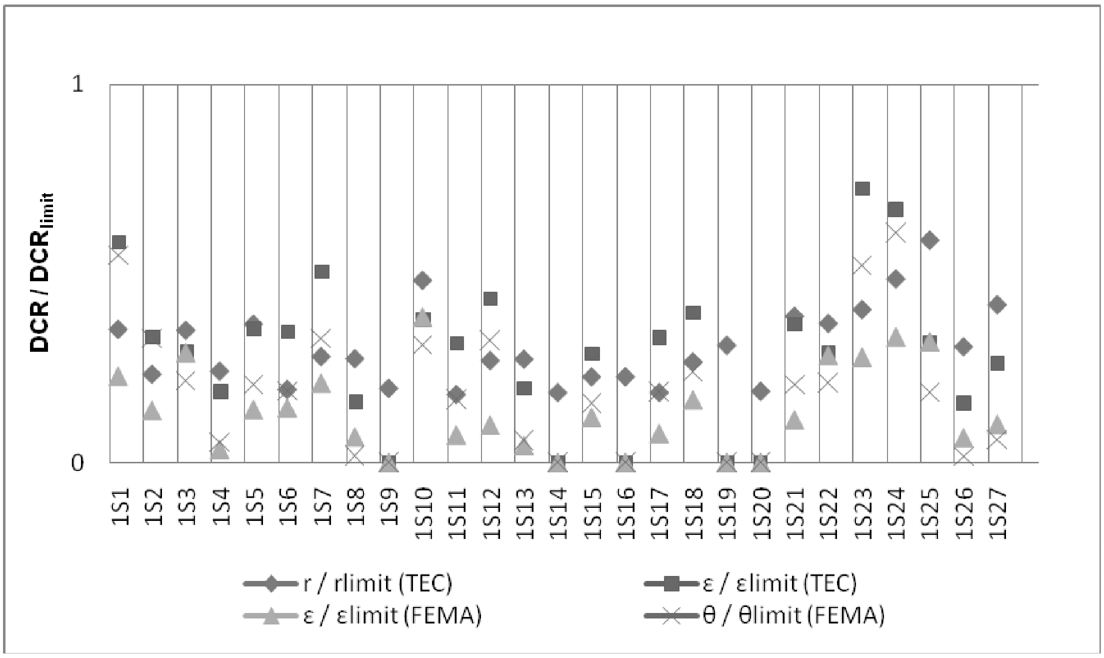

Figure 6.21 Performance Results of 1st Storey Columns for Re-designed Building (-Y Direction)

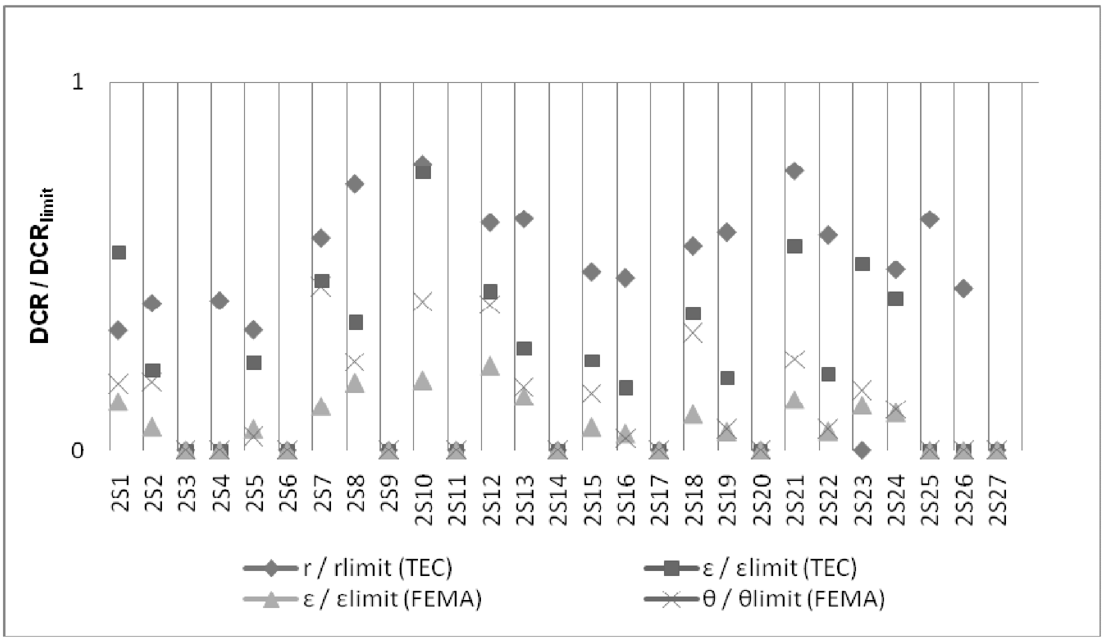

Figure 6.22 Performance Results of 2nd Storey Columns for Re-designed Building (-Y Direction)

Graphical procedure of column capacity calculation underestimates the tensile force in the column. Figure 6.19 shows the capacity calculation of 1S1 column of re-designed building. From Table 6.7 it can be seen that column is under tension for the +X direction. Although there is large tensile force in the column, the graphical procedure results in a satisfactory moment capacity despite that the tensile axial force due to earthquake plus gravity loading exceeds the capacity (Figure 6.23, Table 6.8). In other words, this procedure is not reliable for the columns which are under tension. This is a major drawback of the graphical procedure.

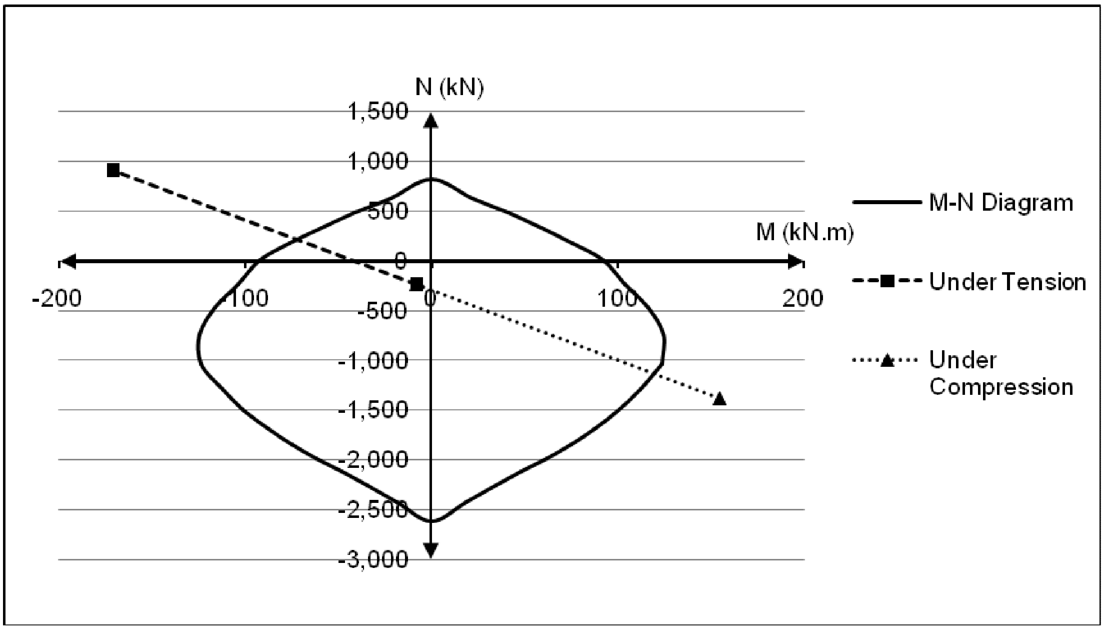

Figure 6.23 Graphical Procedure of Column Capacity Calculation

Table 6.8 Capacity of the Column 1S1 for +X and –X Directions

		Nd (kN)	Md (kN.m)	Ne (kN)	Me (kN.m)	Nk (kN)	Mk (kN.m)
1S1	+X Direction	-232.34	-8.18	1144.94	-162.87	209.69	-71.06
	-X Direction	-232.34	-8.18	-1144.94	162.87	-1165.08	124.51

6.2 Conclusions

From the results of three case study buildings that represent retrofitted, properly designed and vulnerable buildings, the following conclusions can be drawn.

- For nonlinear assessment, TEC 2007 is more conservative than FEMA 356 procedure. For buildings that do not comply the code (having unconfined sections), FEMA 356 provides the most unconservative results. For properly designed building, the strain limits given in TEC 2007 are too high making the procedure unconservative.
- The effect of axial load on the limit values is significant. For first story columns TEC 2007 nonlinear yields similar results compared to TEC 2007 linear whereas in second story it yields more conservative results.
- In this study it is observed that. nonlinear assessment is more conservative when the building is more flexible and goes into inelastic range, while linear assessment gives larger demand to capacity ratios for the rigid direction as the building behaves nearly elastic.
- Graphical calculation of column capacity for linear elastic analysis can be simplified by dividing the graph at least 4 or 6 parts and assuming each part as linear. Besdies, for the columns under tension this procedures appears to be unconservative and not too reliable.
- The properly designed building that satisfies the requirements of the code shows adequate performance for life safety performance level.
- A suitable and reasonable retrofit alternative results in satisfactory performance regardless of the assessment procedure.
- The performance limits for both linear and nonlinear procedures seem to be very conservative and can be adjusted to result in more economical decisions regarding the existing buildings.

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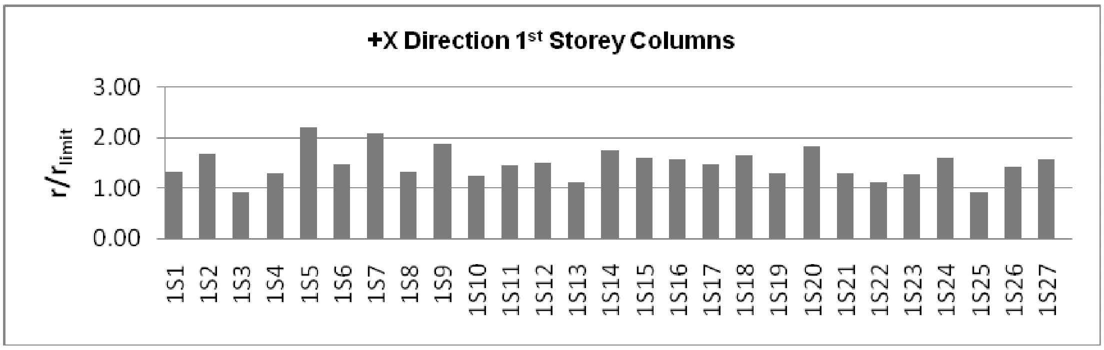
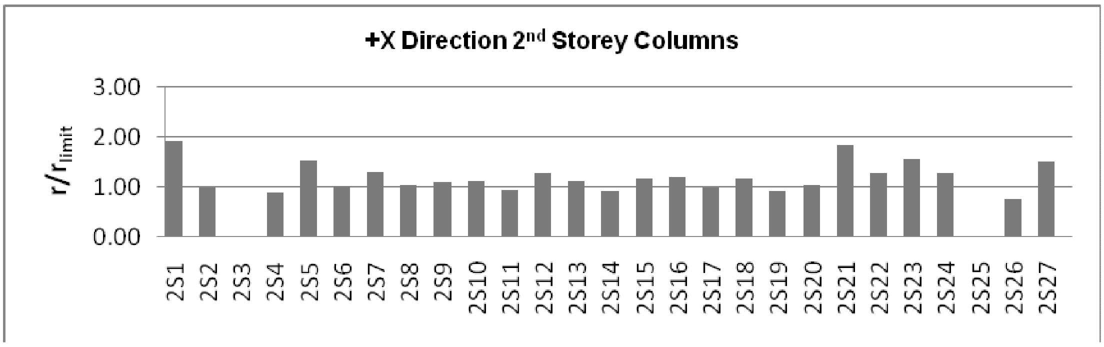
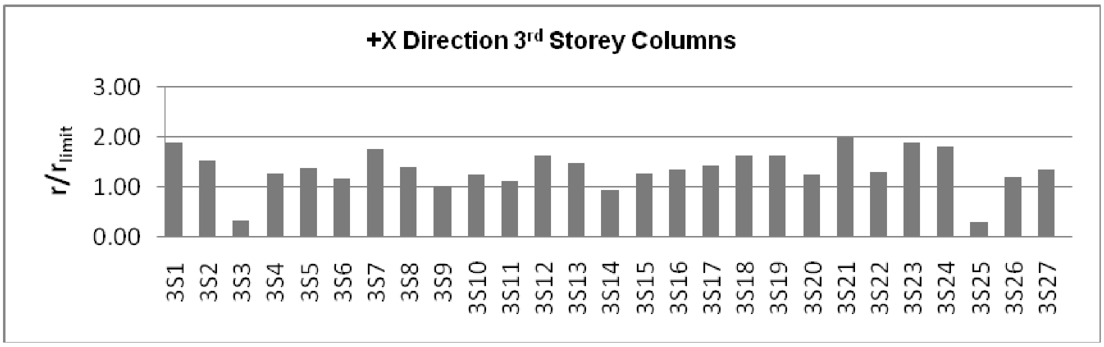
APPENDIX A

ASSESSMENT RESULTS OF EXISTING BUILDING







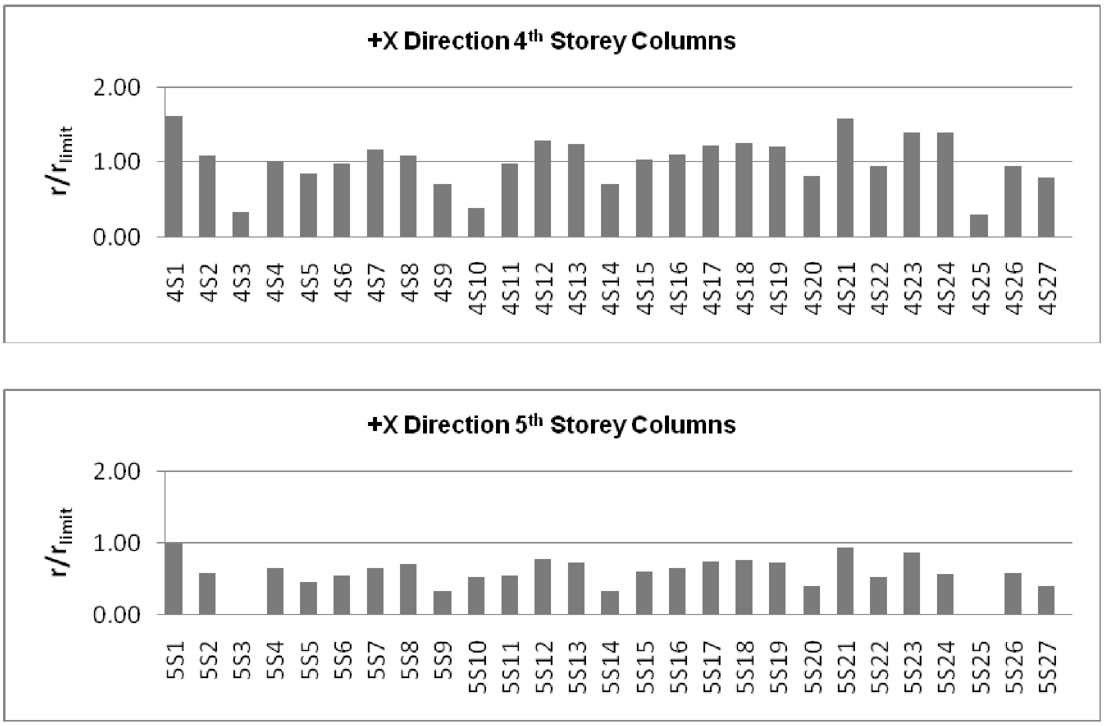
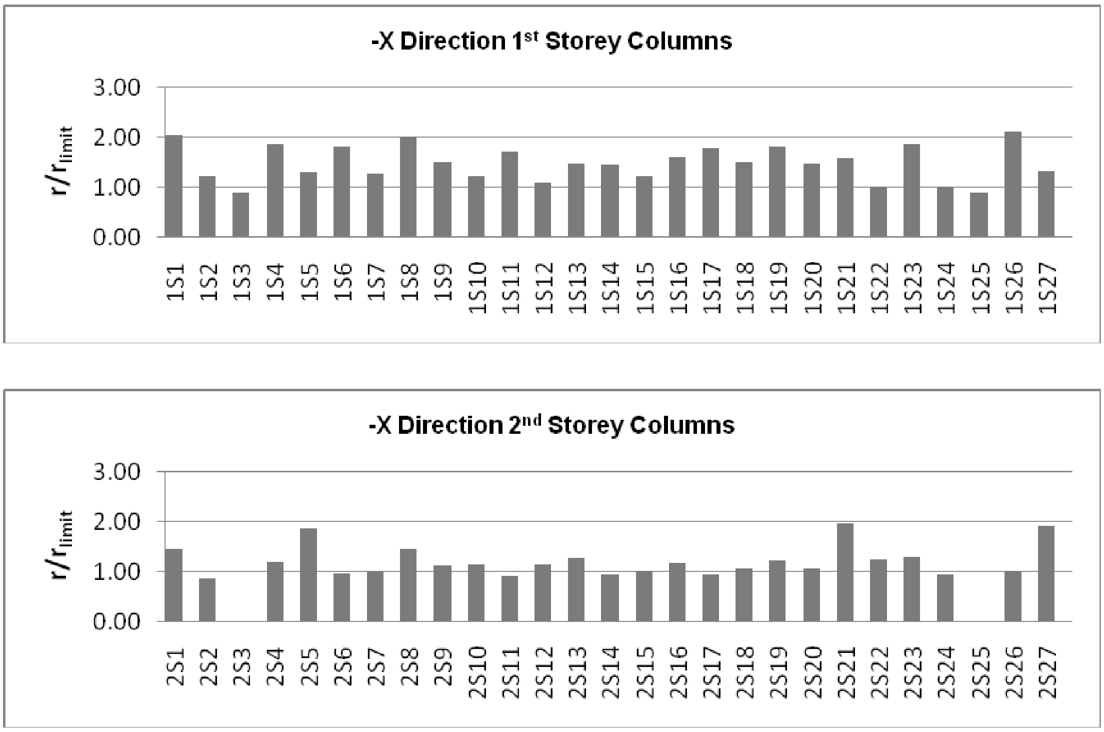
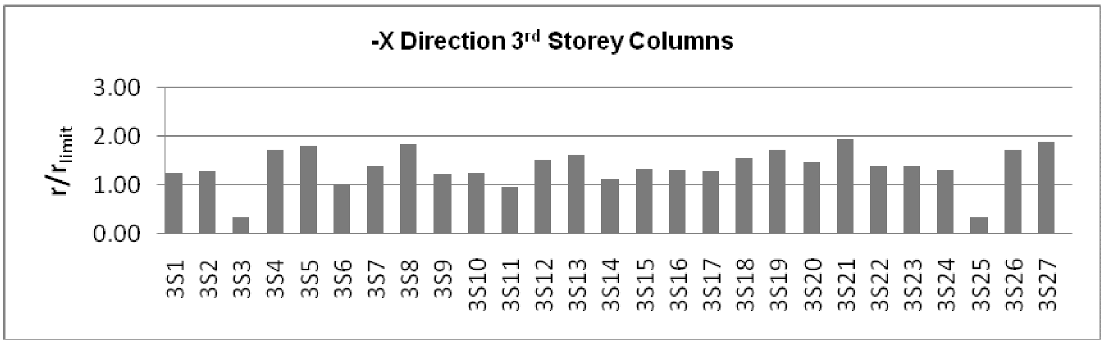
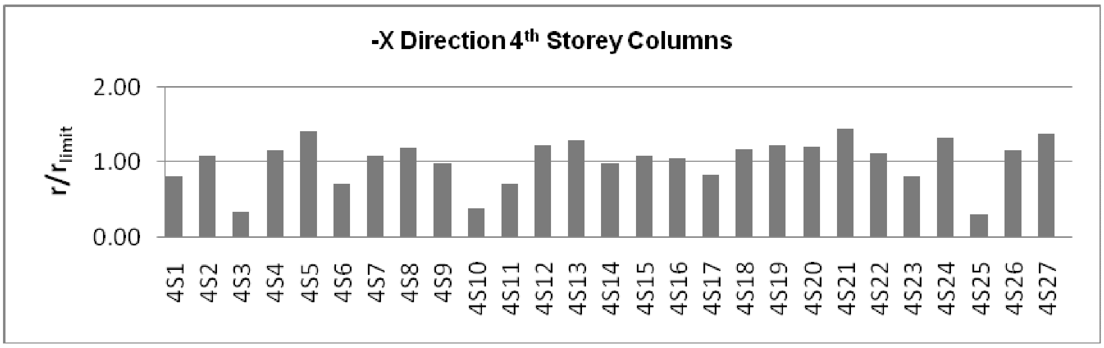


Figure A.1 r / r_{limit} for Columns (+X direction)







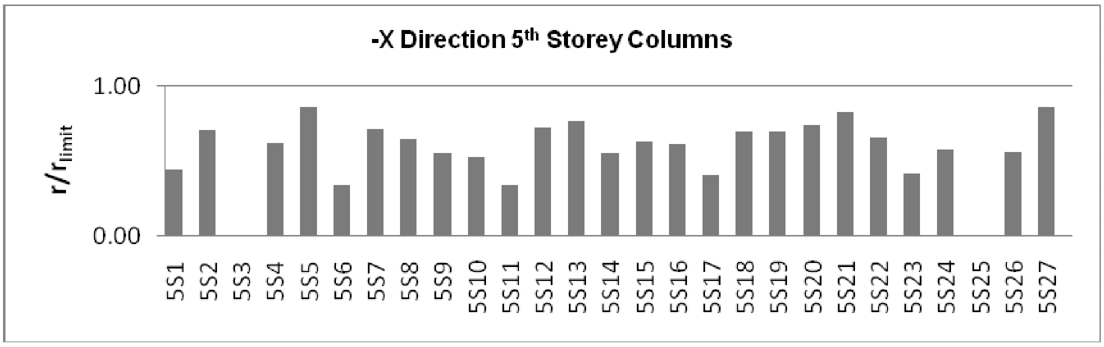
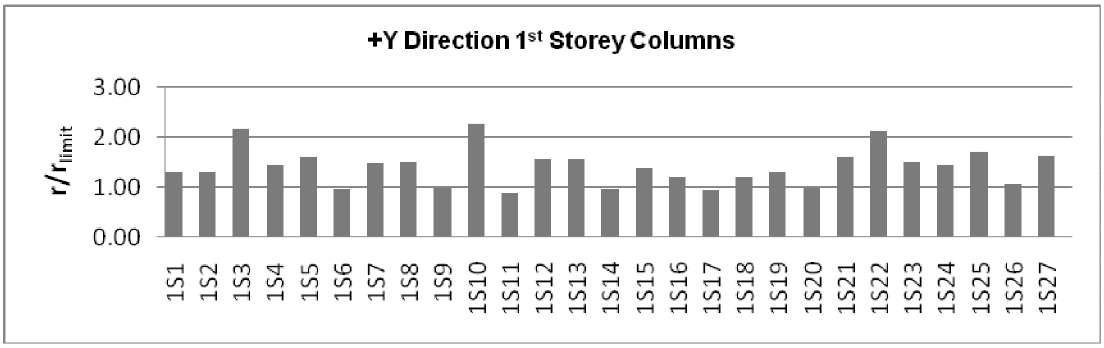


Figure A.2 r / r_{limit} for Columns (-X direction)



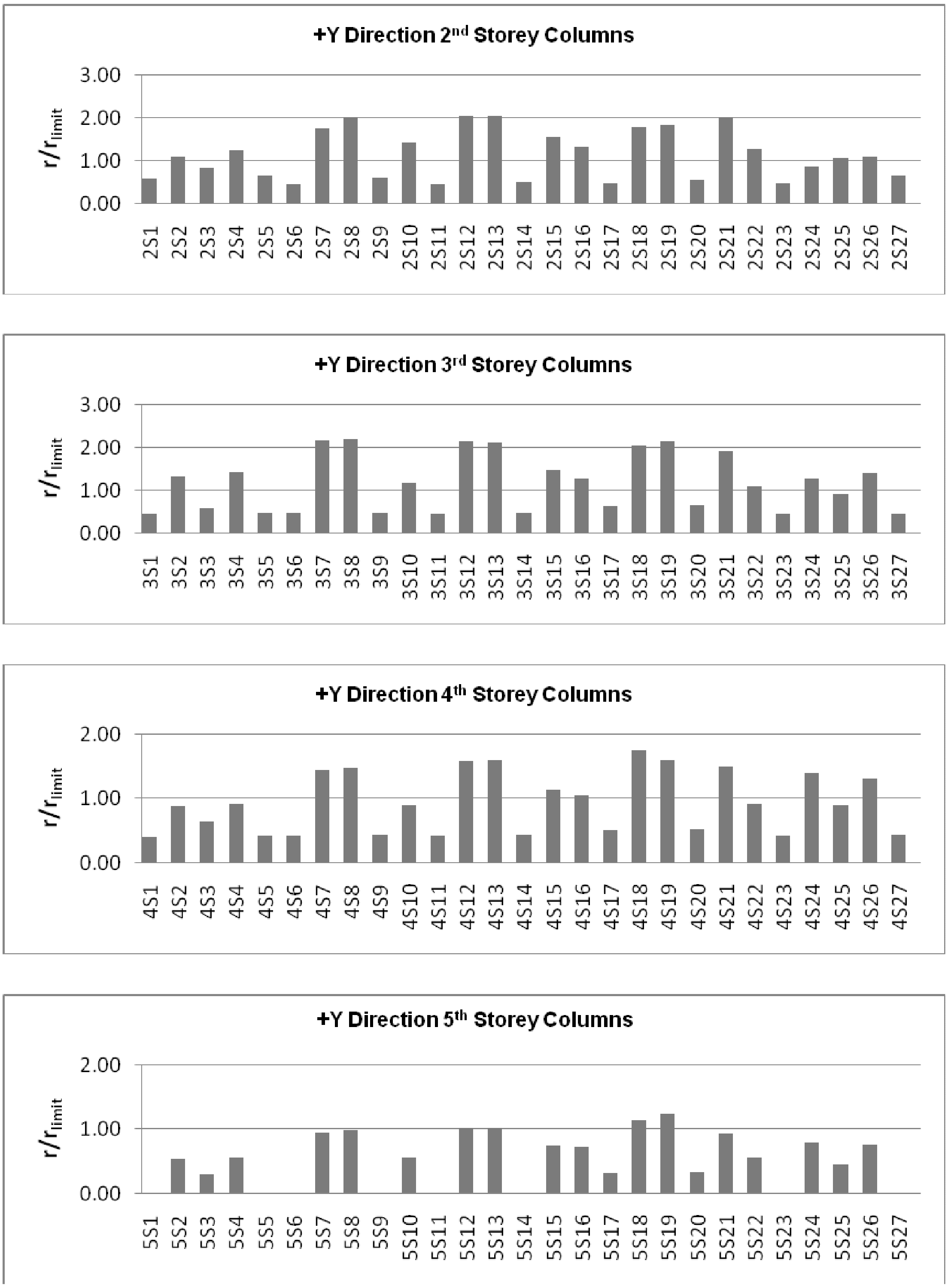
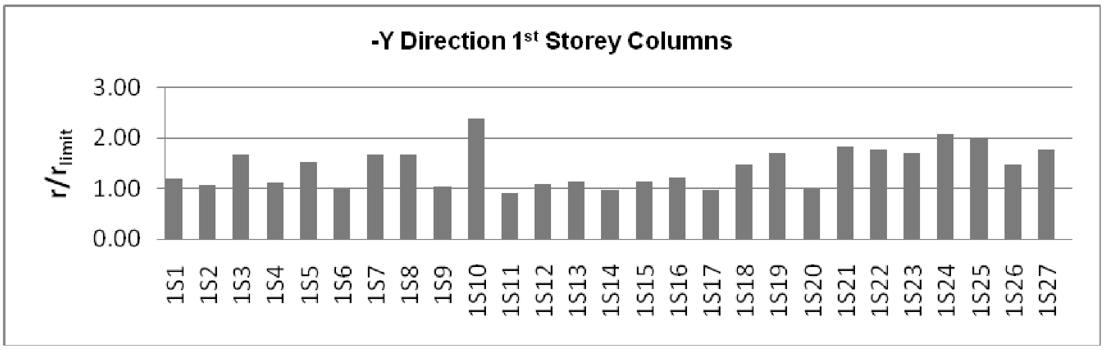
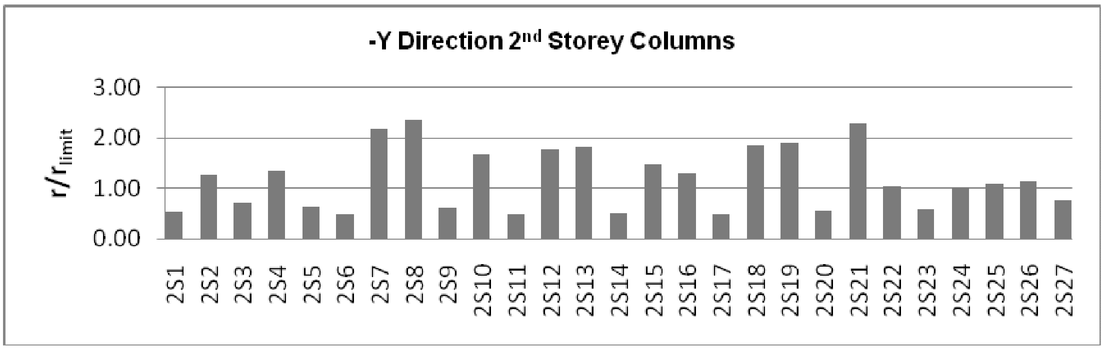


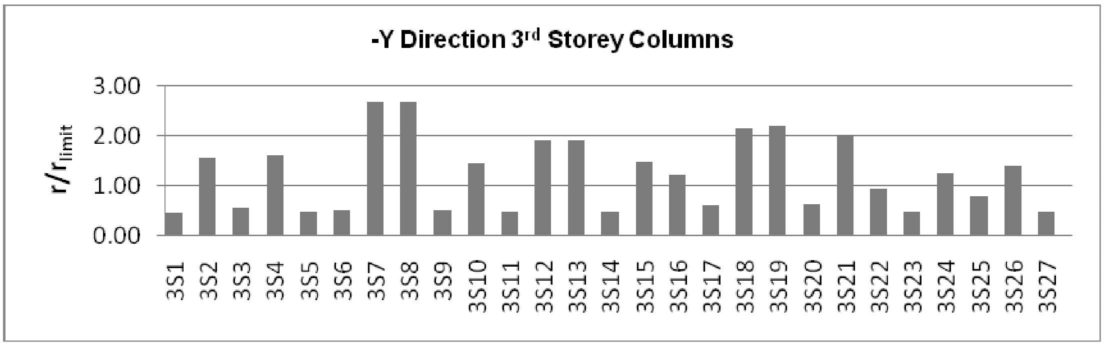
Figure A.3 r / r_{limit} for Columns (+Y direction)



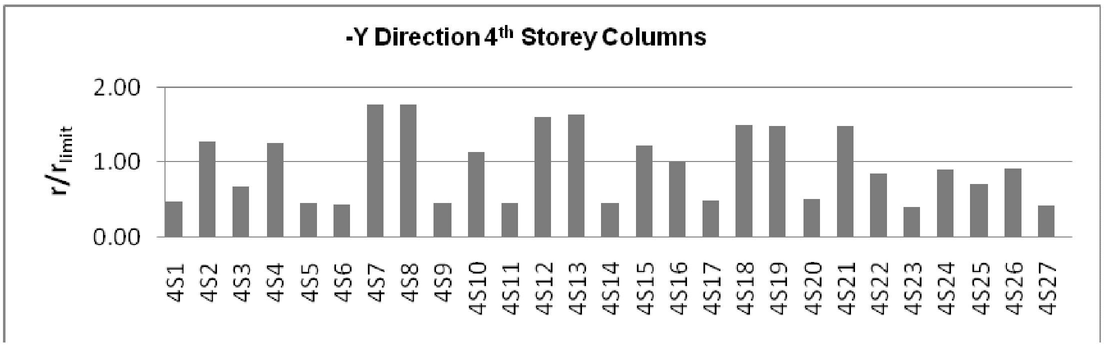
-Y Direction 1st Storey Columns
3.00
2.00
1.00
0.00
r/r_limit
1S1 1S2 1S3 1S4 1S5 1S6 1S7 1S8 1S9 1S10 1S11 1S12 1S13 1S14 1S15 1S16 1S17 1S18 1S19 1S20 1S21 1S22 1S23 1S24 1S25 1S26 1S27



-Y Direction 2nd Storey Columns
3.00
2.00
1.00
0.00
r/r_limit
2S1 2S2 2S3 2S4 2S5 2S6 2S7 2S8 2S9 2S10 2S11 2S12 2S13 2S14 2S15 2S16 2S17 2S18 2S19 2S20 2S21 2S22 2S23 2S24 2S25 2S26 2S27



-Y Direction 3rd Storey Columns
3.00
2.00
1.00
0.00
r/r_limit
3S1 3S2 3S3 3S4 3S5 3S6 3S7 3S8 3S9 3S10 3S11 3S12 3S13 3S14 3S15 3S16 3S17 3S18 3S19 3S20 3S21 3S22 3S23 3S24 3S25 3S26 3S27



-Y Direction 4th Storey Columns
2.00
1.00
0.00
r/r_limit
4S1 4S2 4S3 4S4 4S5 4S6 4S7 4S8 4S9 4S10 4S11 4S12 4S13 4S14 4S15 4S16 4S17 4S18 4S19 4S20 4S21 4S22 4S23 4S24 4S25 4S26 4S27

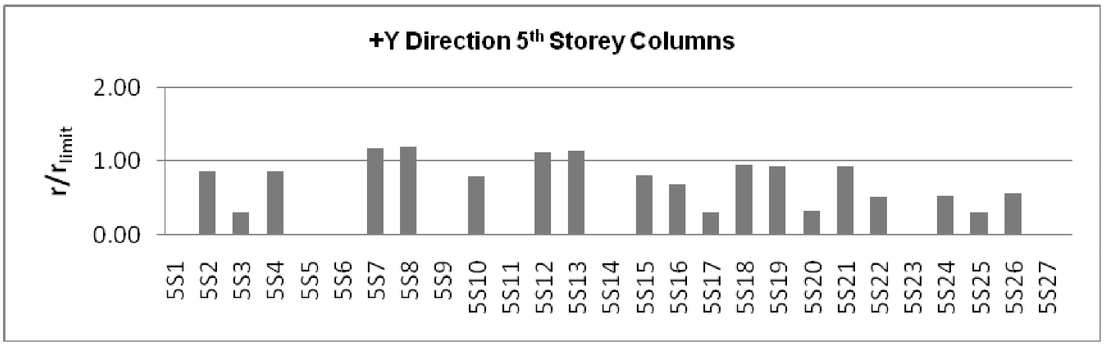
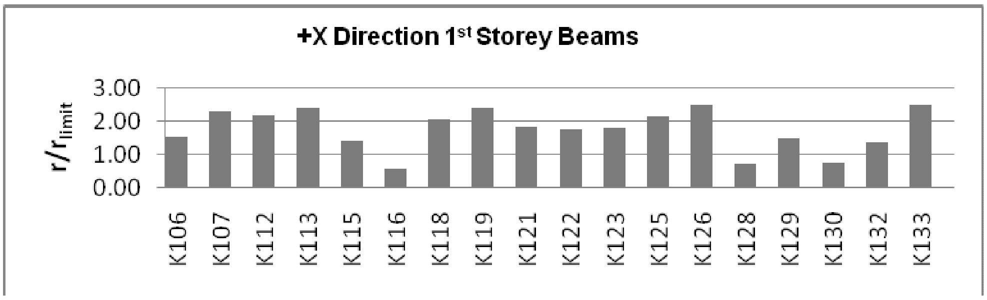
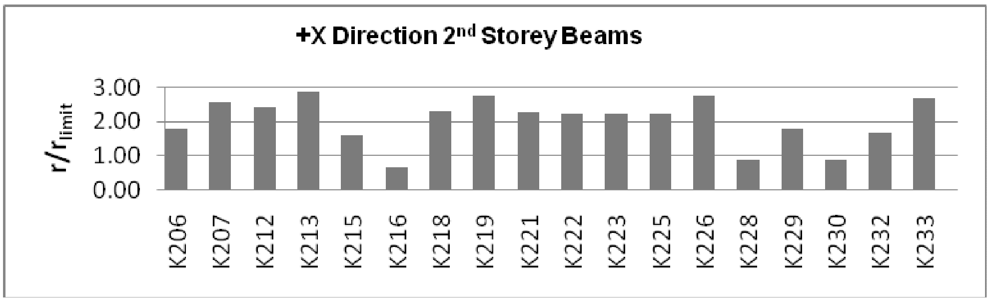
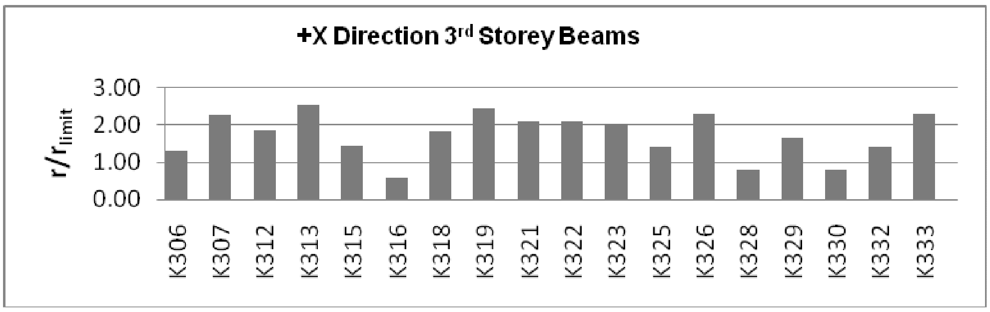
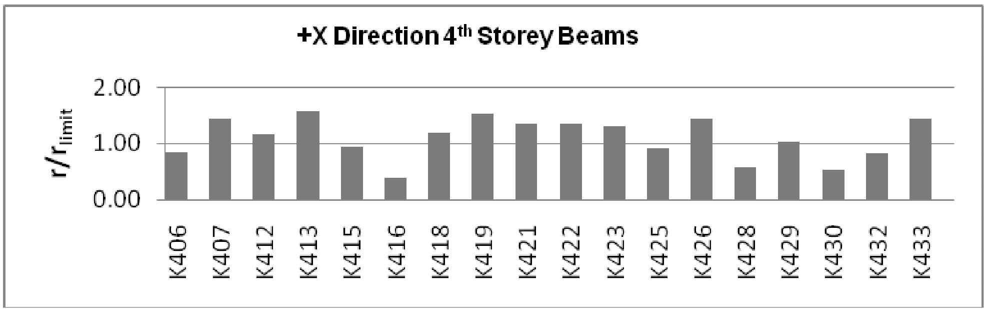


Figure A.4 r / r_{limit} for Columns (-Y direction)









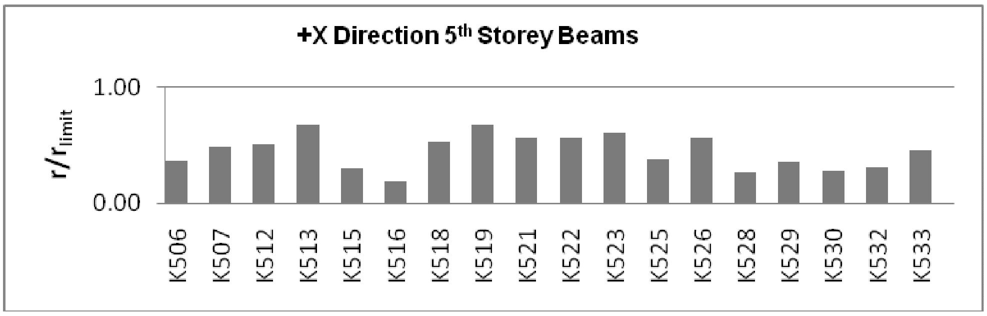
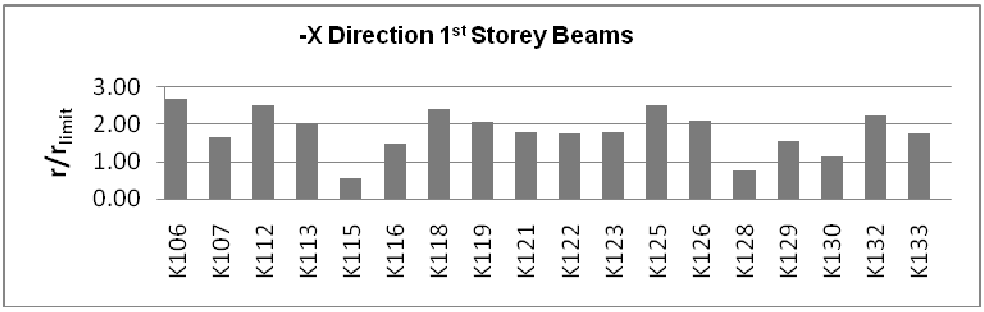
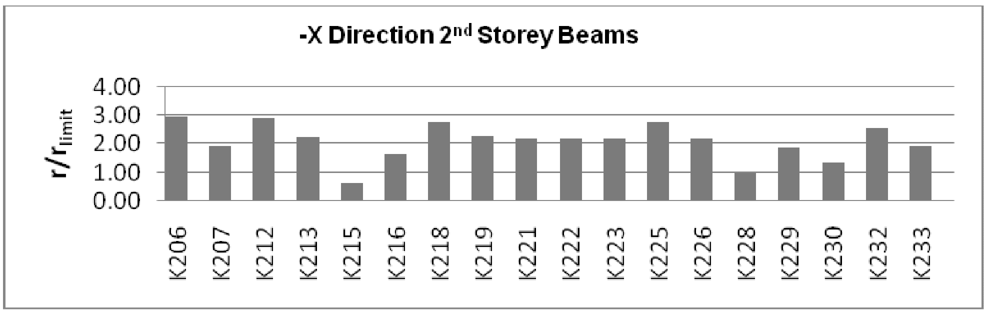
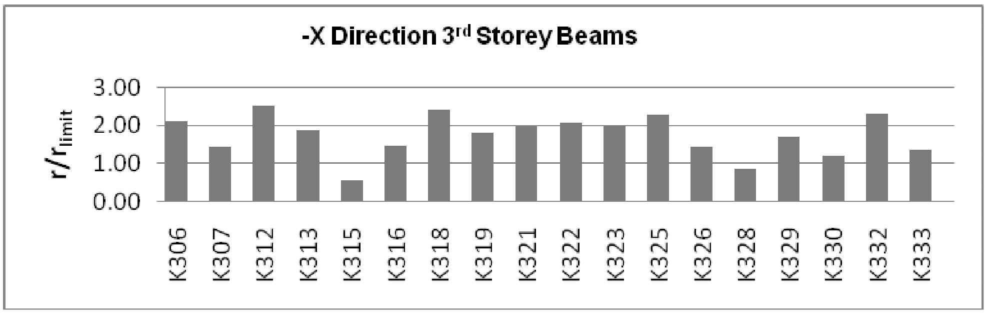
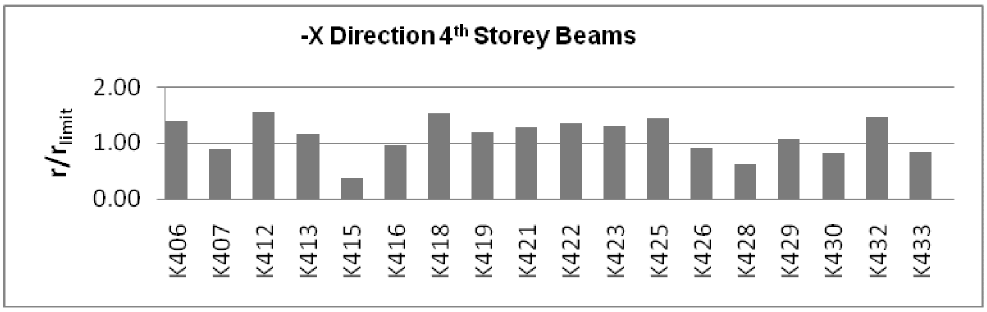


Figure A.5 r / r_{limit} for Beams (+X direction)









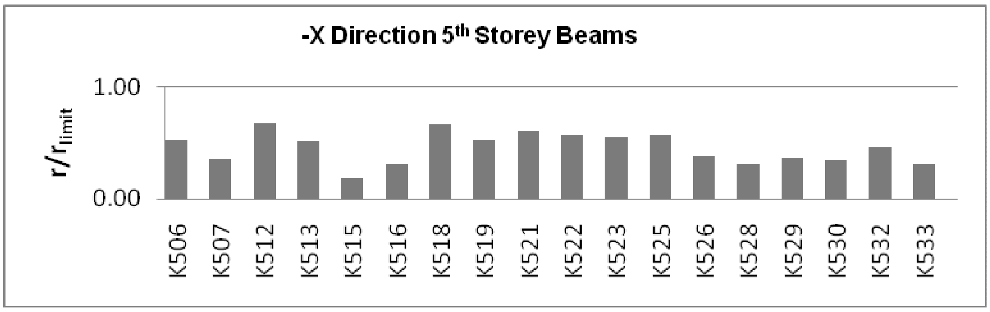
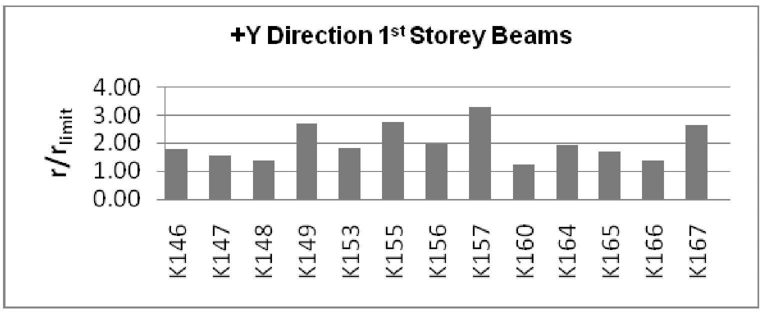


Figure A.6 r / r_{limit} for Beams (-X direction)



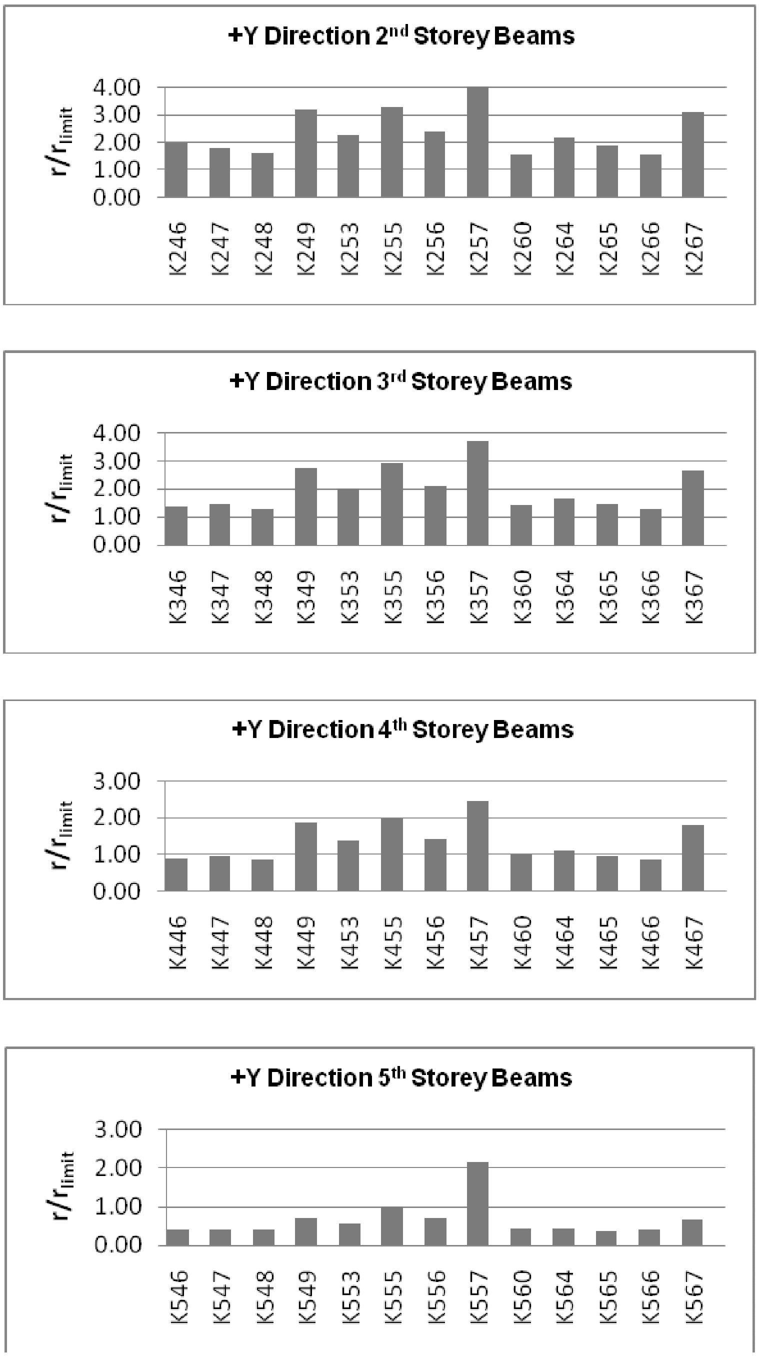


Figure A.7 r / r_{limit} for Beams (+Y direction)

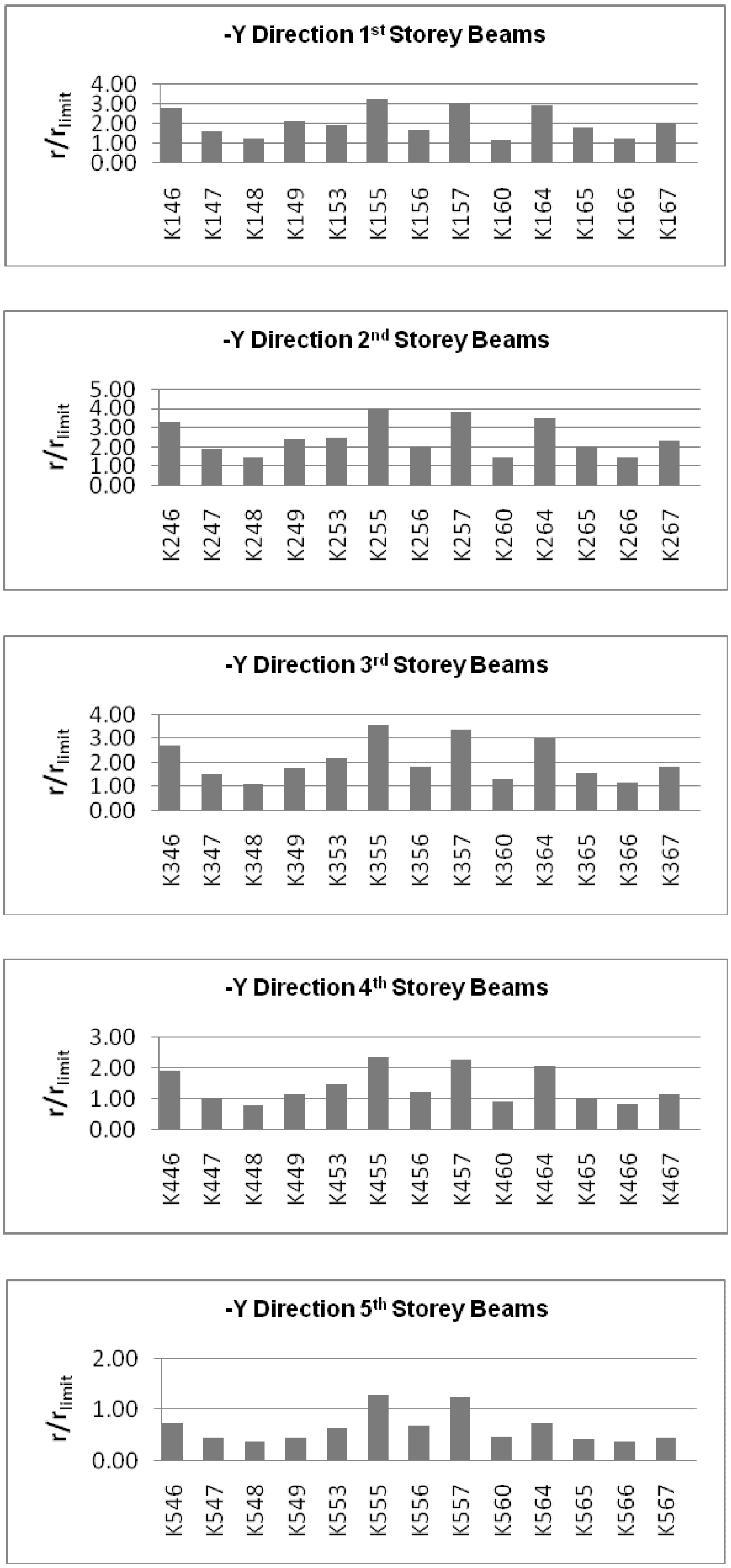
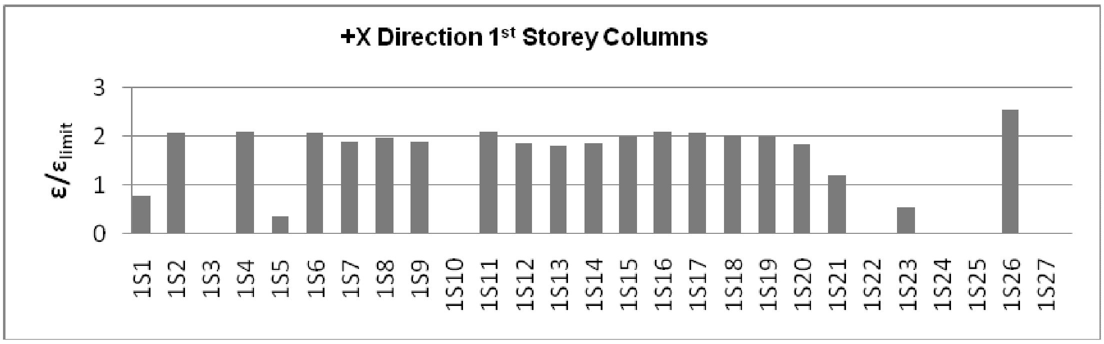
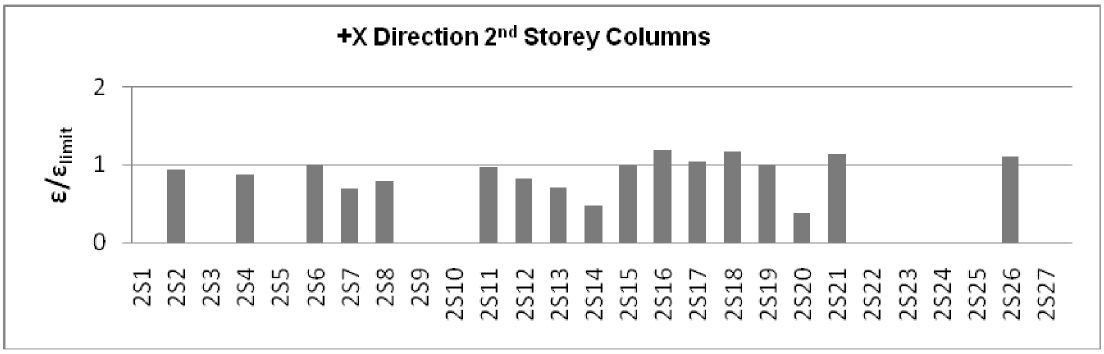
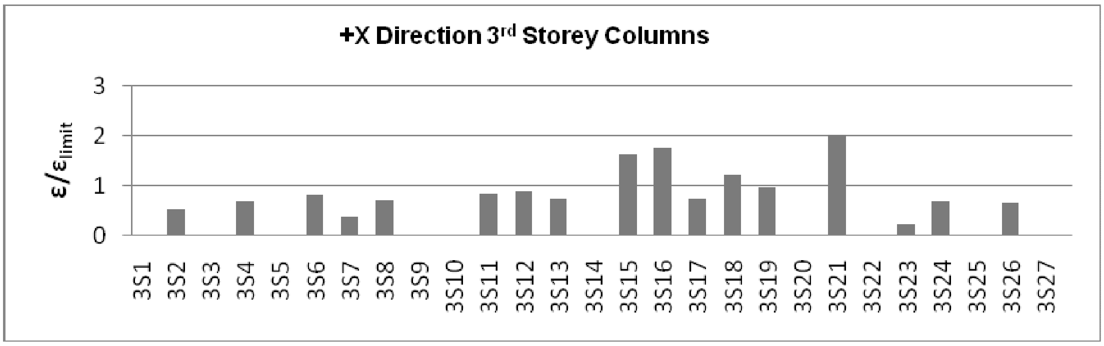
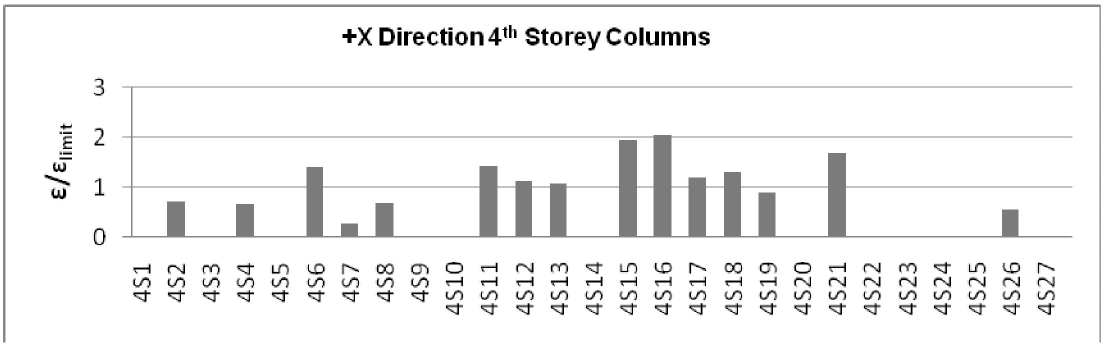


Figure A.8 r / r_{limit} for Beams (-Y direction)


+X Direction 1st Storey Columns
ε/ε_limit
0
1
2
3
1S1 1S2 1S3 1S4 1S5 1S6 1S7 1S8 1S9 1S10 1S11 1S12 1S13 1S14 1S15 1S16 1S17 1S18 1S19 1S20 1S21 1S22 1S23 1S24 1S25 1S26 1S27


+X Direction 2nd Storey Columns
ε/ε_limit
0
1
2
2S1 2S2 2S3 2S4 2S5 2S6 2S7 2S8 2S9 2S10 2S11 2S12 2S13 2S14 2S15 2S16 2S17 2S18 2S19 2S20 2S21 2S22 2S23 2S24 2S25 2S26 2S27


+X Direction 3rd Storey Columns
ε/ε_limit
0
1
2
3
3S1 3S2 3S3 3S4 3S5 3S6 3S7 3S8 3S9 3S10 3S11 3S12 3S13 3S14 3S15 3S16 3S17 3S18 3S19 3S20 3S21 3S22 3S23 3S24 3S25 3S26 3S27


+X Direction 4th Storey Columns
ε/ε_limit
0
1
2
3
4S1 4S2 4S3 4S4 4S5 4S6 4S7 4S8 4S9 4S10 4S11 4S12 4S13 4S14 4S15 4S16 4S17 4S18 4S19 4S20 4S21 4S22 4S23 4S24 4S25 4S26 4S27

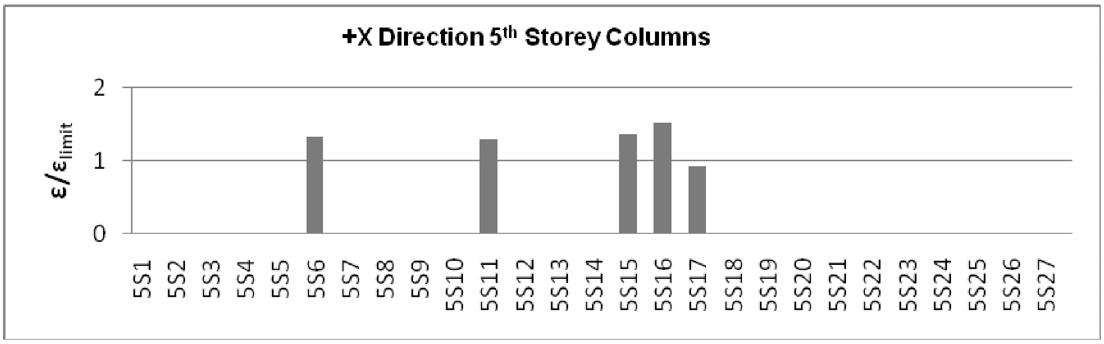
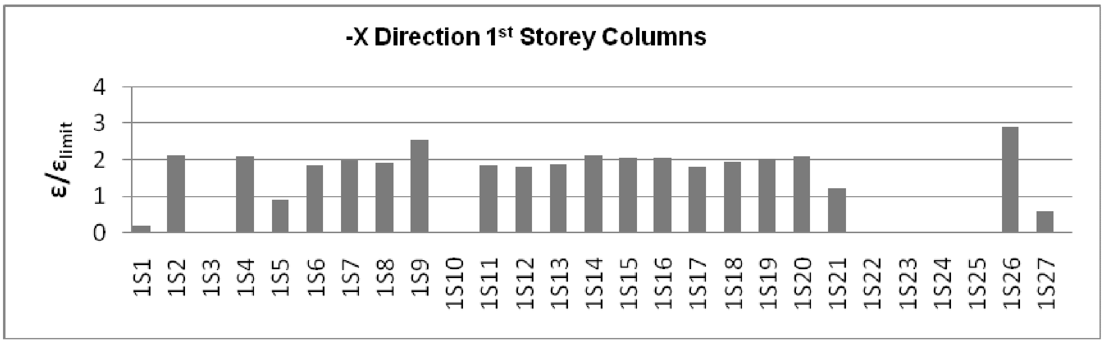
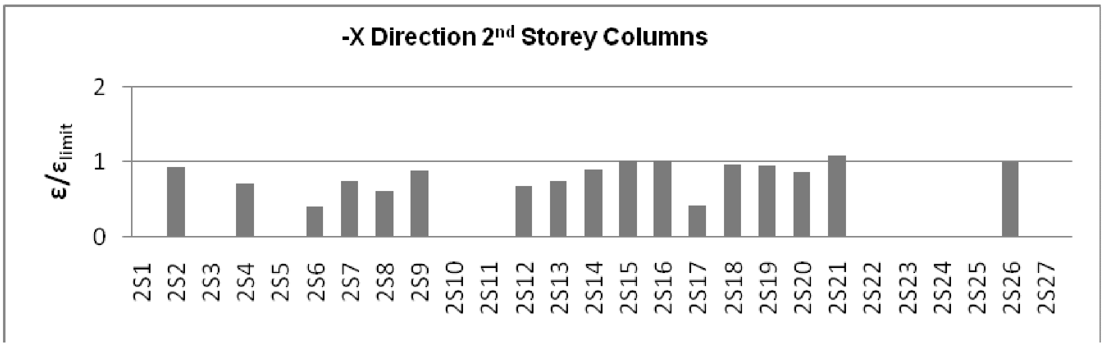
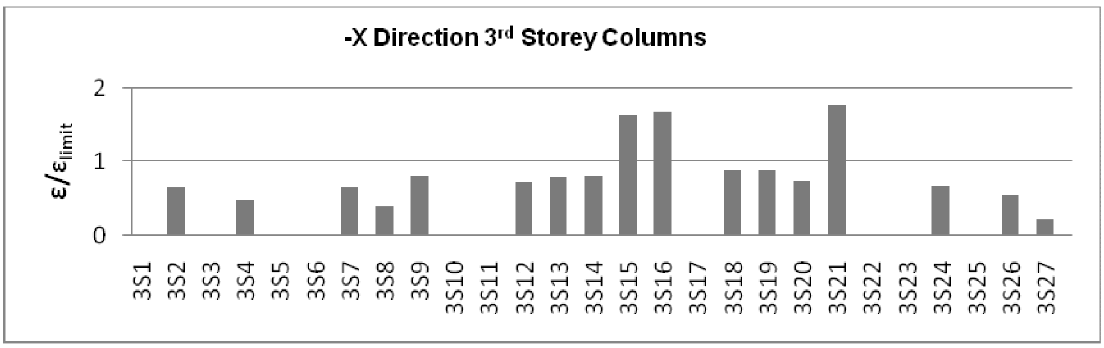


Figure A.9 ε / ε_{limit} for Columns (+X direction)







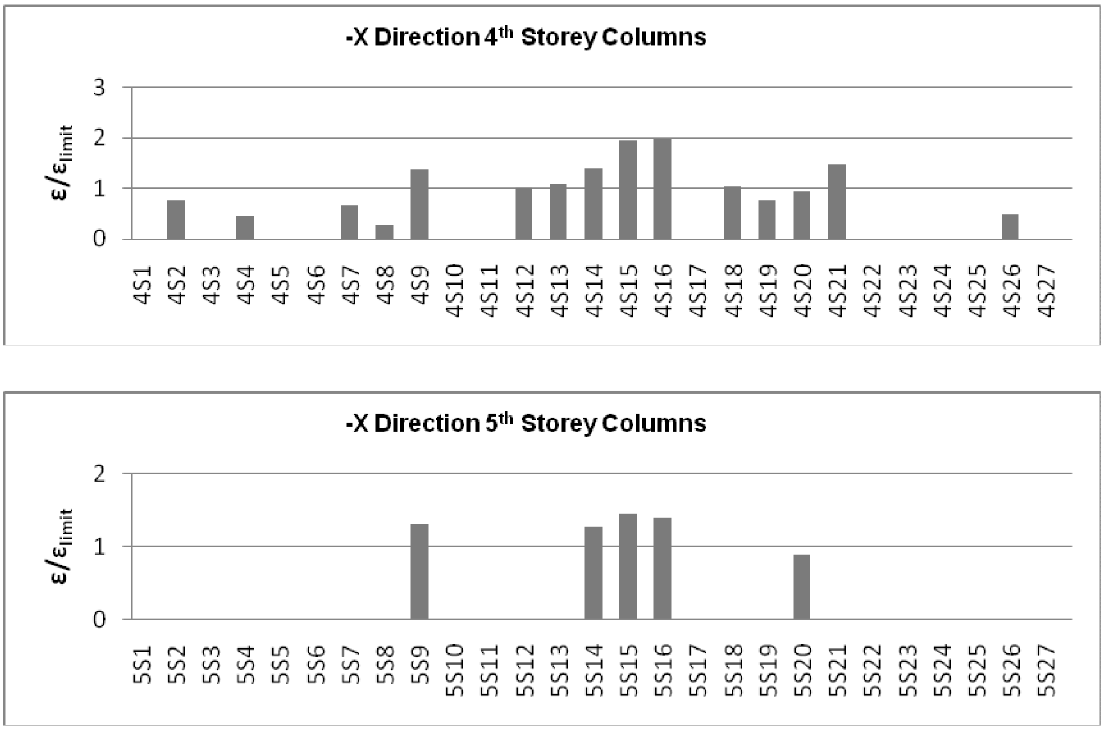
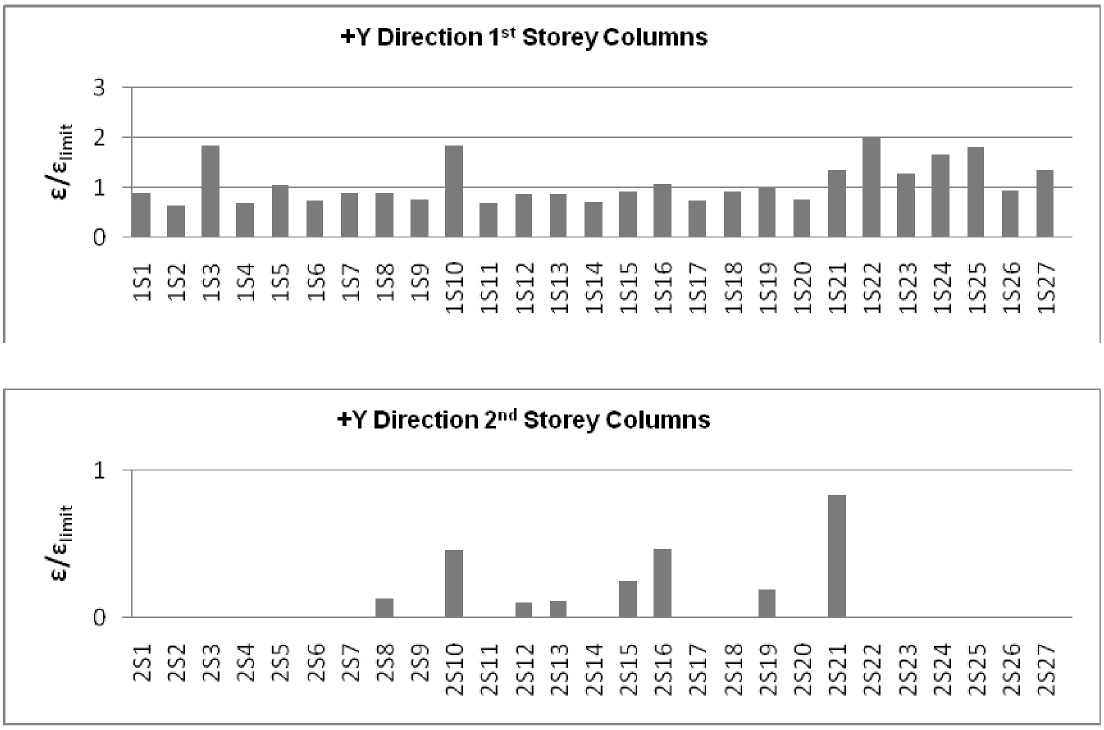
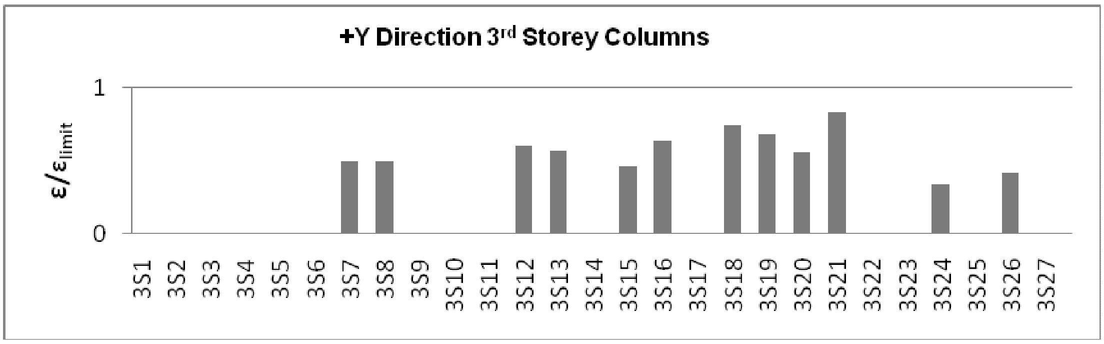
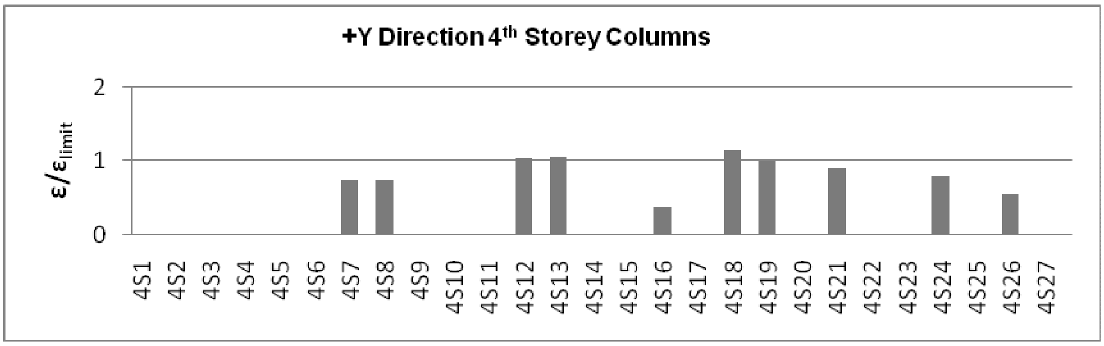


Figure A.10 ε / ε_{limit} for Columns (+X direction)







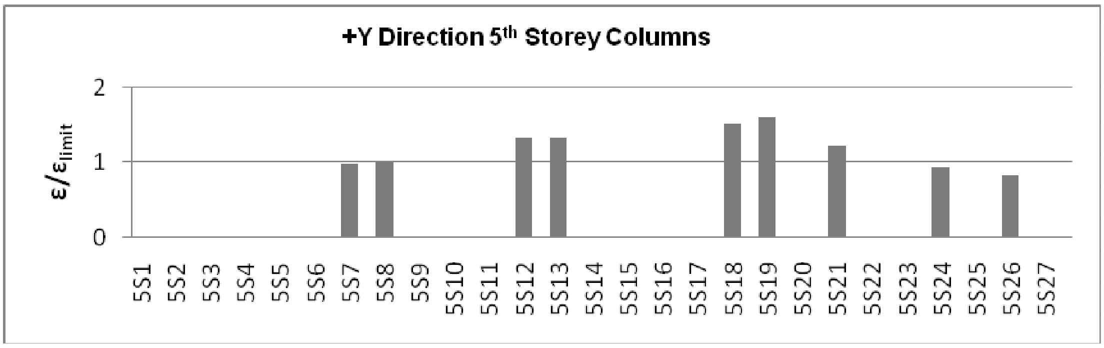
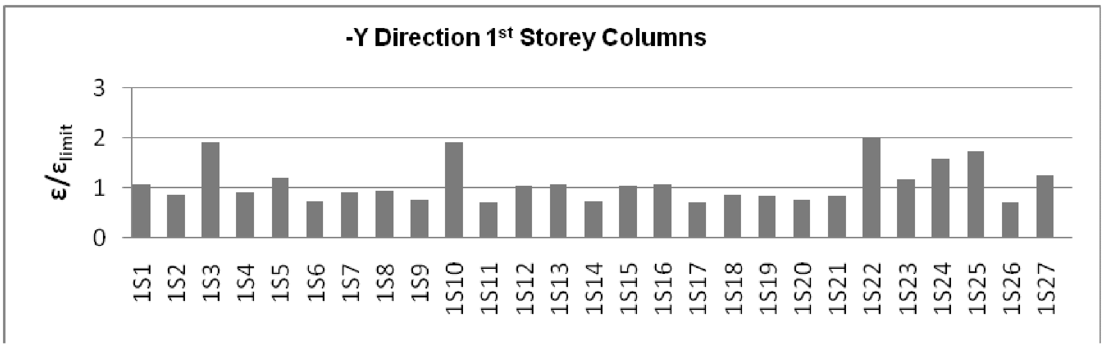


Figure A.11 ε / ε_{limit} for Columns (+Y direction)



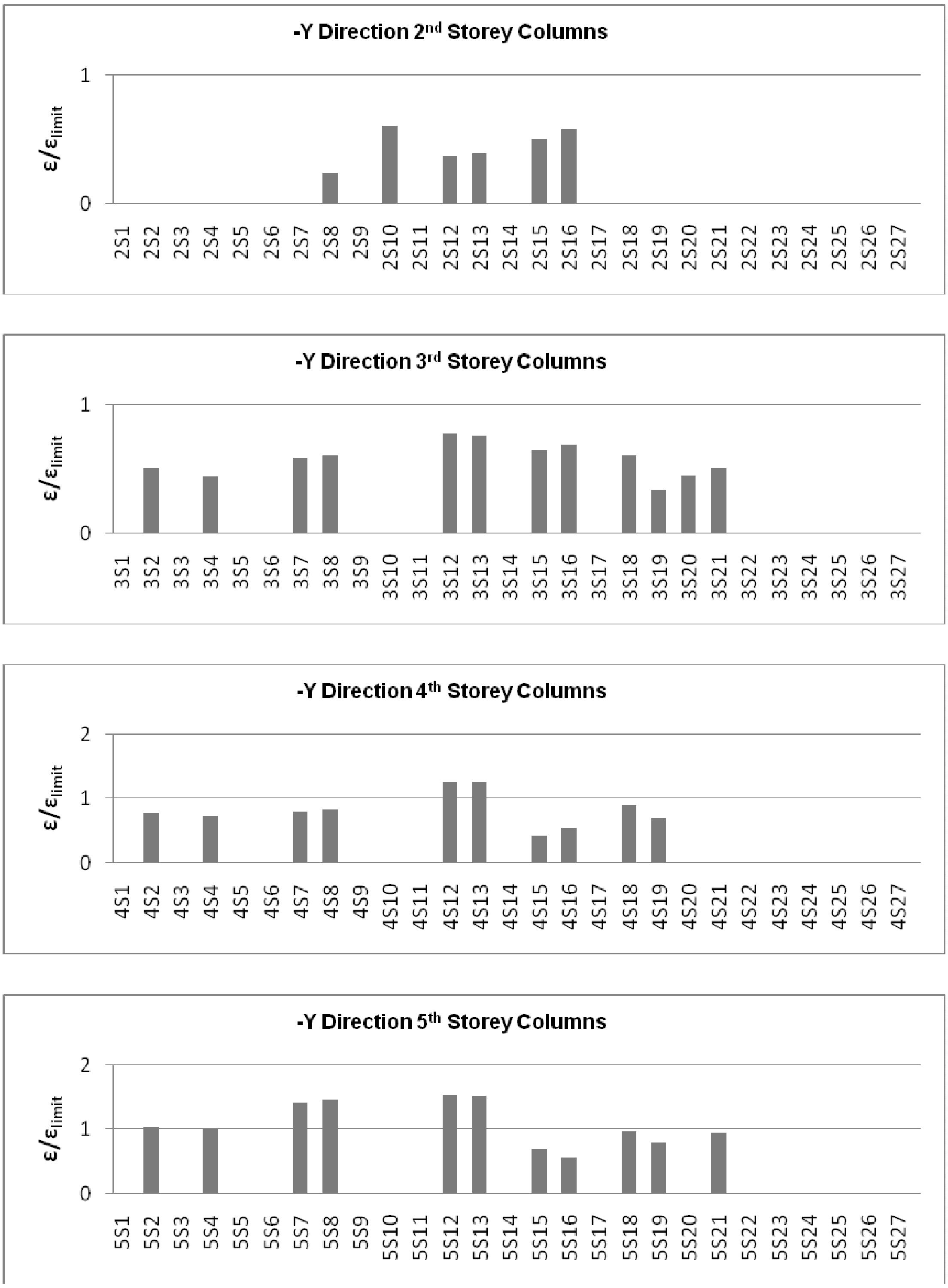
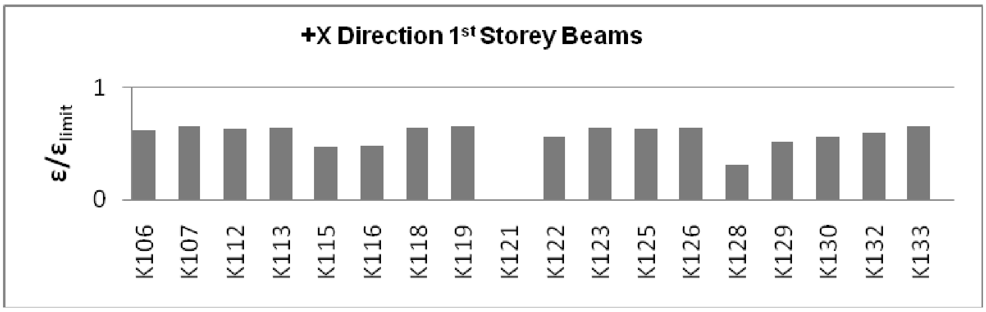
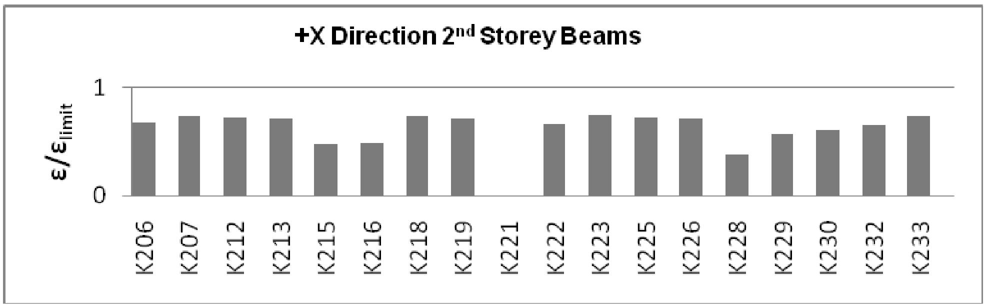
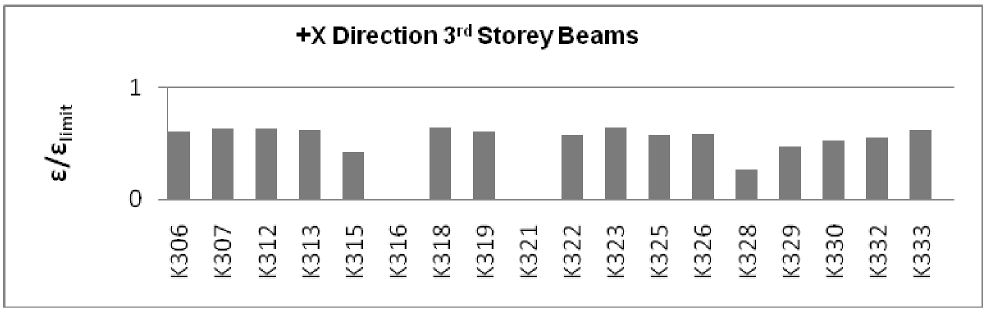
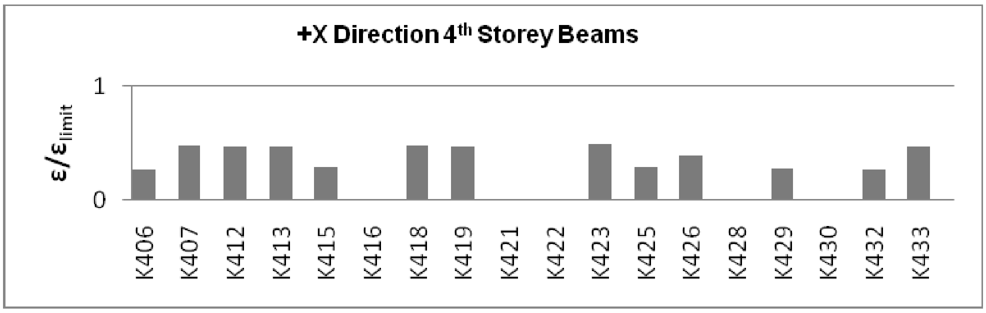


Figure A.12 ε / ε_{limit} for Columns (-Y direction)


+X Direction 1st Storey Beams
ε/εlimit
1
0
K106 K107 K112 K113 K115 K116 K118 K119 K121 K122 K123 K125 K126 K128 K129 K130 K132 K133


+X Direction 2nd Storey Beams
ε/εlimit
1
0
K206 K207 K212 K213 K215 K216 K218 K219 K221 K222 K223 K225 K226 K228 K229 K230 K232 K233


+X Direction 3rd Storey Beams
ε/εlimit
1
0
K306 K307 K312 K313 K315 K316 K318 K319 K321 K322 K323 K325 K326 K328 K329 K330 K332 K333


+X Direction 4th Storey Beams
ε/εlimit
1
0
K406 K407 K412 K413 K415 K416 K418 K419 K421 K422 K423 K425 K426 K428 K429 K430 K432 K433

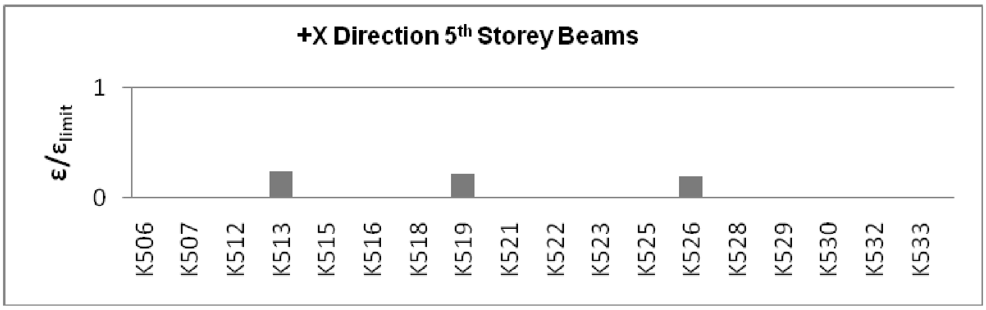
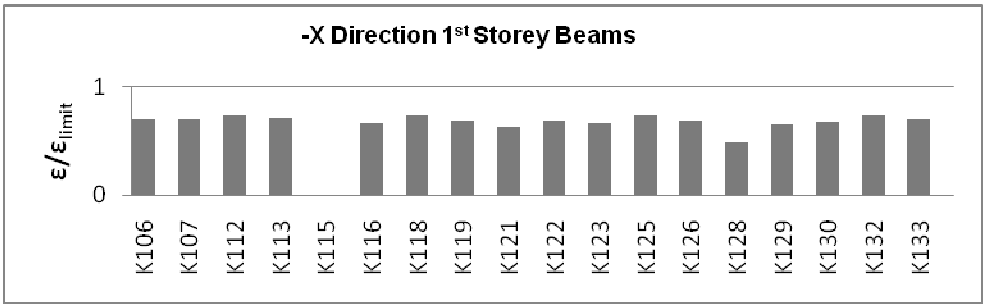
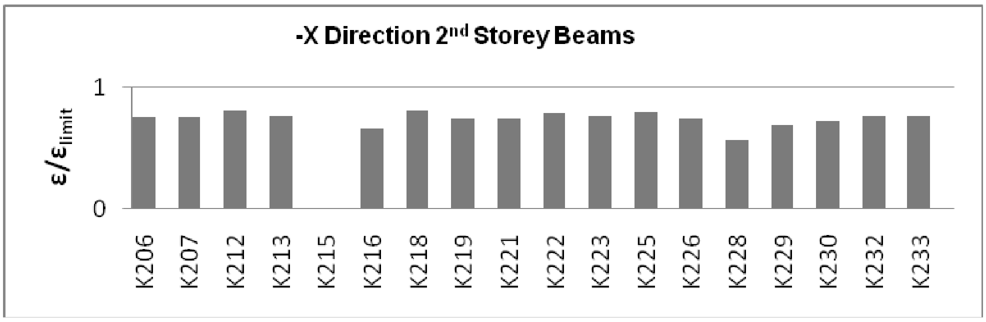
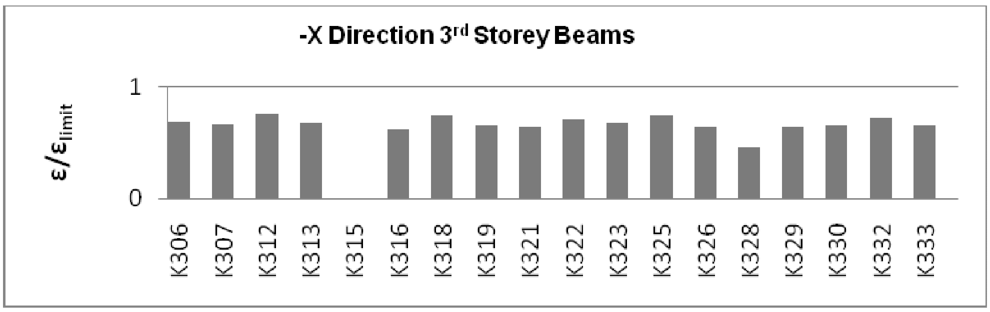


Figure A.13 ε / ε_{limit} for Beams (+X direction)







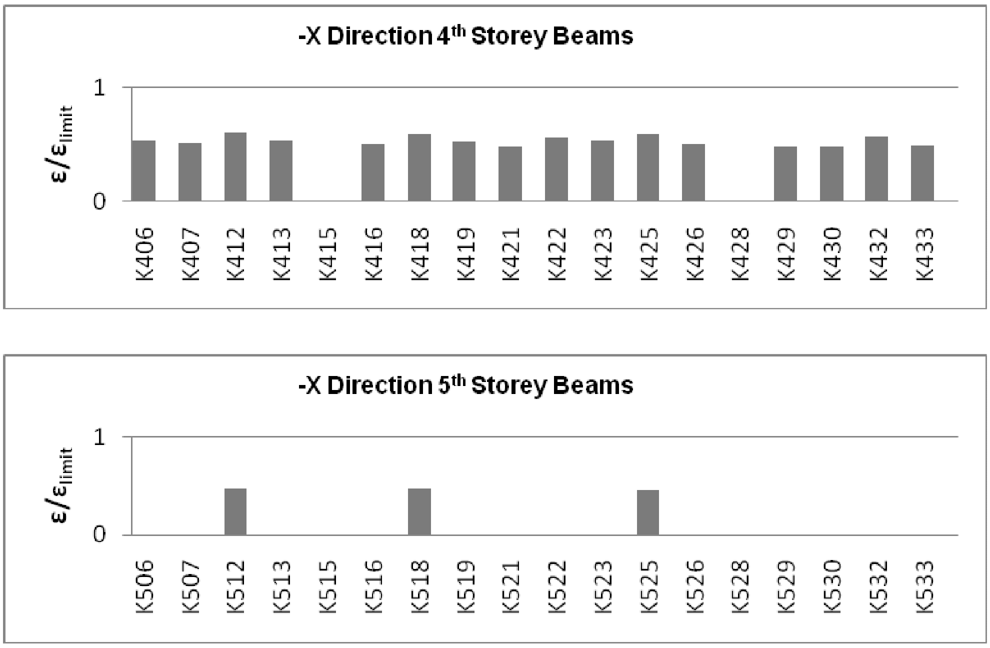
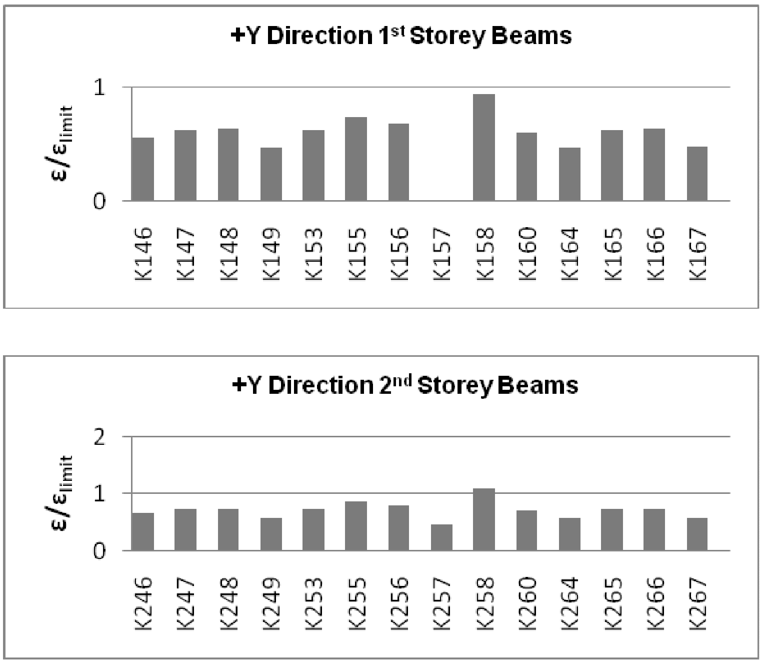


Figure A.14 ε / ε_{limit} for Beams (-X direction)



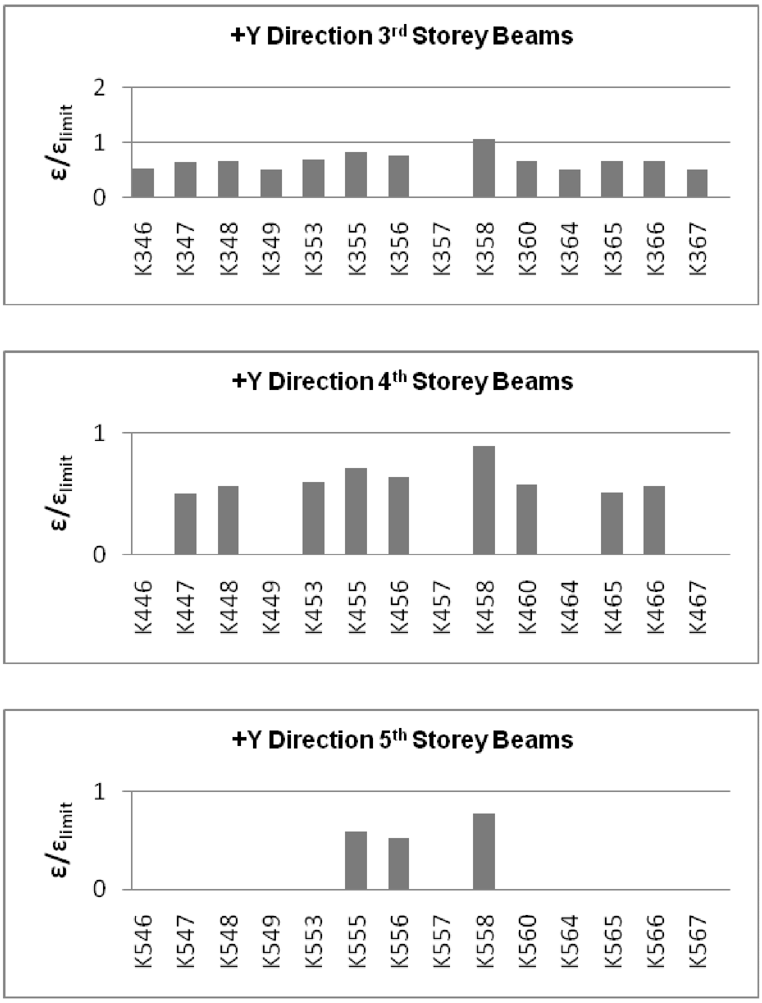
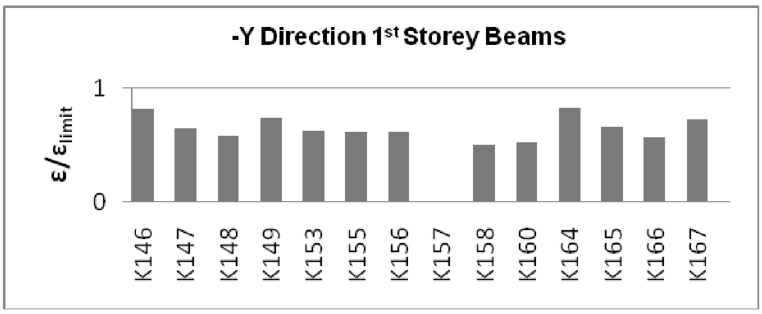


Figure A.15 ε / ε_{limit} for Beams (+Y direction)



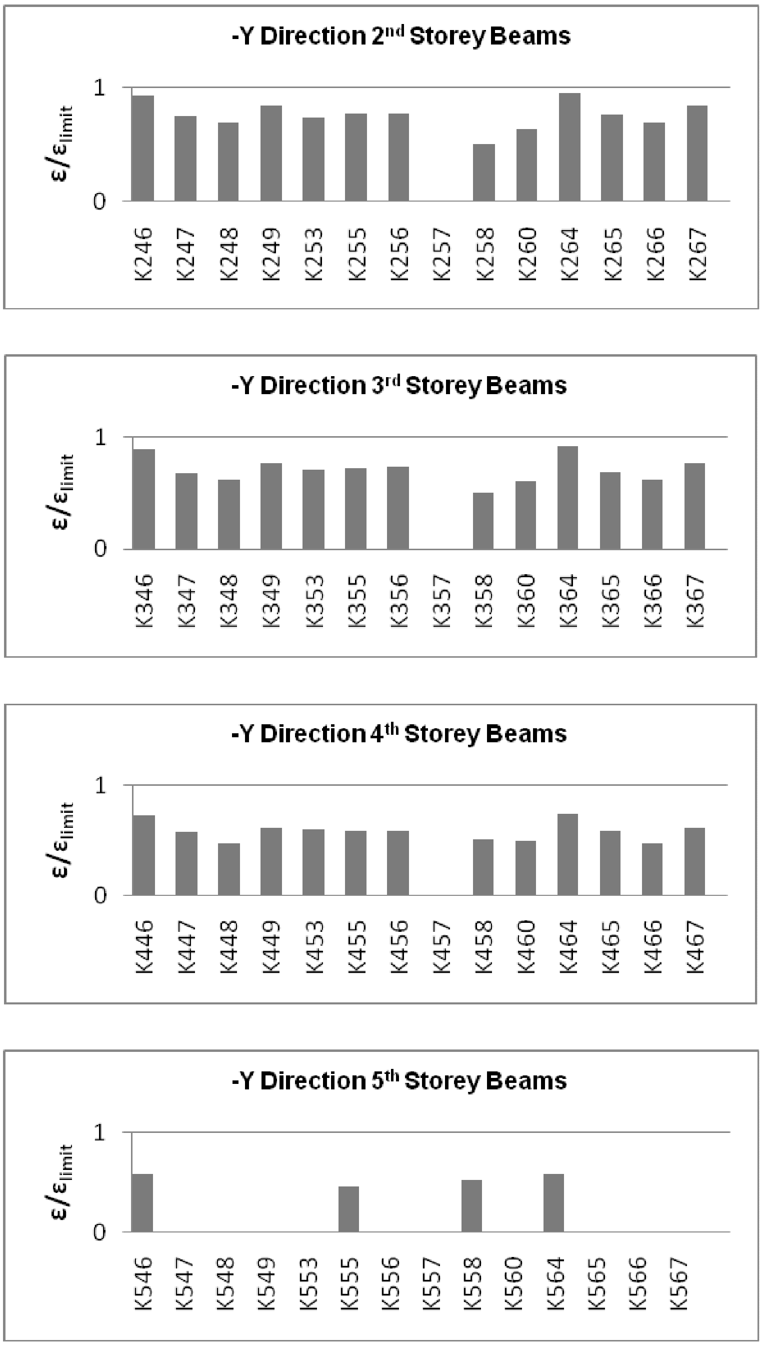


Figure A.16 ε / ε_{limit} for Beams (-Y direction)

APPENDIX B

ASSESSMENT RESULTS OF RETROFITTED BUILDING

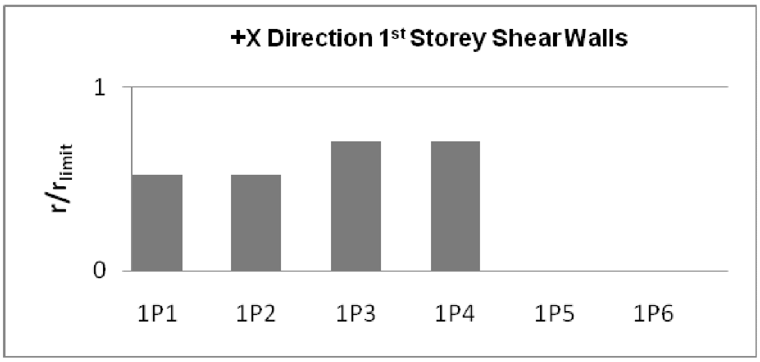


Figure B.1 r / r_{limit} for 1st Storey Shear Walls (+X Direction)

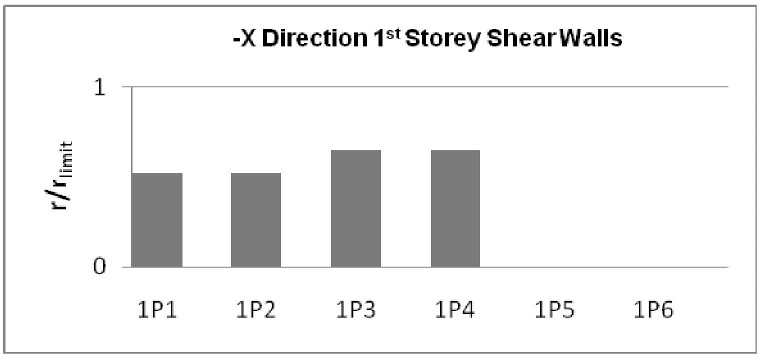


Figure B.2 r / r_{limit} for 1st Storey Shear Walls (-X Direction)

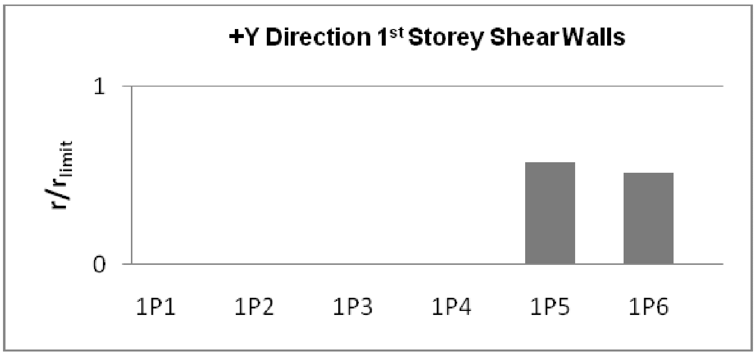


Figure B.3 r / r_{limit} for 1st Storey Shear Walls (+Y Direction)

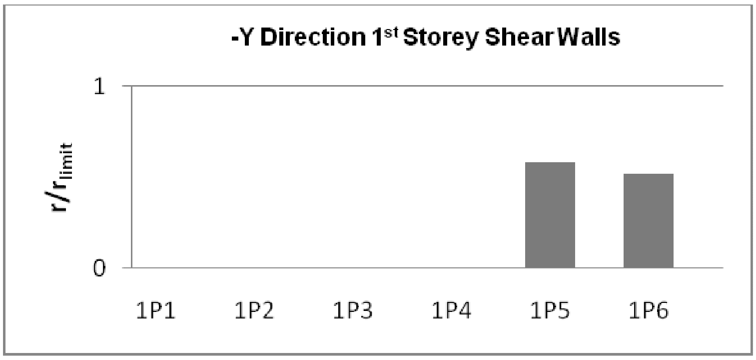
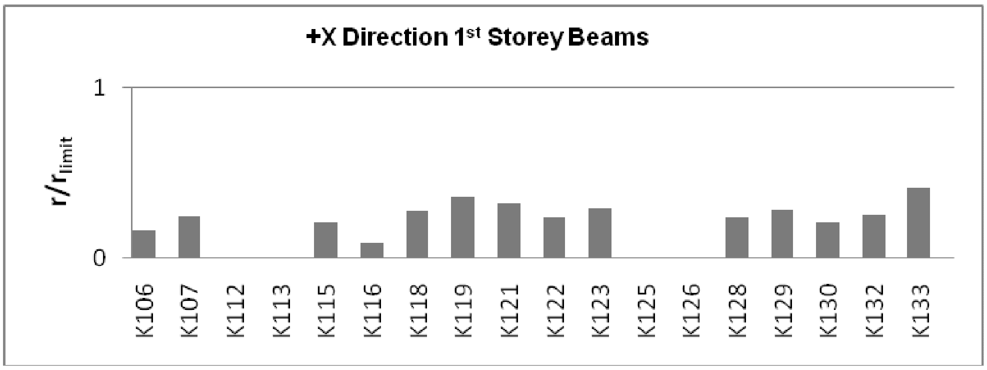
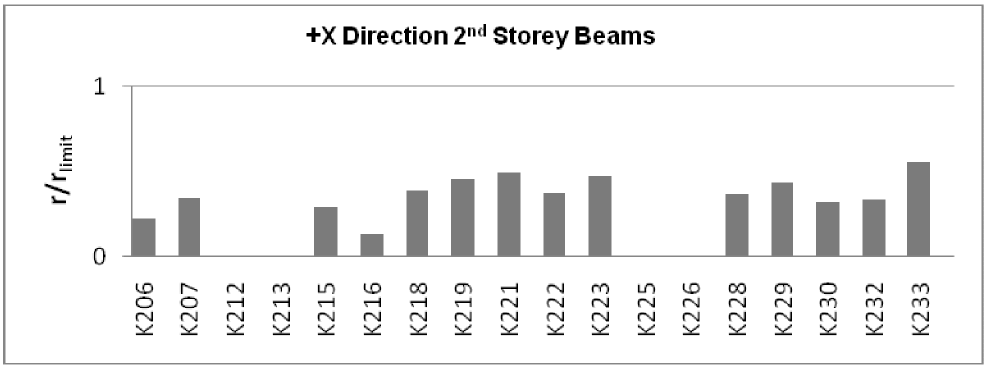
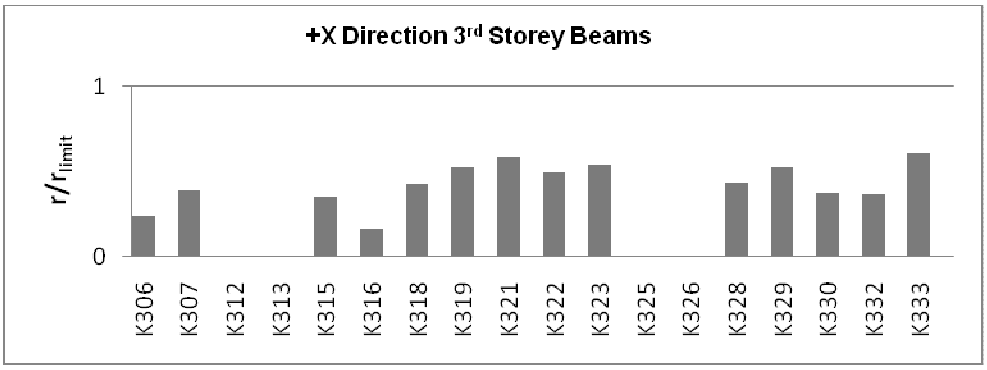
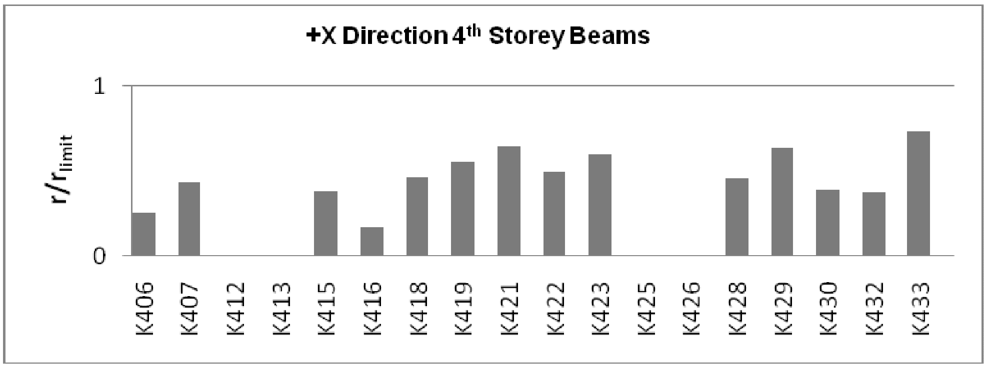


Figure B.4 r / r_{limit} for 1st Storey Shear Walls (-Y Direction)


+X Direction 1st Storey Beams
1
0
r/r_limit
K106
K107
K112
K113
K115
K116
K118
K119
K121
K122
K123
K125
K126
K128
K129
K130
K132
K133


+X Direction 2nd Storey Beams
1
0
r/r_limit
K206
K207
K212
K213
K215
K216
K218
K219
K221
K222
K223
K225
K226
K228
K229
K230
K232
K233


+X Direction 3rd Storey Beams
1
0
r/r_limit
K306
K307
K312
K313
K315
K316
K318
K319
K321
K322
K323
K325
K326
K328
K329
K330
K332
K333


+X Direction 4th Storey Beams
1
0
r/r_limit
K406
K407
K412
K413
K415
K416
K418
K419
K421
K422
K423
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K426
K428
K429
K430
K432
K433

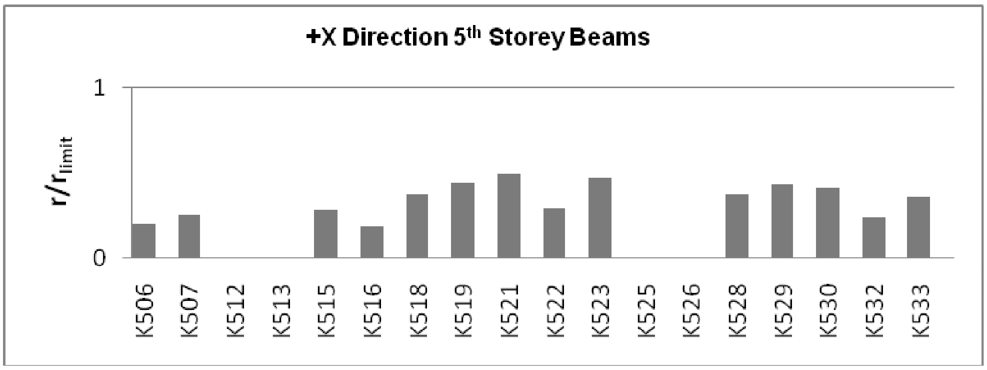
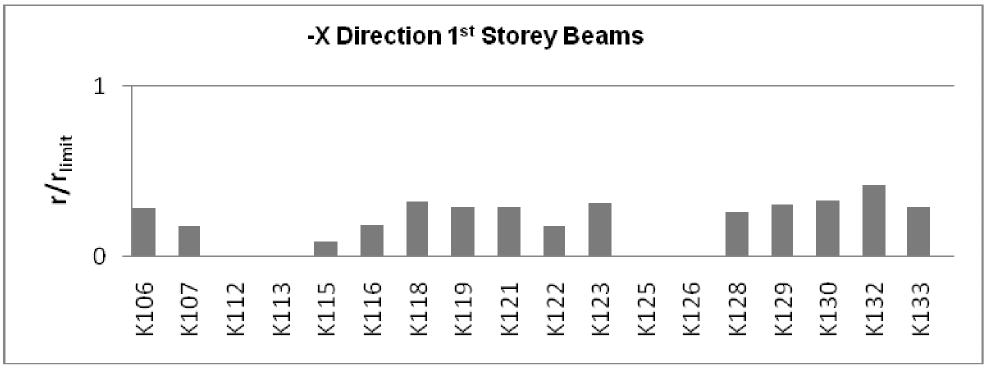
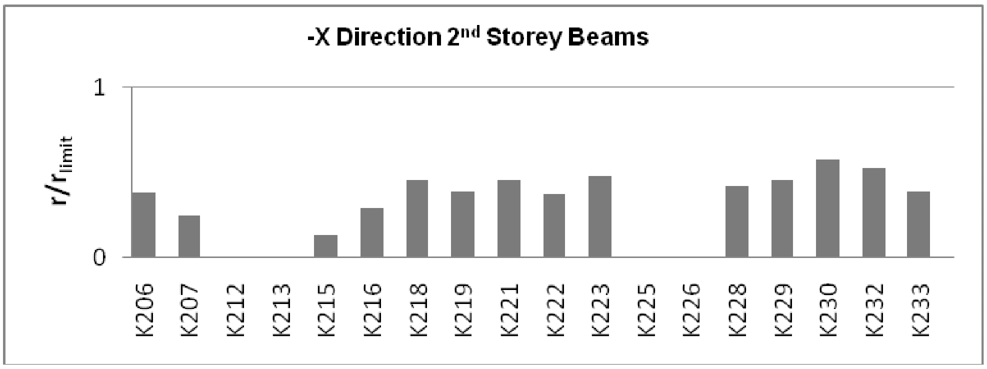
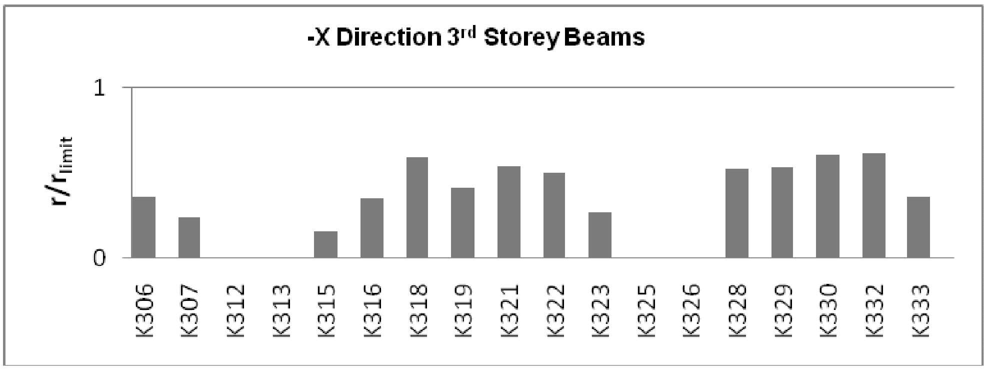
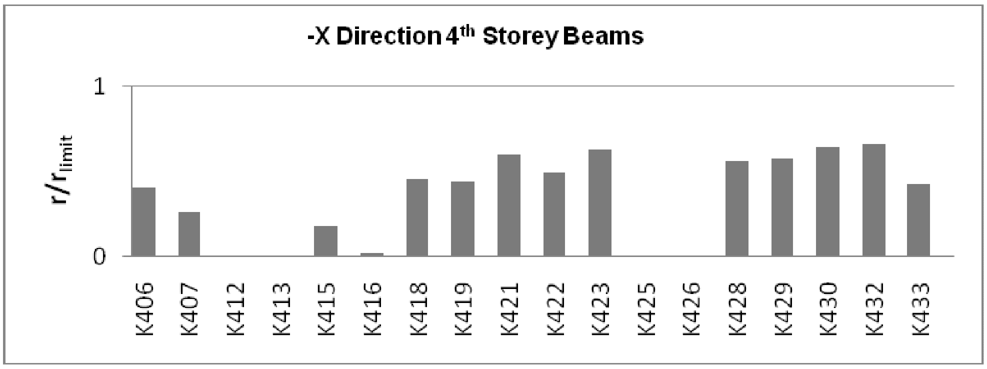


Figure B.5 r / r_{limit} for Beams (+X Direction)









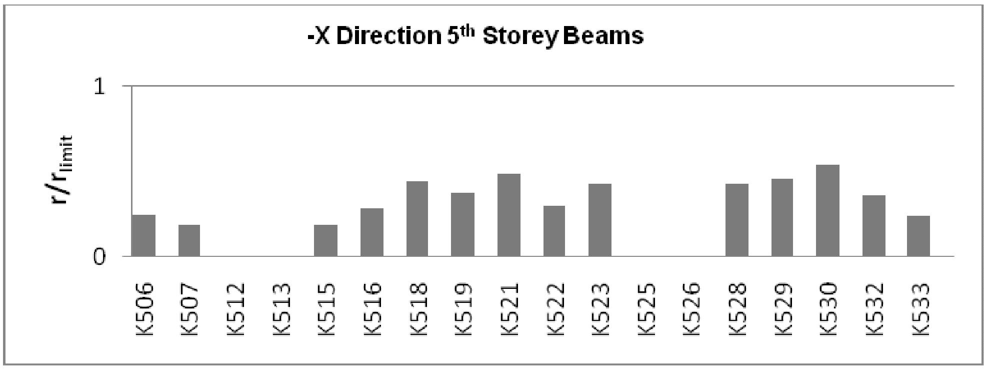
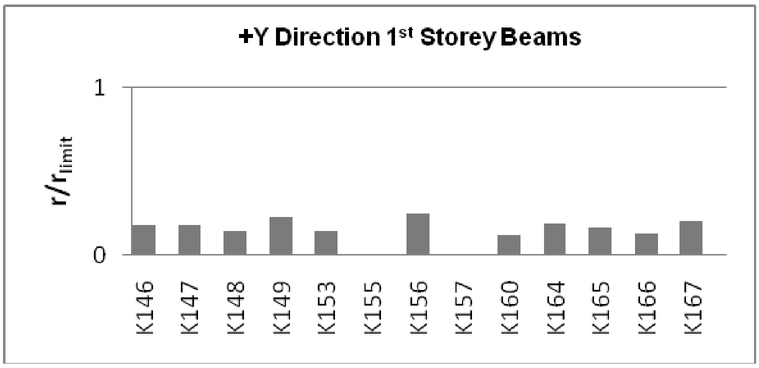


Figure B.6 r / r_{limit} for Beams (-X Direction)


+Y Direction 1st Storey Beams
1
0
r/r_limit
K146
K147
K148
K149
K153
K155
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K157
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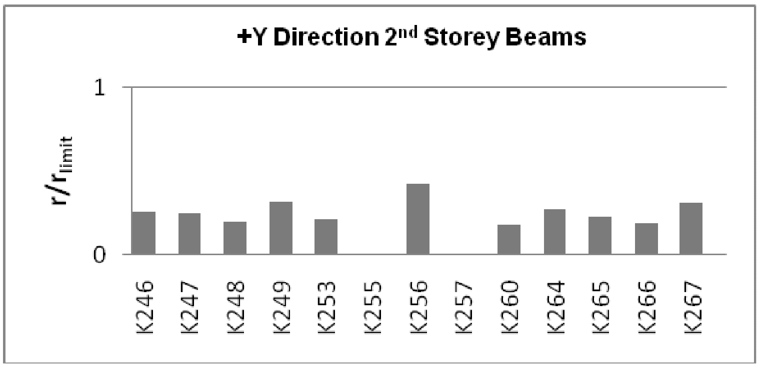
K165
K166
K167


+Y Direction 2nd Storey Beams
1
0
r/r_limit
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K247
K248
K249
K253
K255
K256
K257
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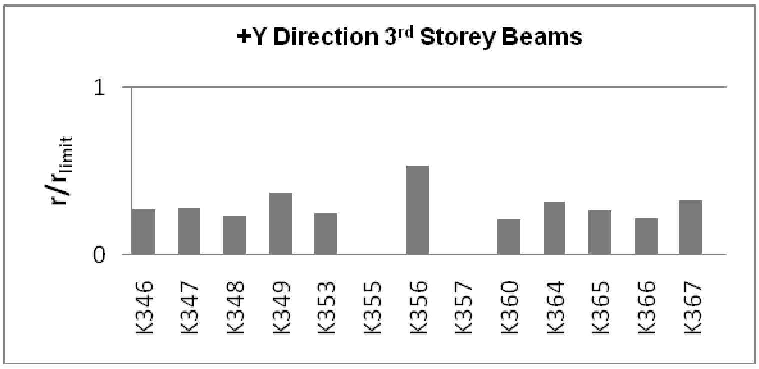
K265
K266
K267


+Y Direction 3rd Storey Beams
1
0
r/r_limit
K346
K347
K348
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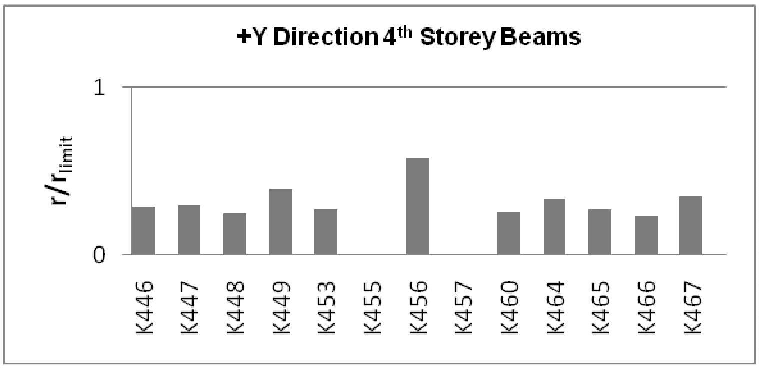
K367


+Y Direction 4th Storey Beams
1
0
r/r_limit
K446
K447
K448
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K457
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K465
K466
K467

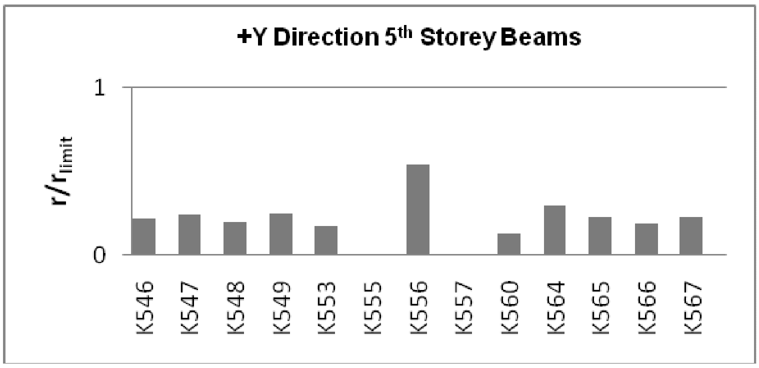
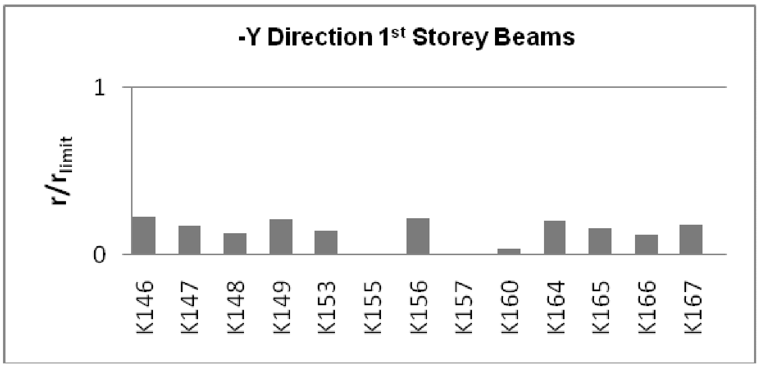
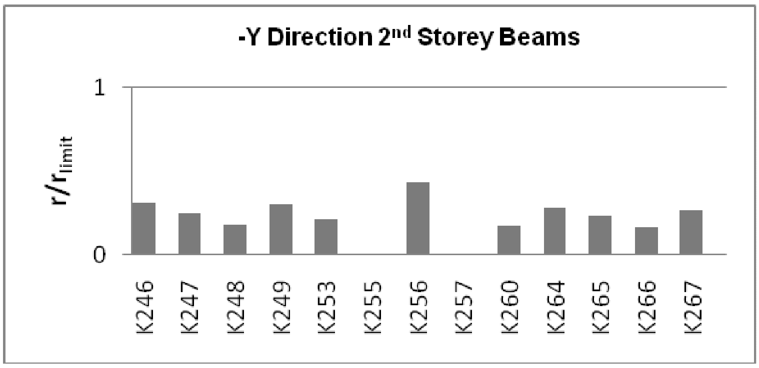


Figure B.7 r / r_{limit} for Beams (+Y Direction)





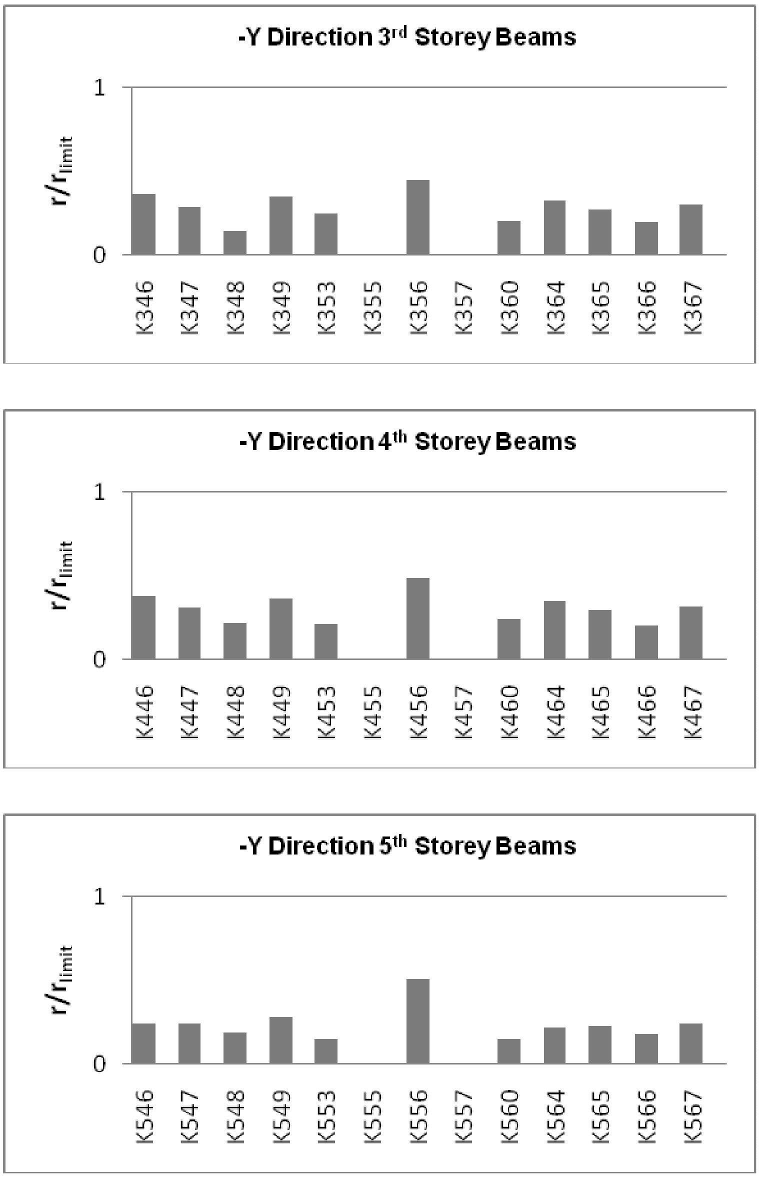


Figure B.8 r / r_{limit} for Beams (-Y Direction)

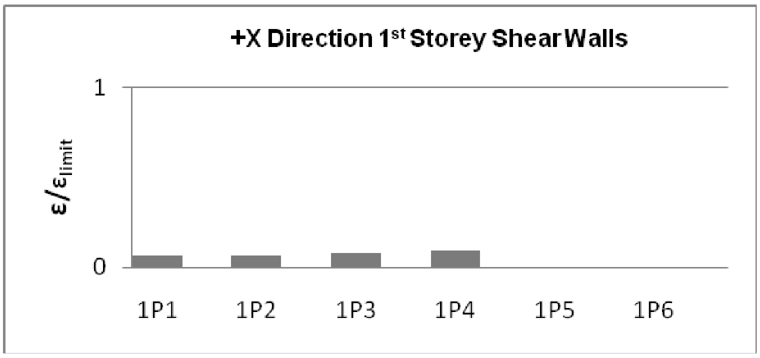


Figure B.9 ε / ε_{limit} for 1st Storey Shear Walls Beams (+X Direction)

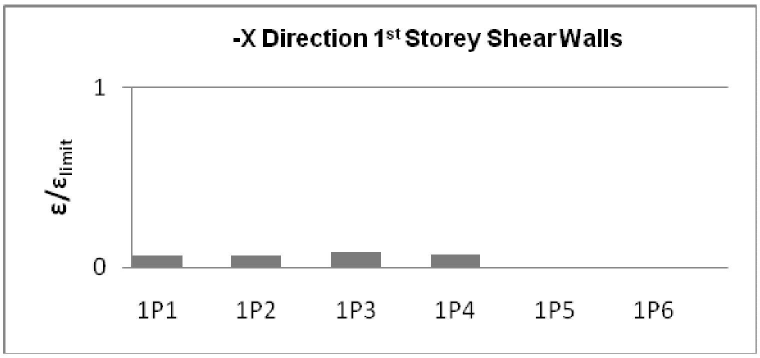


Figure B.10 ε / ε_{limit} for 1st Storey Shear Walls Beams (-X Direction)

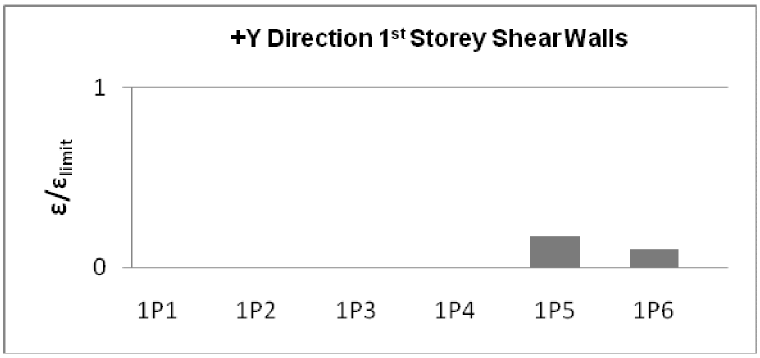


Figure B.11 ε / ε_{limit} for 1st Storey Shear Walls Beams (+Y Direction)

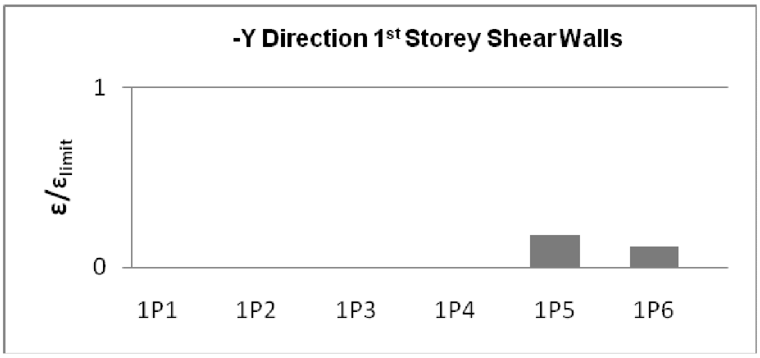


Figure B.12 ε / ε_{limit} for 1st Storey Shear Walls Beams (-Y Direction)

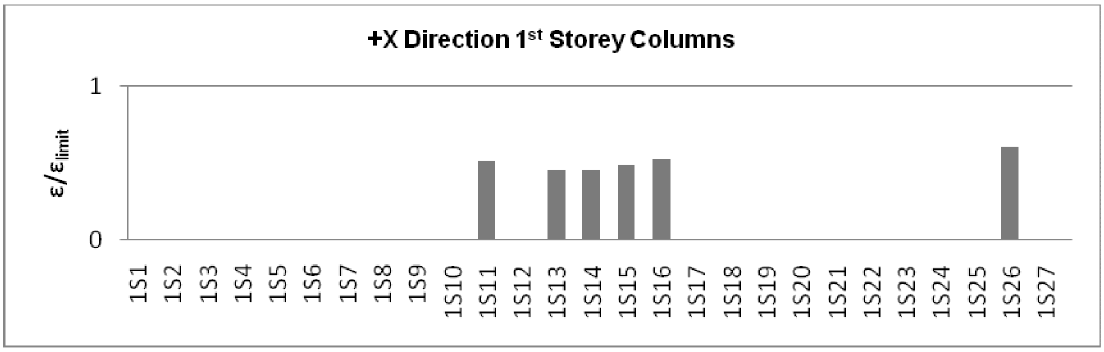


Figure B.13 ε / ε_{limit} for 1st Storey Columns (+X Direction)

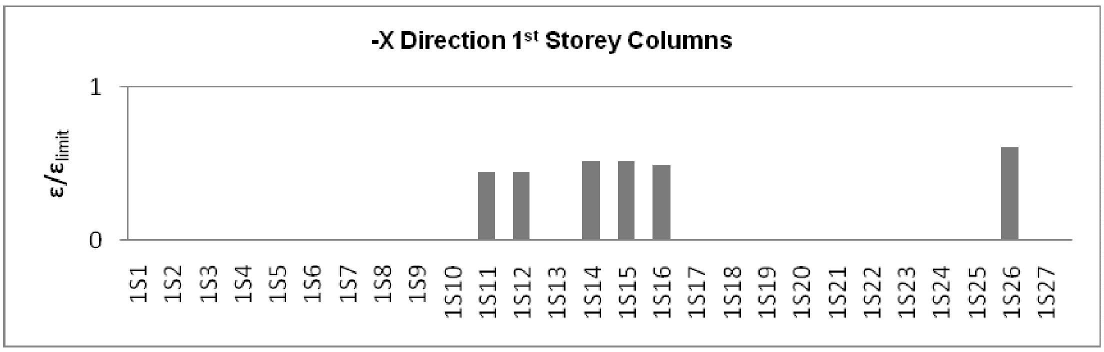
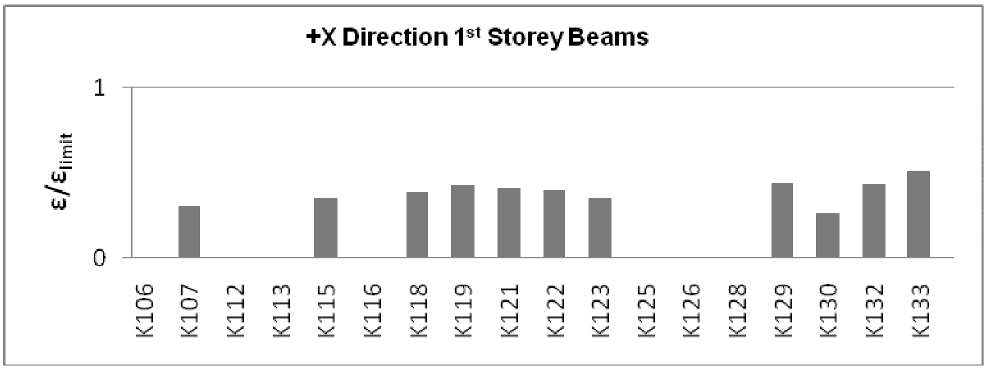
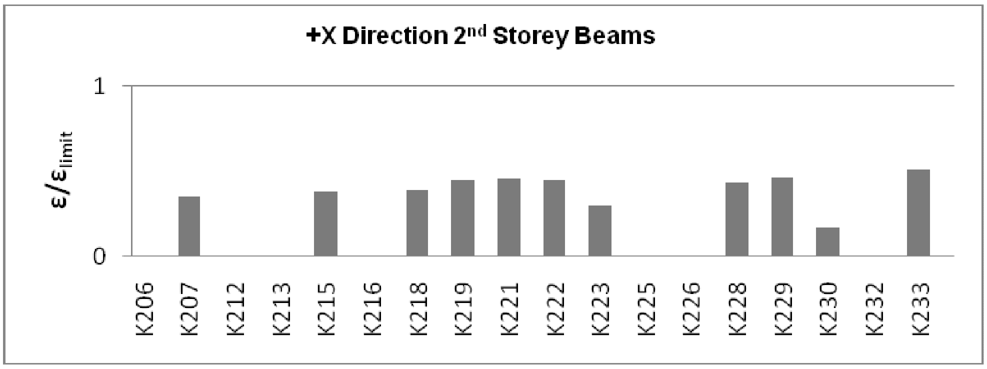
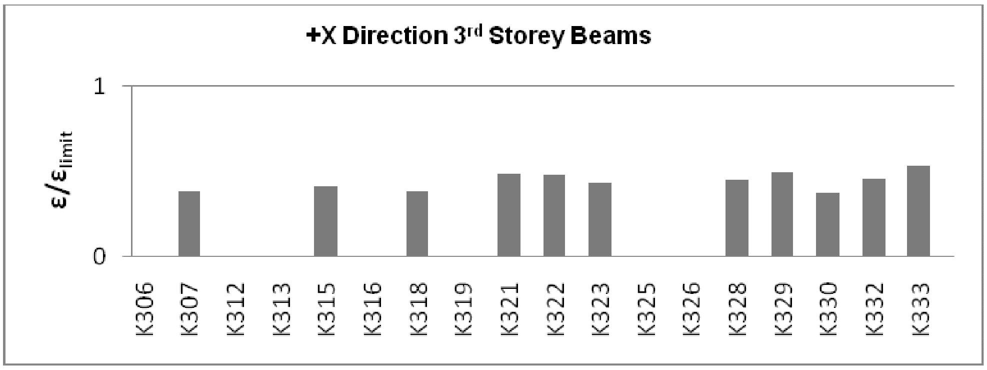
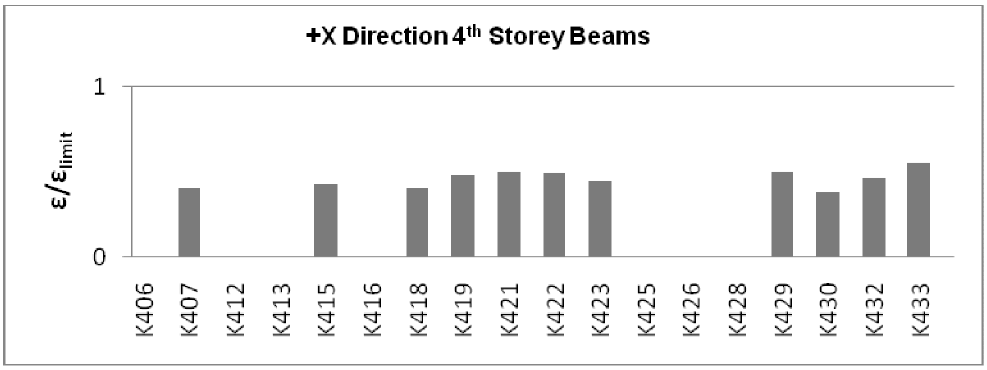


Figure B.14 ε / ε_{limit} for 1st Storey Columns (-X Direction)









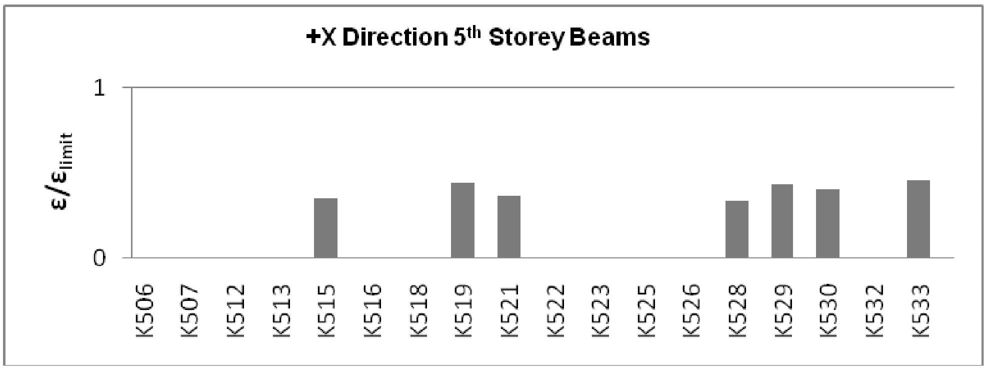
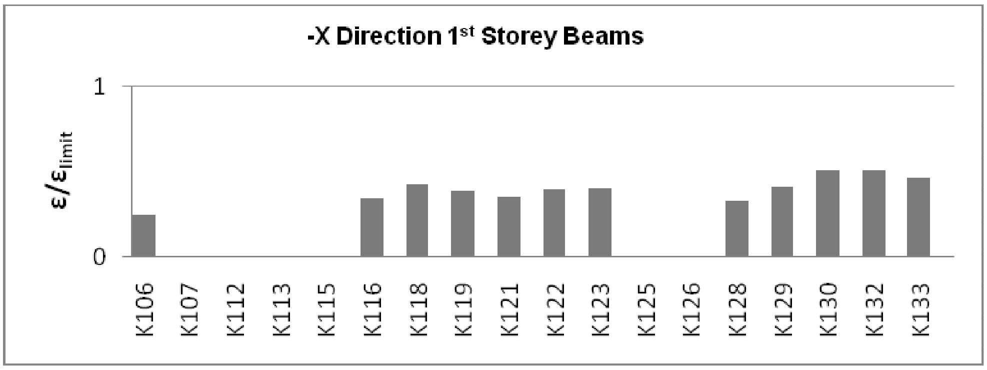
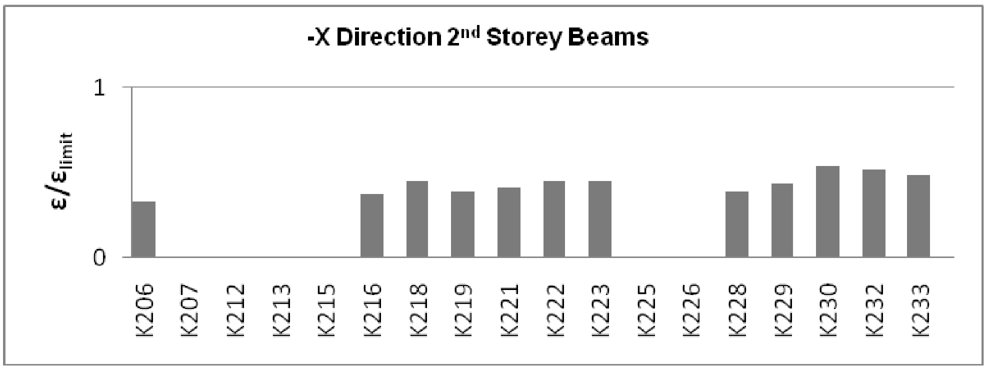


Figure B.15 $\varepsilon / \varepsilon_{limit}$ for Beams (+X Direction)





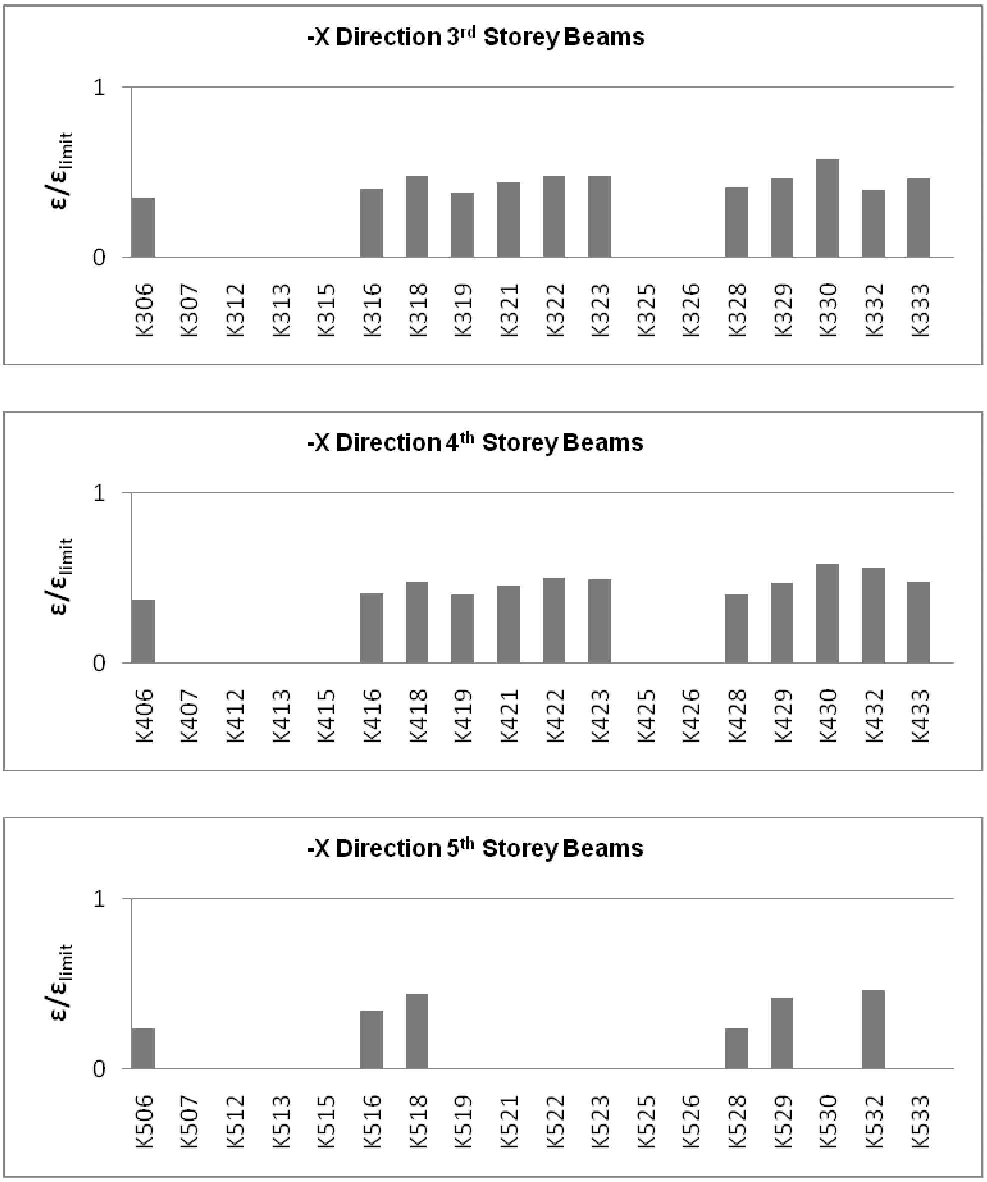
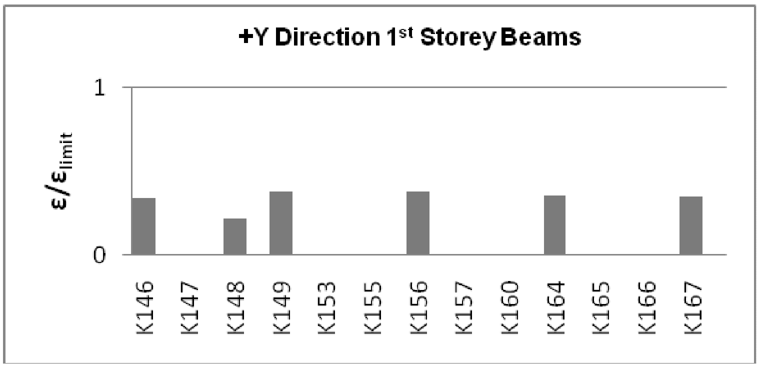


Figure B.16 ε / ε_{limit} for Beams (-X Direction)


+Y Direction 1st Storey Beams
ε/ε limit
1
0
K146
K147
K148
K149
K153
K155
K156
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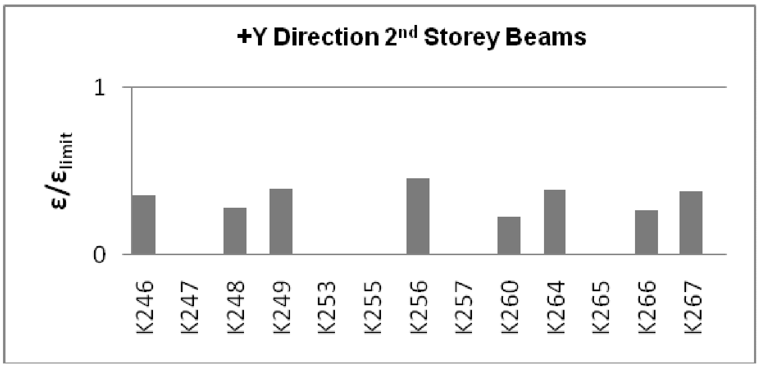
K165
K166
K167


+Y Direction 2nd Storey Beams
ε/ε limit
1
0
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K248
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K255
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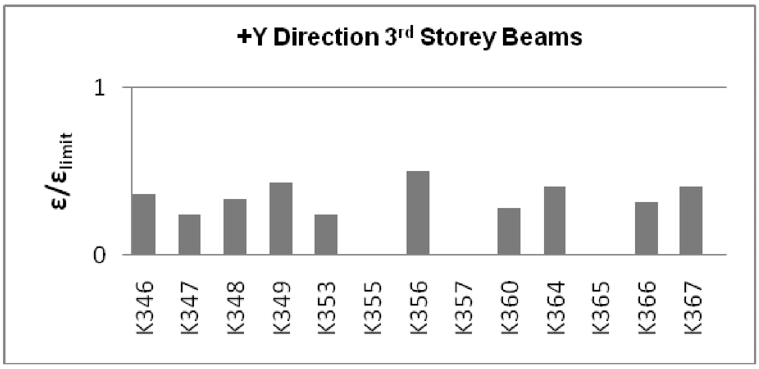
K266
K267


+Y Direction 3rd Storey Beams
ε/ε limit
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0
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K347
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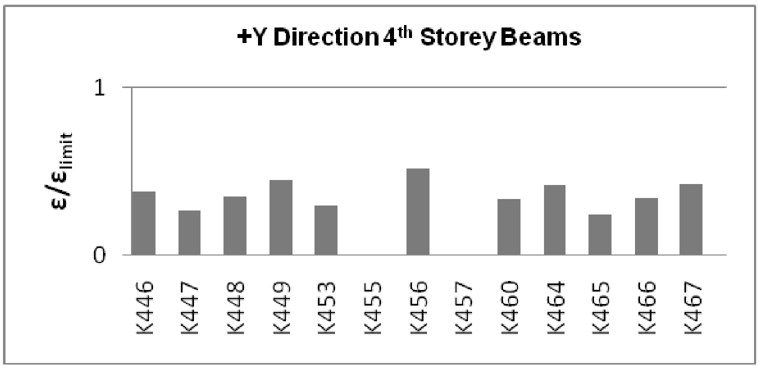
K367


+Y Direction 4th Storey Beams
ε/ε limit
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0
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K467

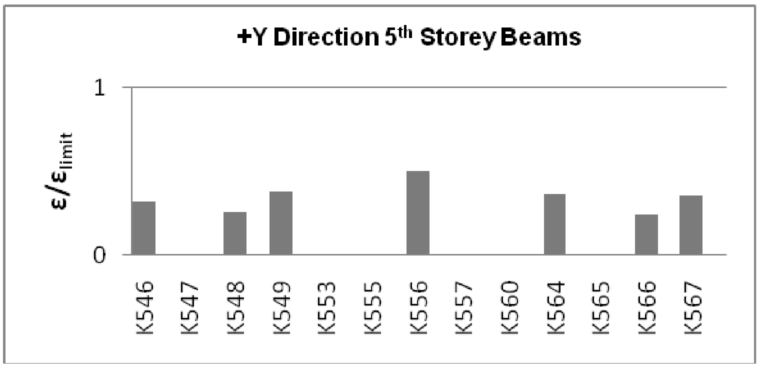
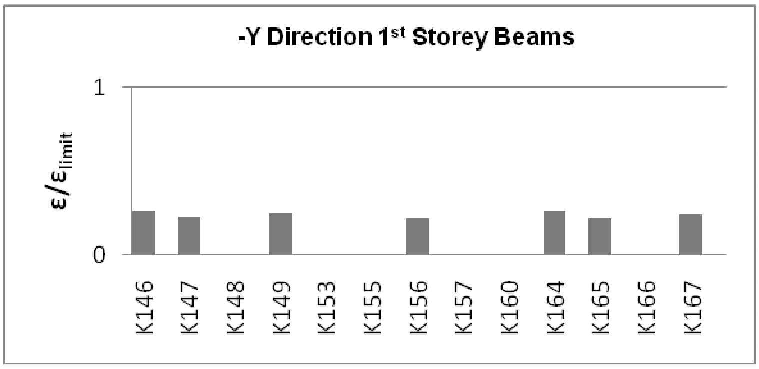
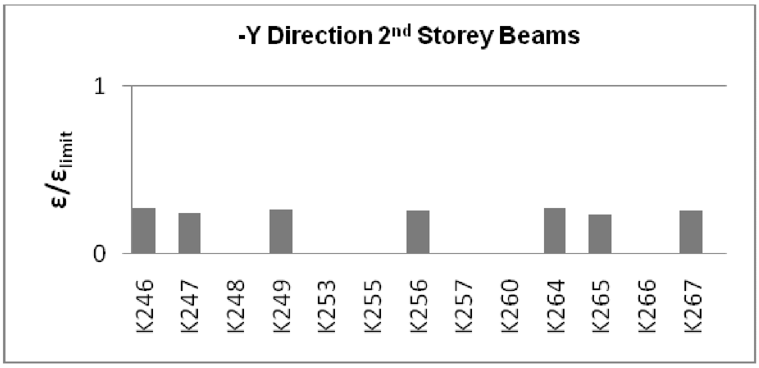
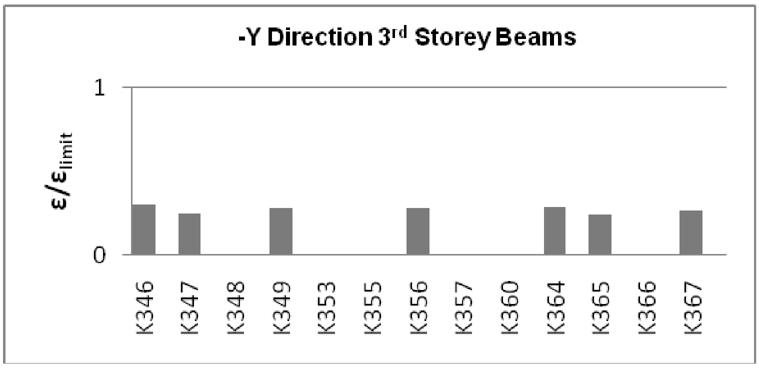
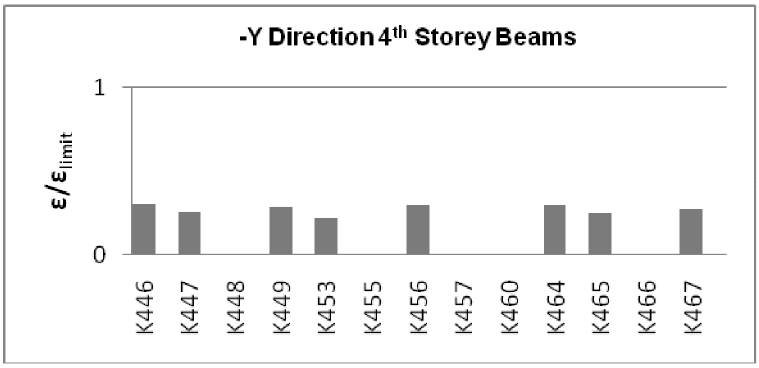


Figure B.17 ε / ε_{limit} for Beams (+Y Direction)









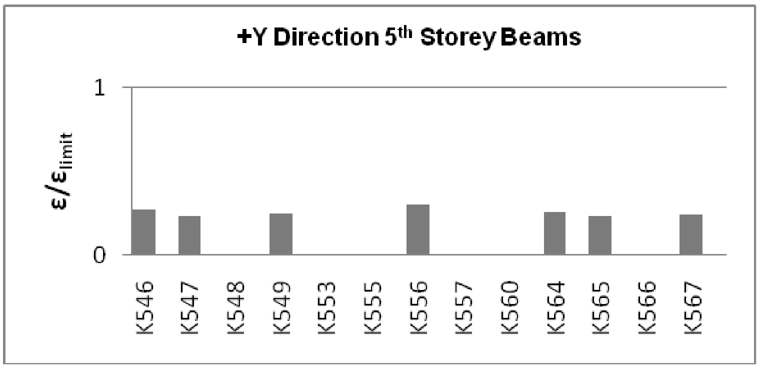
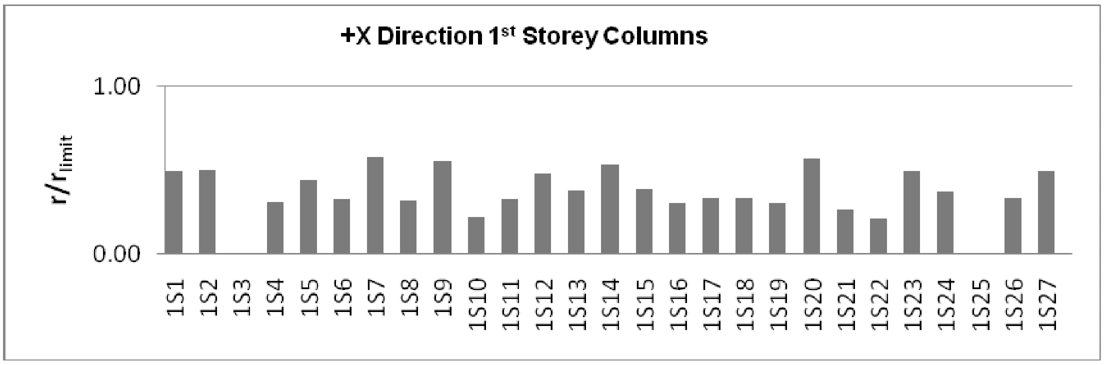
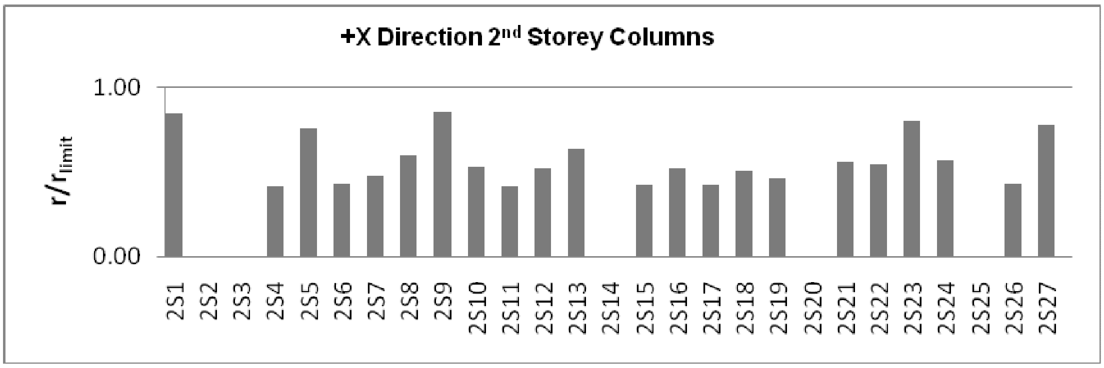


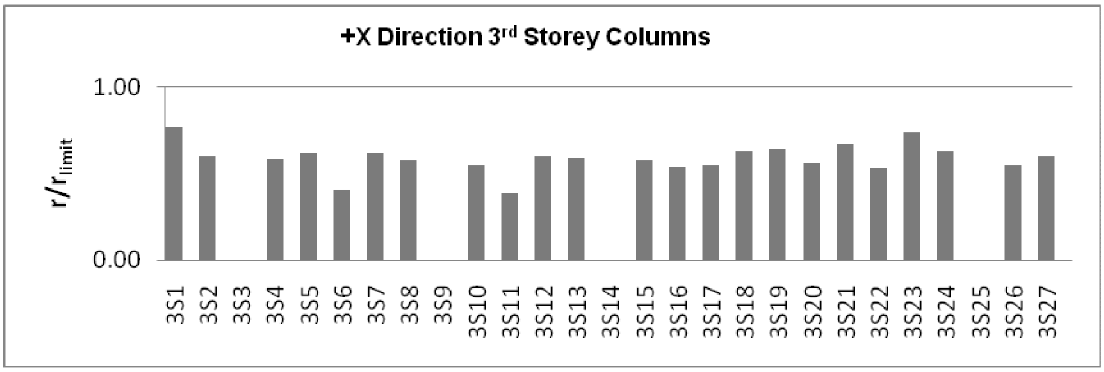
Figure B.18 ε / ε_{limit} for Beams (-Y Direction)

APPENDIX C

ASSESSMENT RESULTS OF RE-DESIGIN







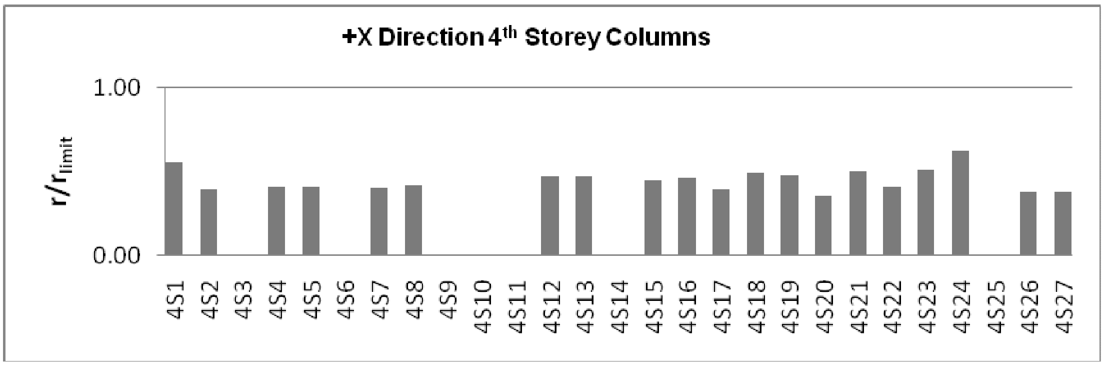
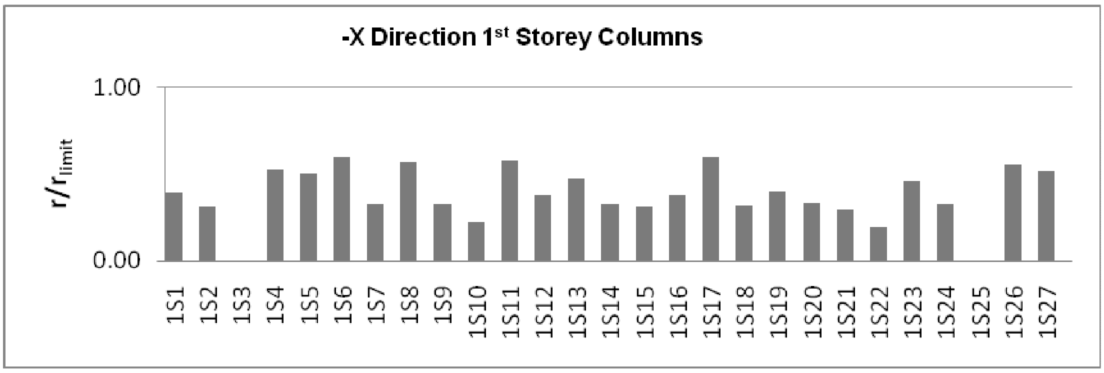
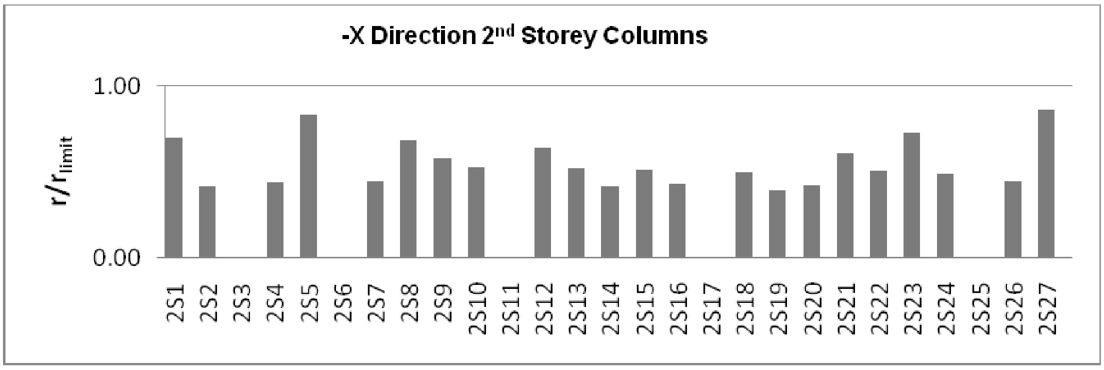
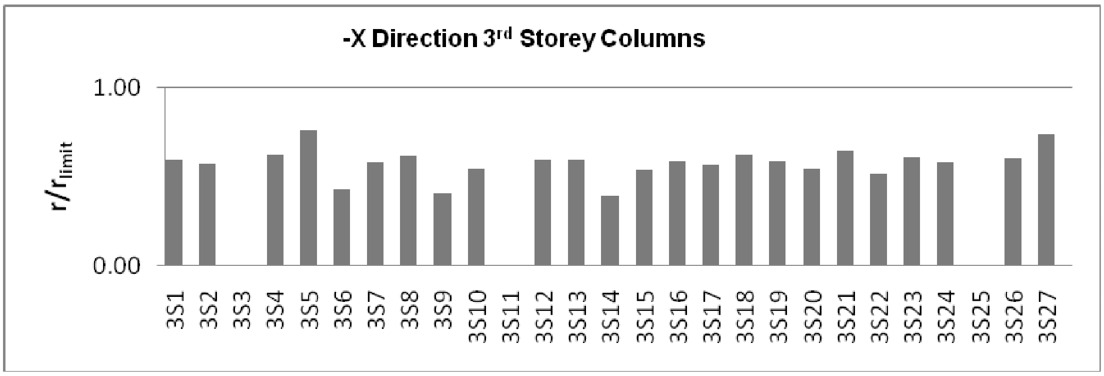


Figure C.1 r / r_{limit} for Columns (+X Direction)







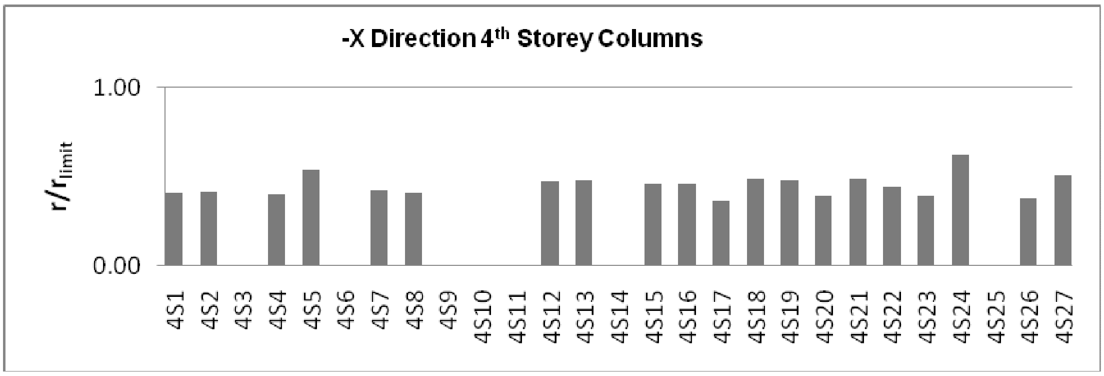
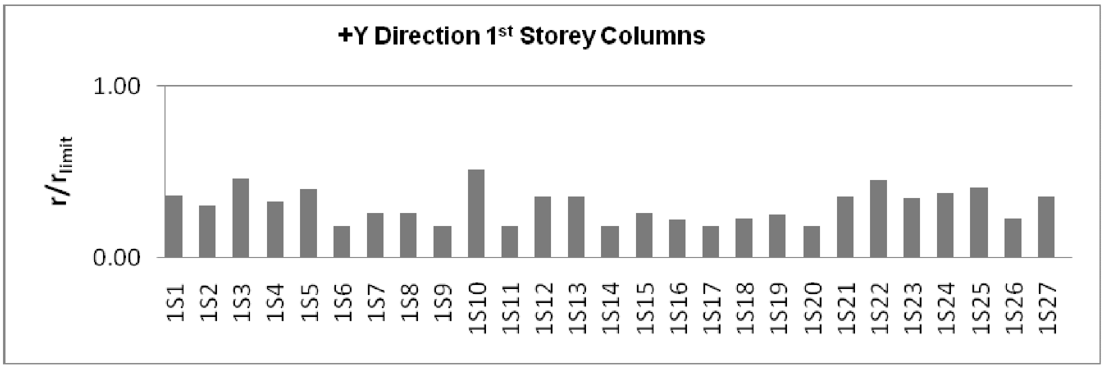


Figure C.2 r / r_{limit} for Columns (-X Direction)



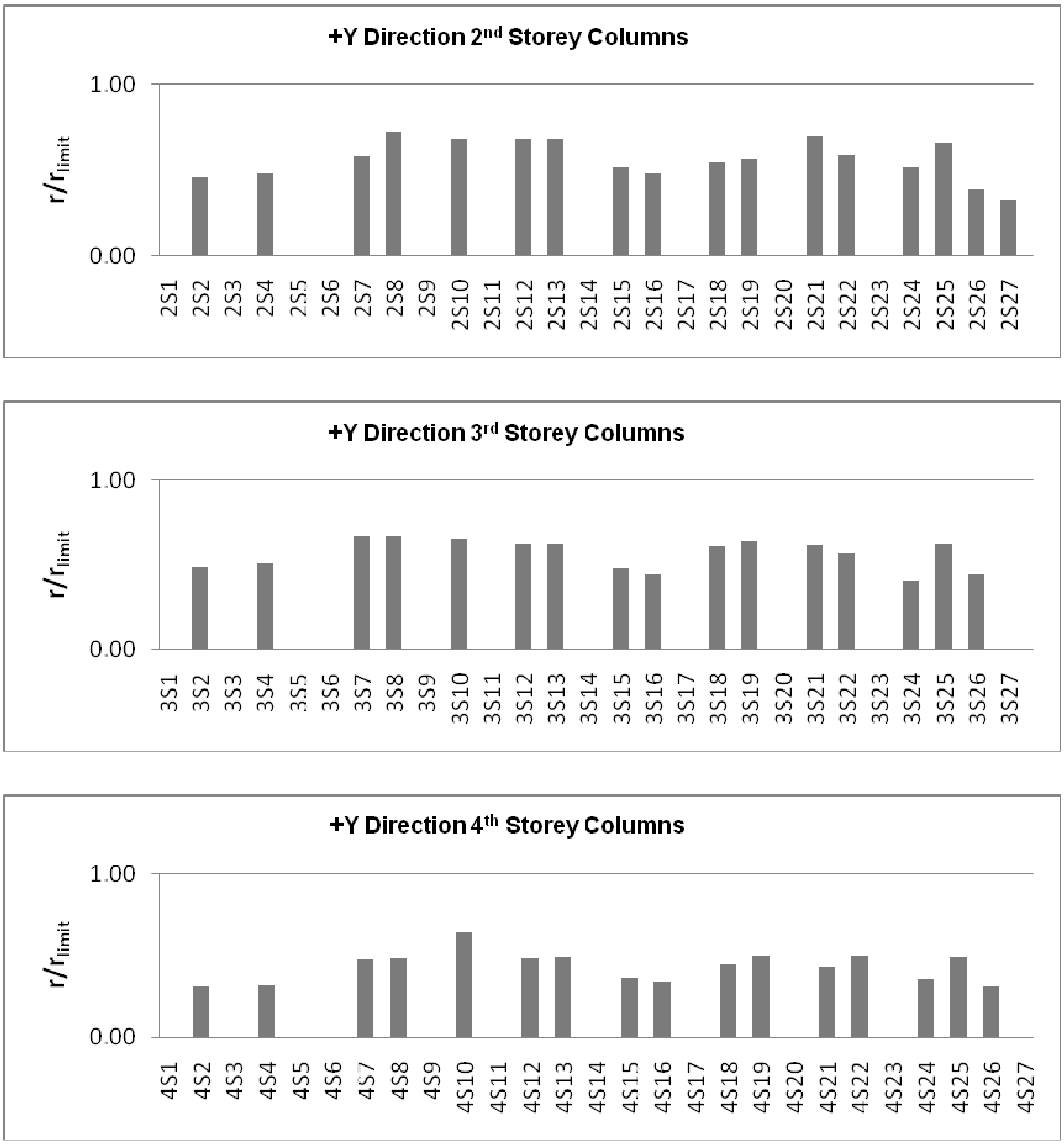
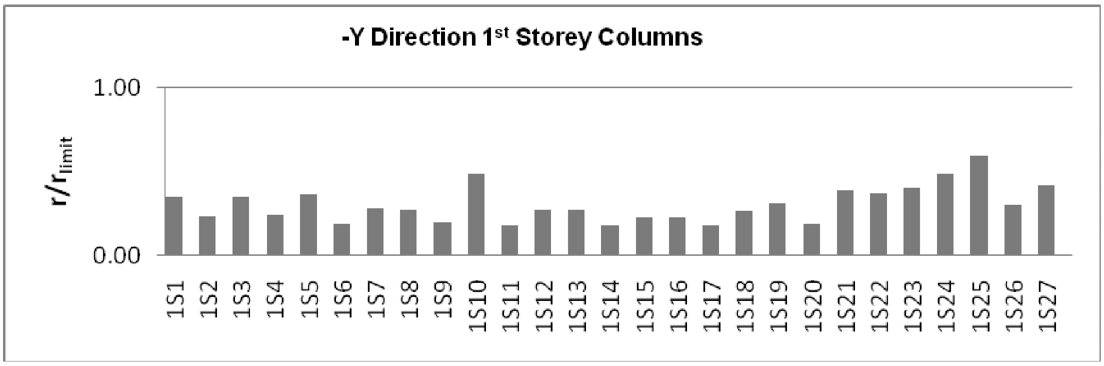
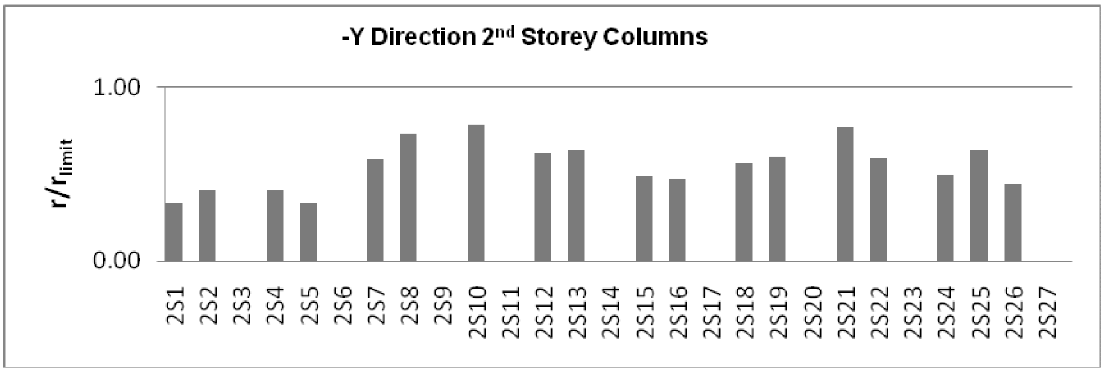
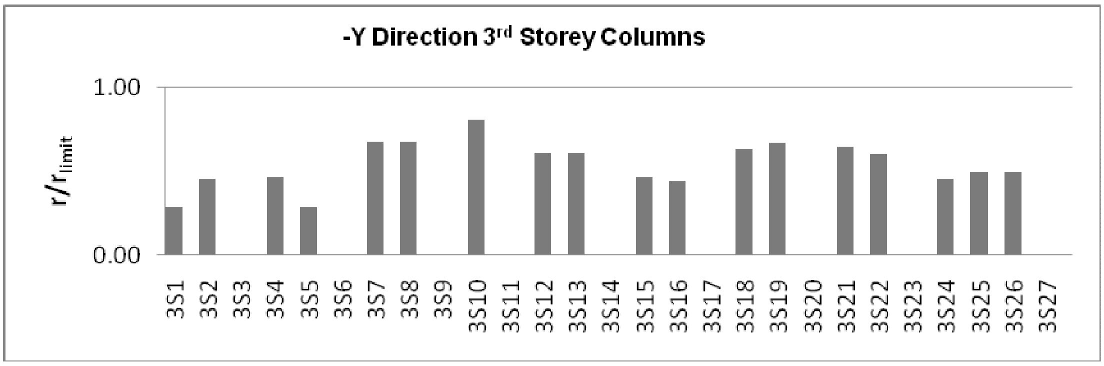


Figure C.3 r / r_{limit} for Columns (+Y Direction)







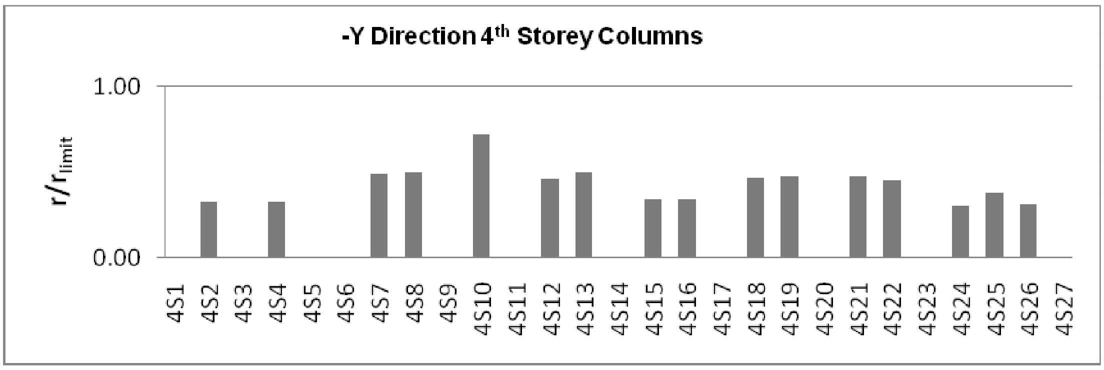
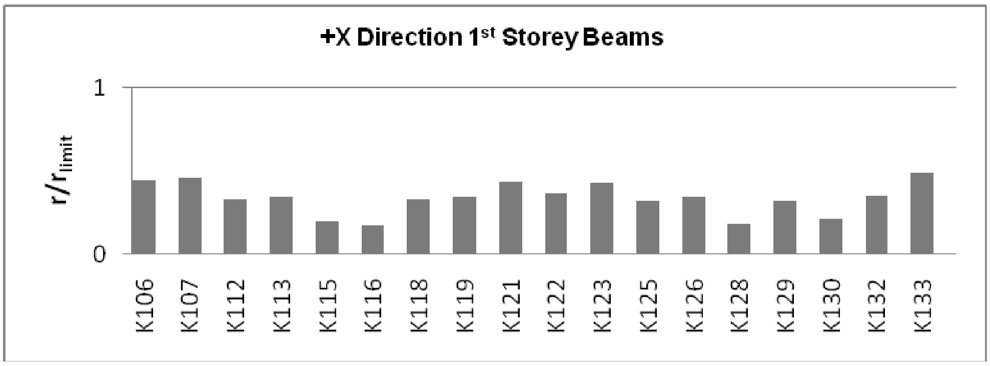
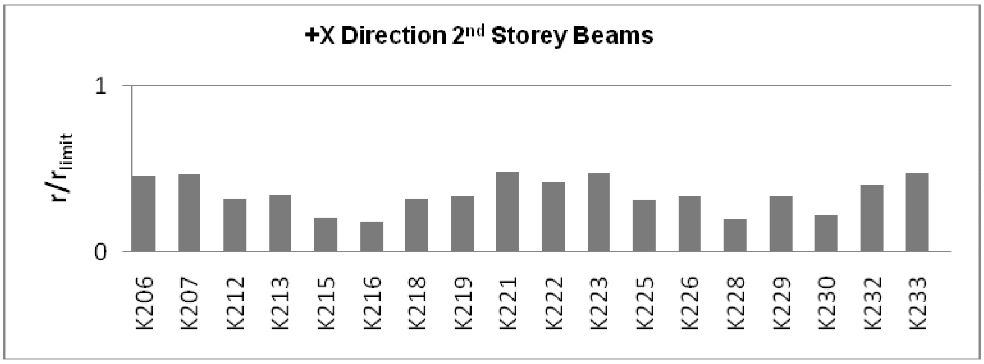
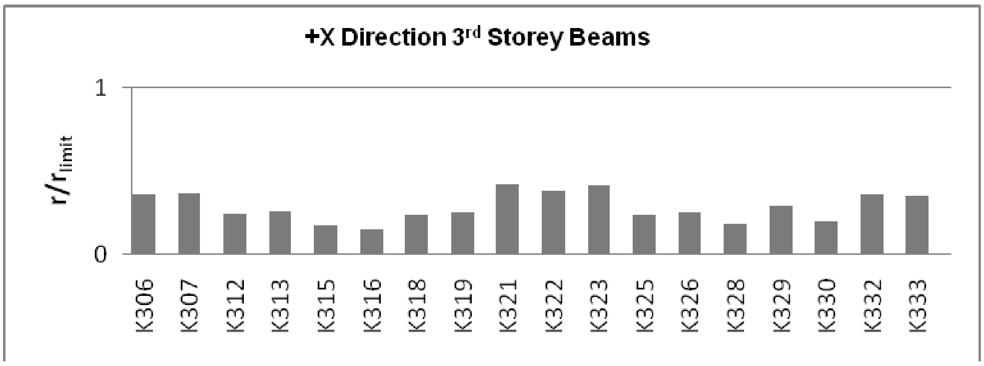
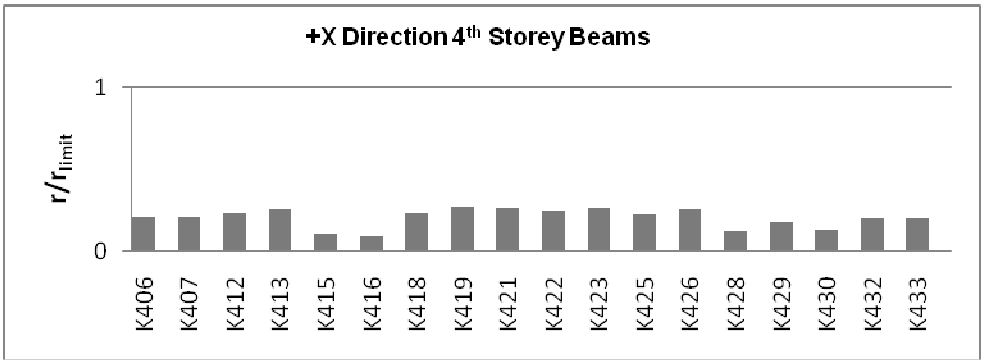


Figure C.4 r / r_{limit} for Columns (-Y Direction)


+X Direction 1st Storey Beams
r/r limit
1
0
K106 K107 K112 K113 K115 K116 K118 K119 K121 K122 K123 K125 K126 K128 K129 K130 K132 K133


+X Direction 2nd Storey Beams
r/r limit
1
0
K206 K207 K212 K213 K215 K216 K218 K219 K221 K222 K223 K225 K226 K228 K229 K230 K232 K233


+X Direction 3rd Storey Beams
r/r limit
1
0
K306 K307 K312 K313 K315 K316 K318 K319 K321 K322 K323 K325 K326 K328 K329 K330 K332 K333


+X Direction 4th Storey Beams
r/r limit
1
0
K406 K407 K412 K413 K415 K416 K418 K419 K421 K422 K423 K425 K426 K428 K429 K430 K432 K433

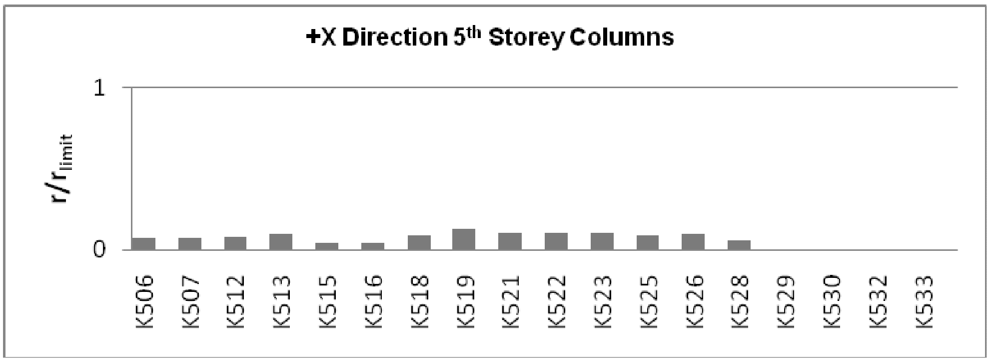
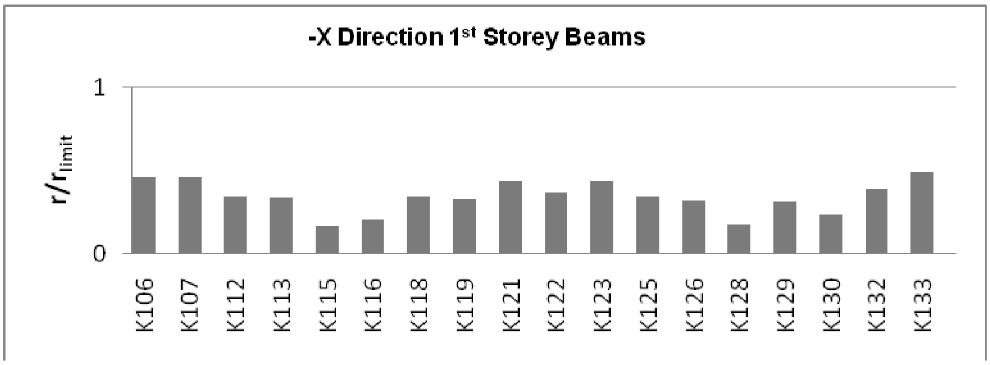
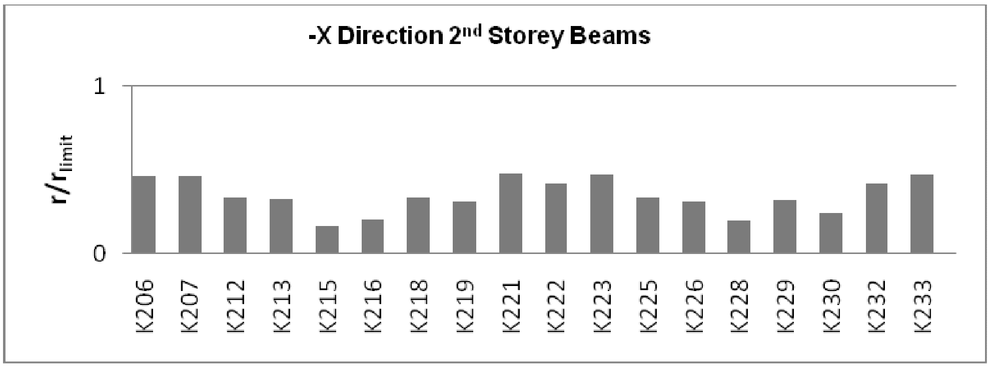
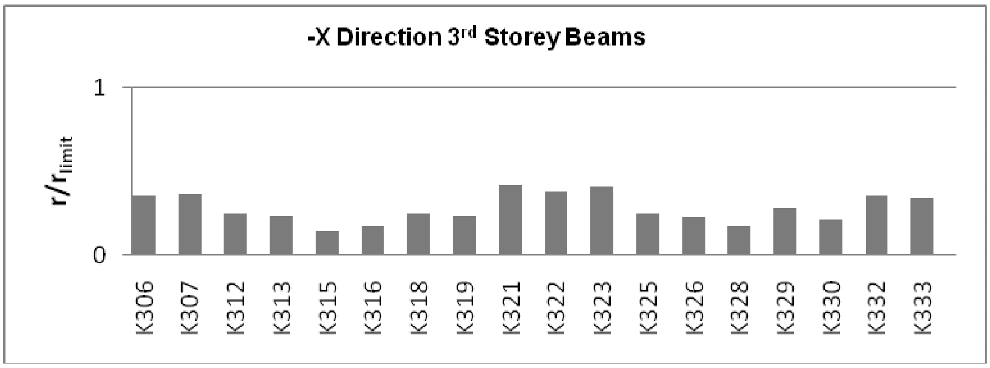
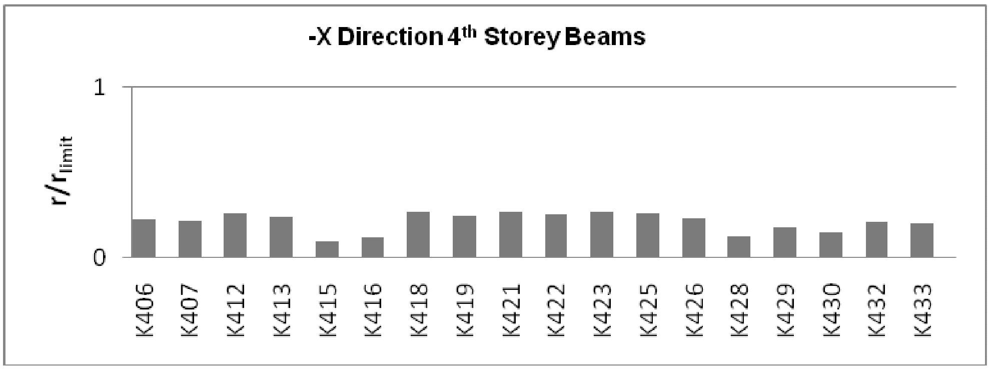


Figure C.5 r / r_{limit} for Columns (+X Direction)









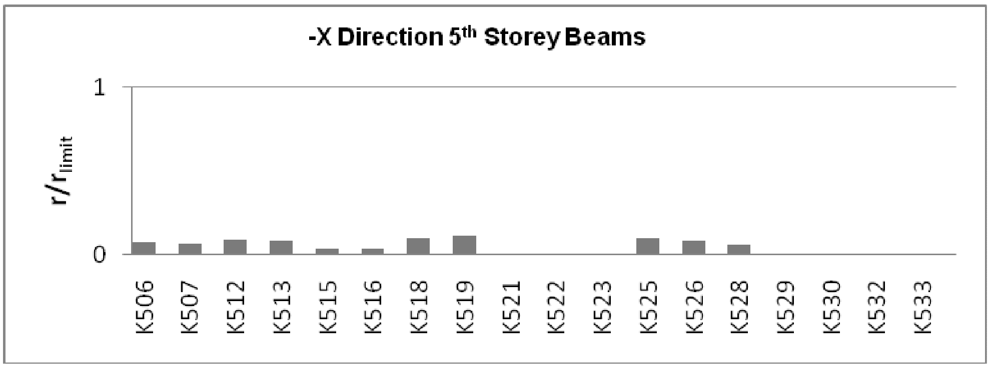
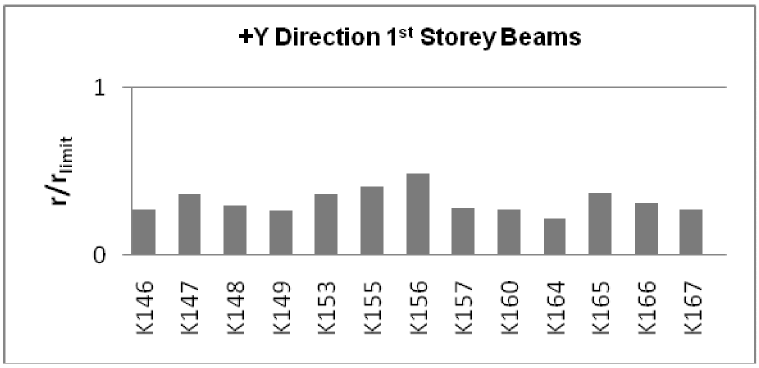


Figure C.6 r / r_{limit} for Beams (-X Direction)


+Y Direction 1st Storey Beams
r/rlimit
1
0
K146
K147
K148
K149
K153
K155
K156
K157
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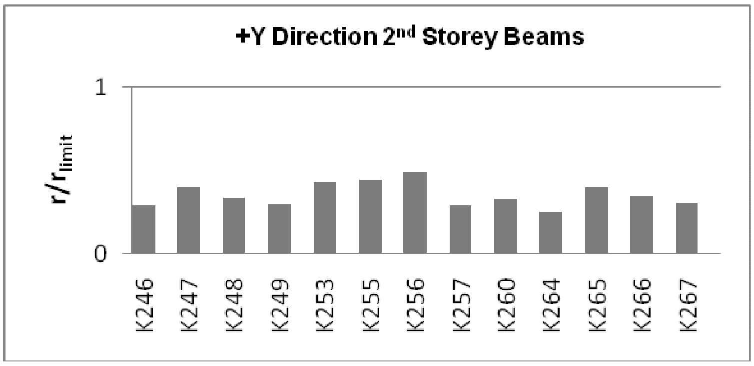
K165
K166
K167


+Y Direction 2nd Storey Beams
r/rlimit
1
0
K246
K247
K248
K249
K253
K255
K256
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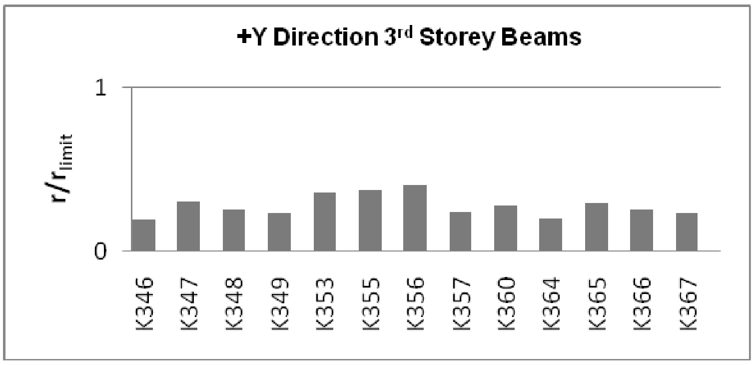
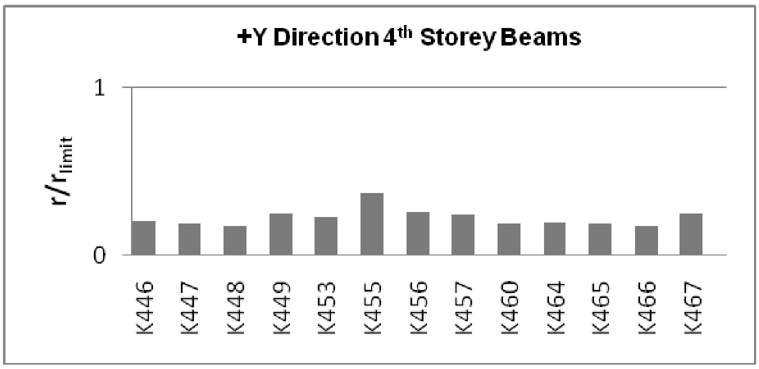
K264
K265
K266
K267


+Y Direction 3rd Storey Beams
r/rlimit
1
0
K346
K347
K348
K349
K353
K355
K356
K357
K360
K364
K365
K366
K367


+Y Direction 4th Storey Beams
r/rlimit
1
0
K446
K447
K448
K449
K453
K455
K456
K457
K460
K464
K465
K466
K467

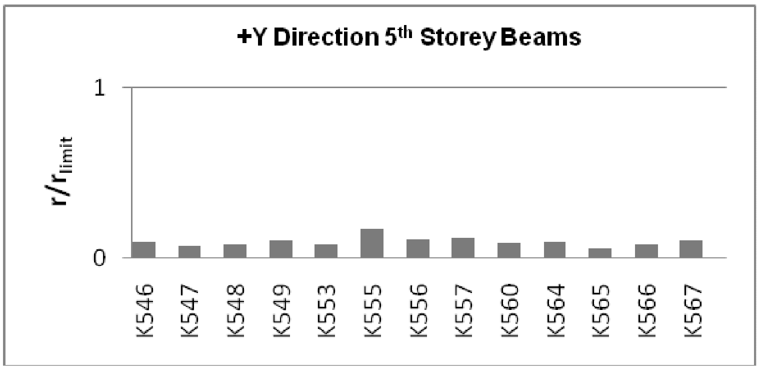
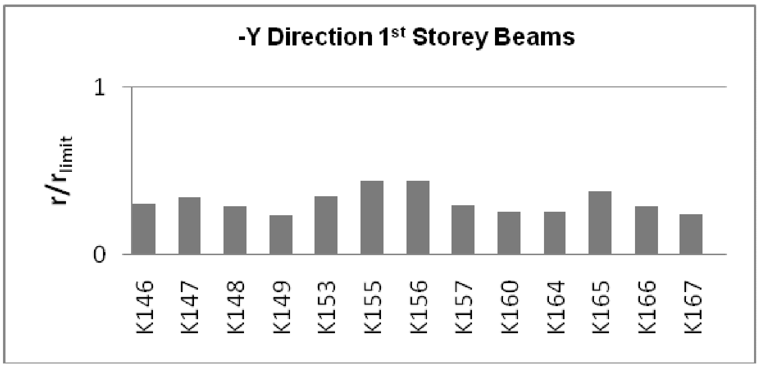
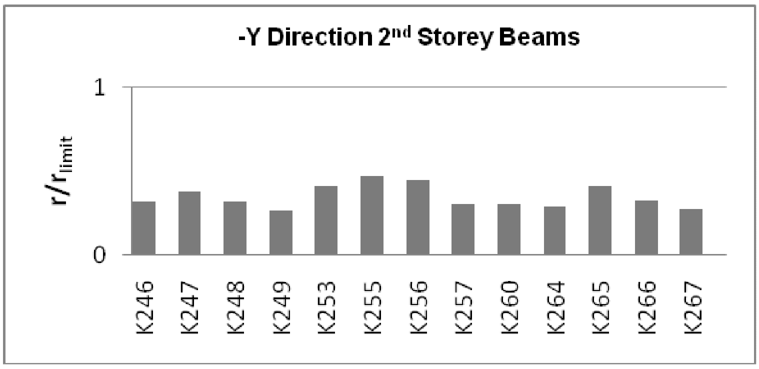


Figure C.7 r / r_{limit} for Beams (+Y Direction)





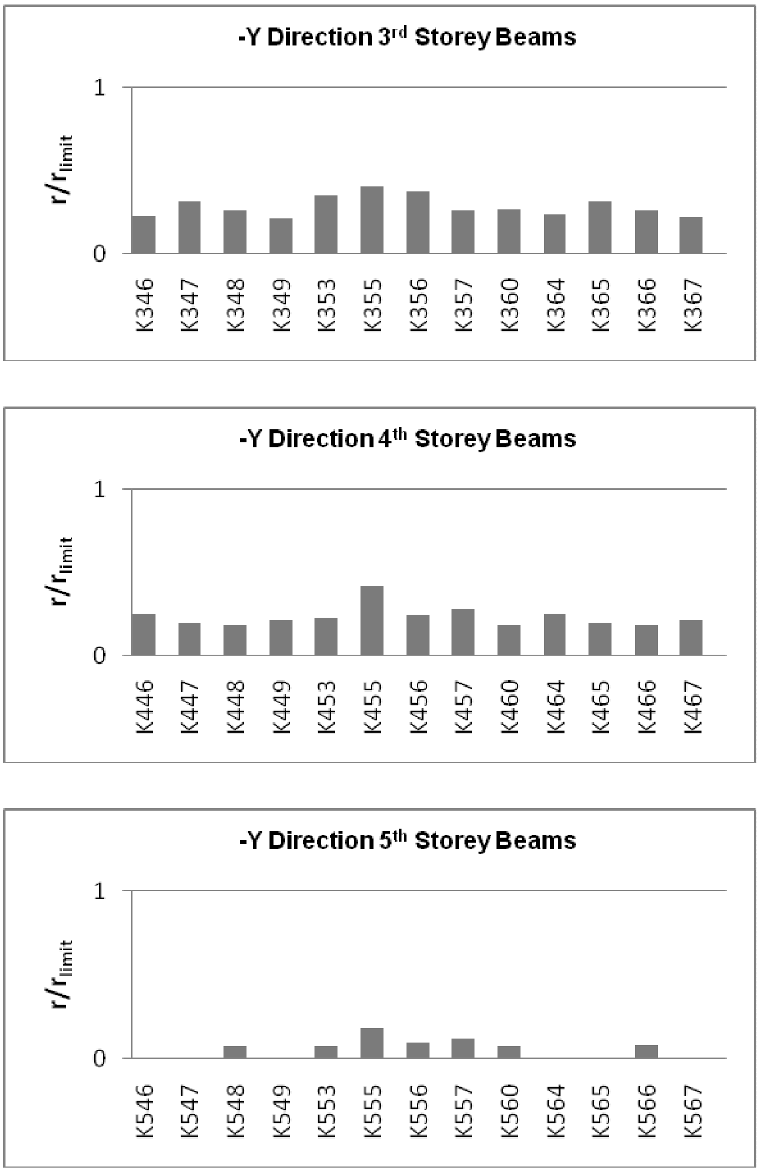
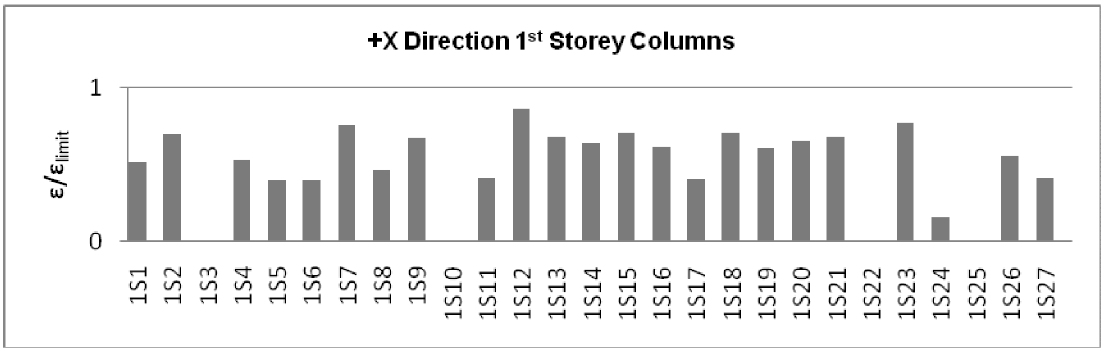
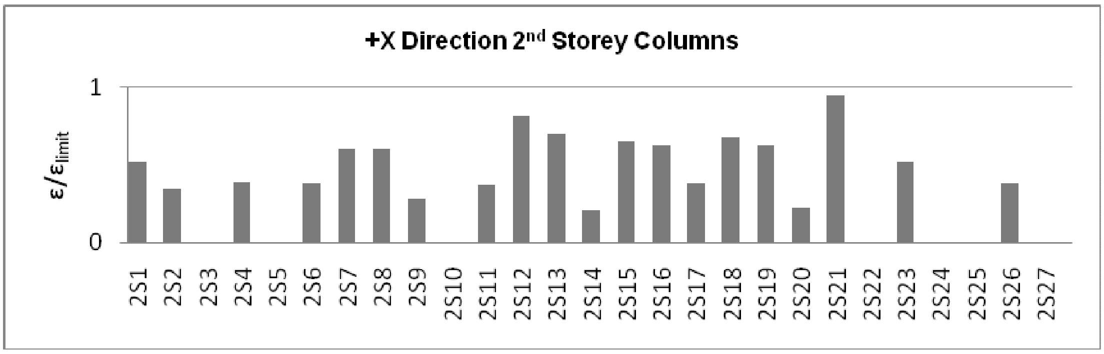
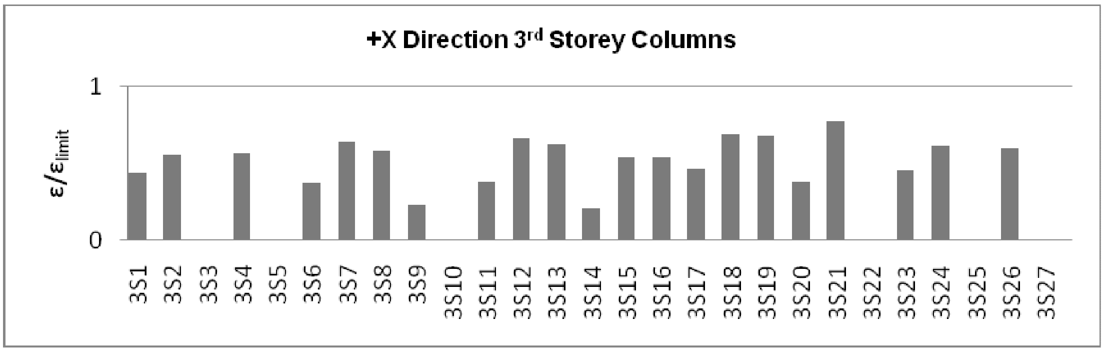


Figure C.8 r / r_{limit} for Beams (-Y Direction)







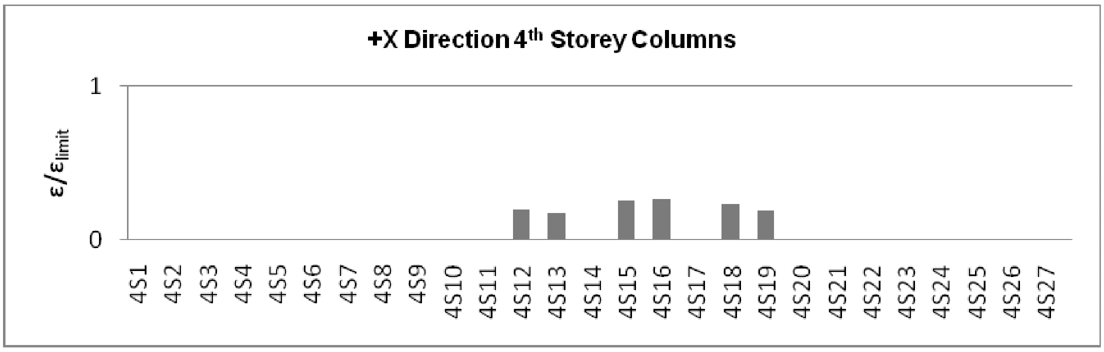


Figure C.9 ε / ε_{limit} for Columns (+X Direction)

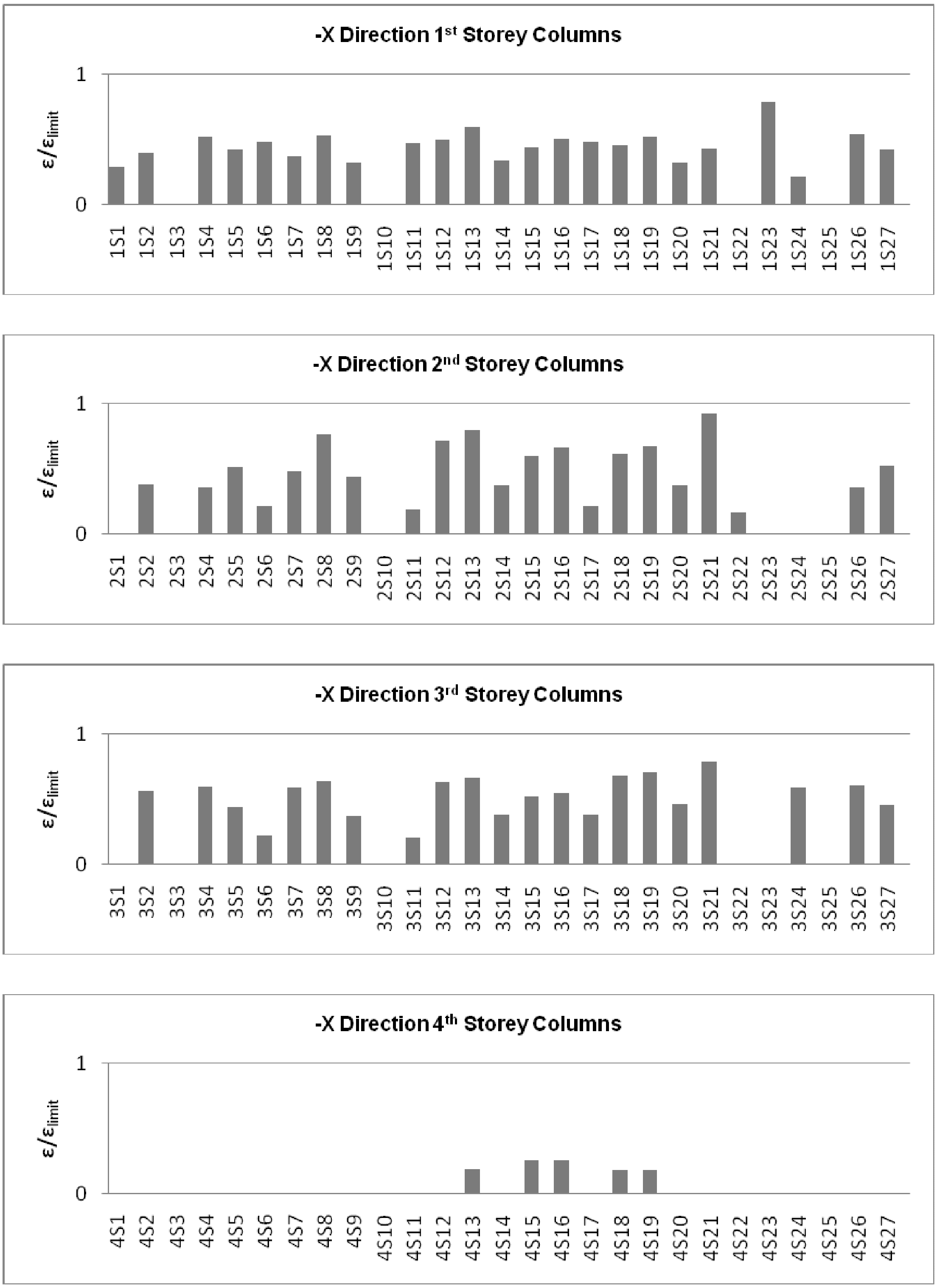
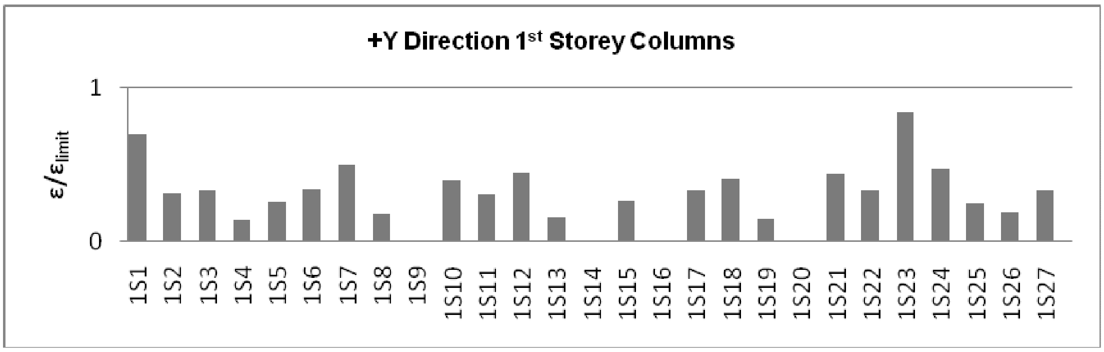
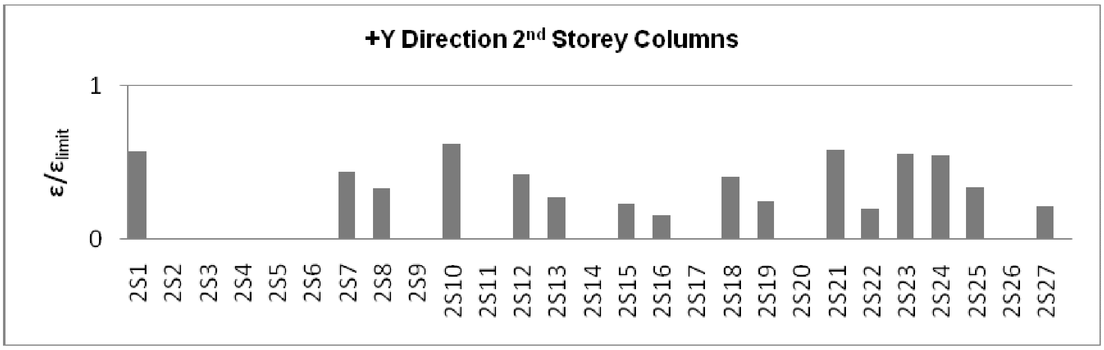
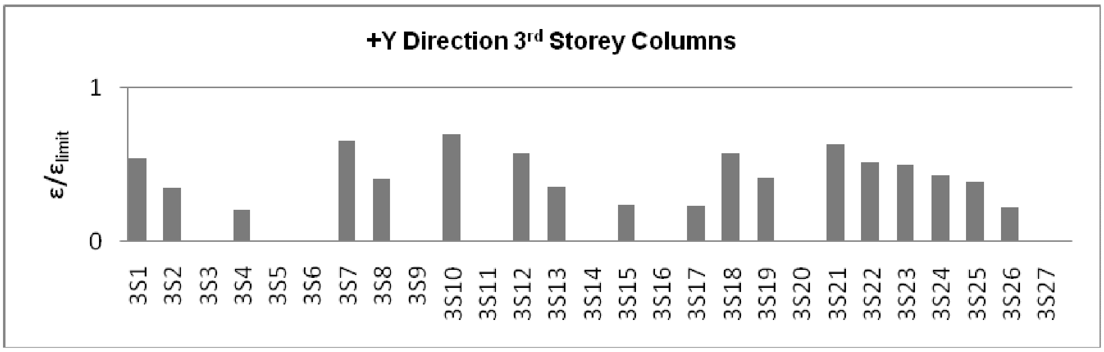


Figure C.10 ε / ε_{limit} for Columns (-X Direction)







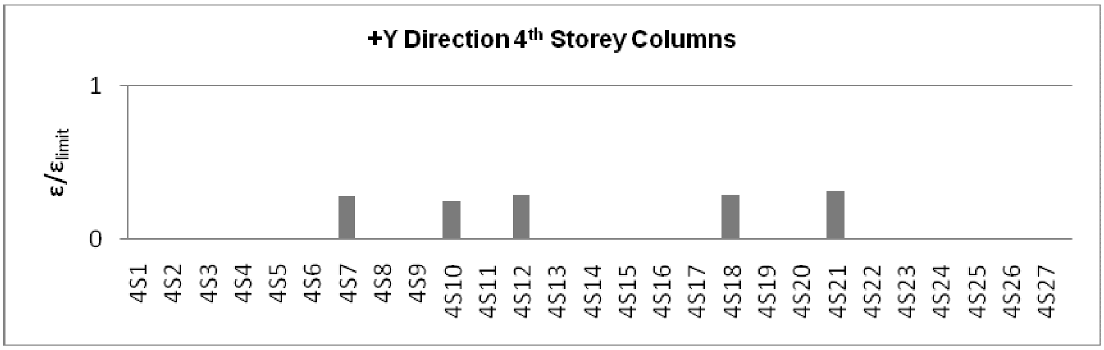


Figure C.11 ε / ε_{limit} for Columns (+Y Direction)

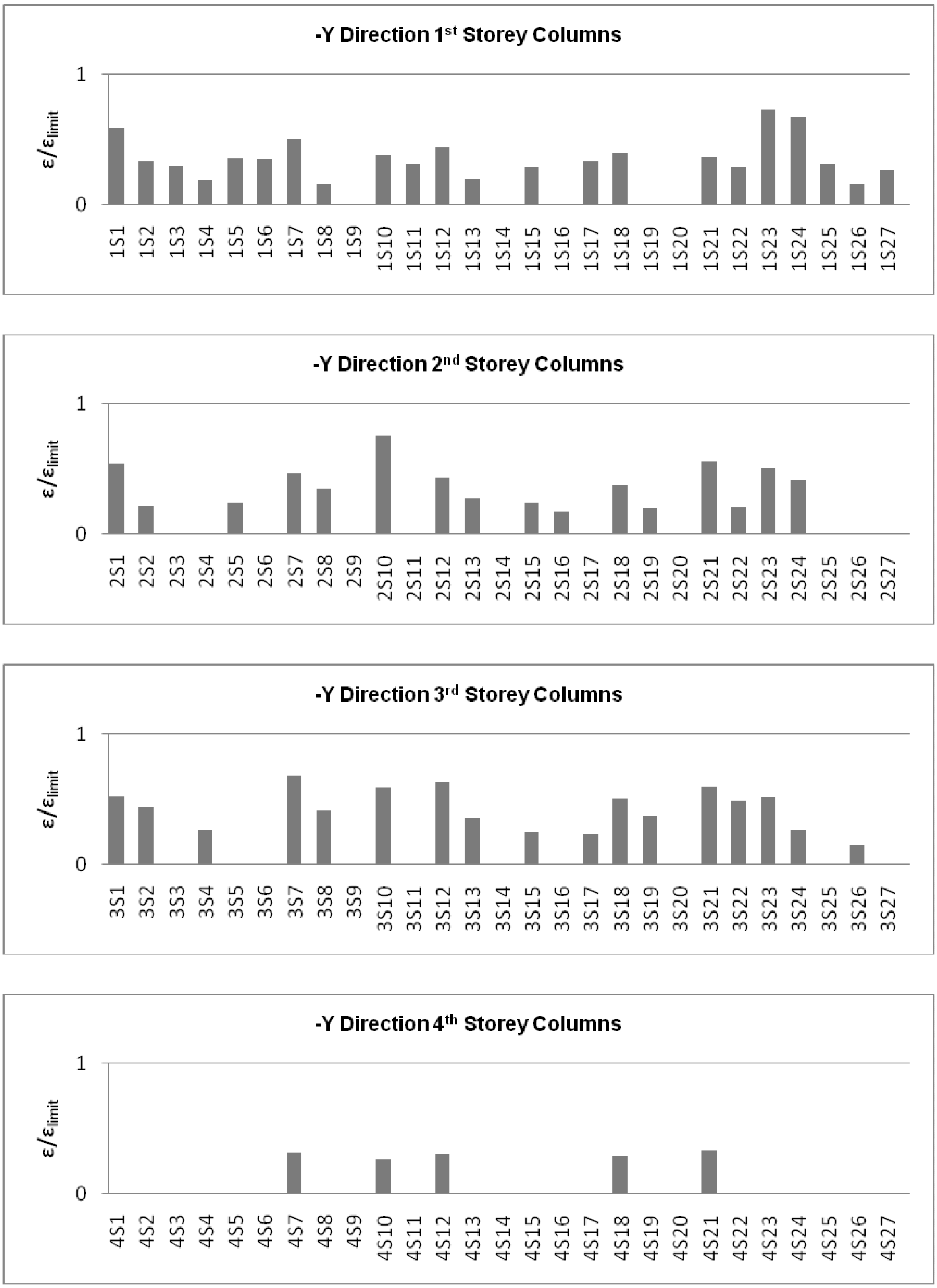
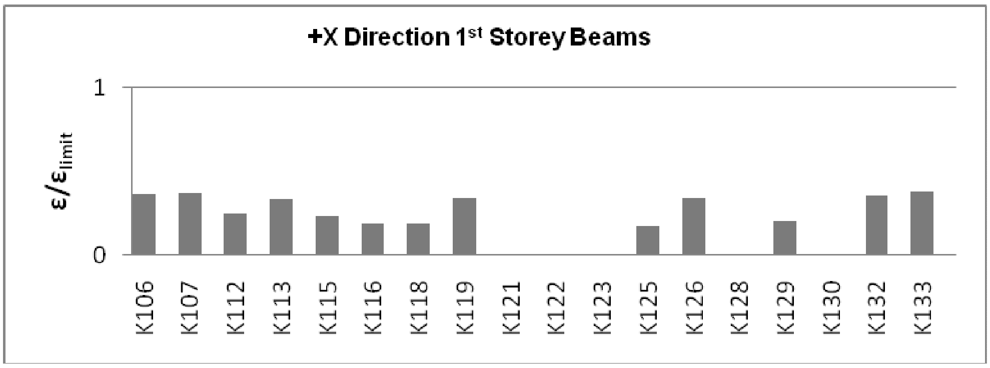
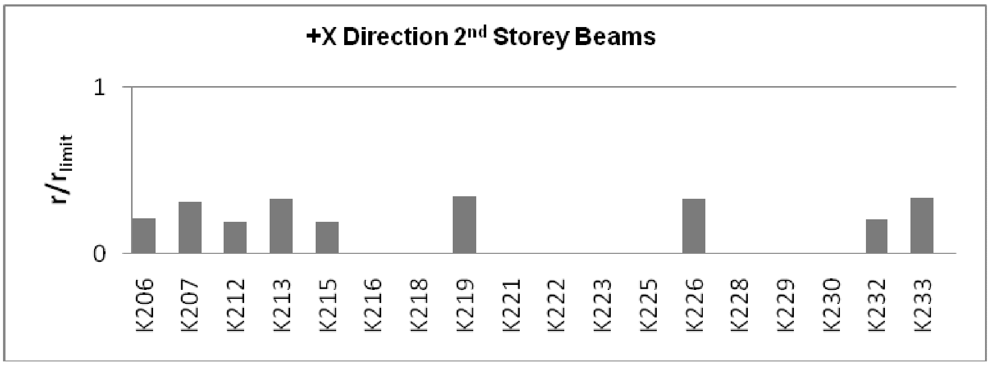


Figure C.12 ε / ε_{limit} for Columns (-Y Direction)





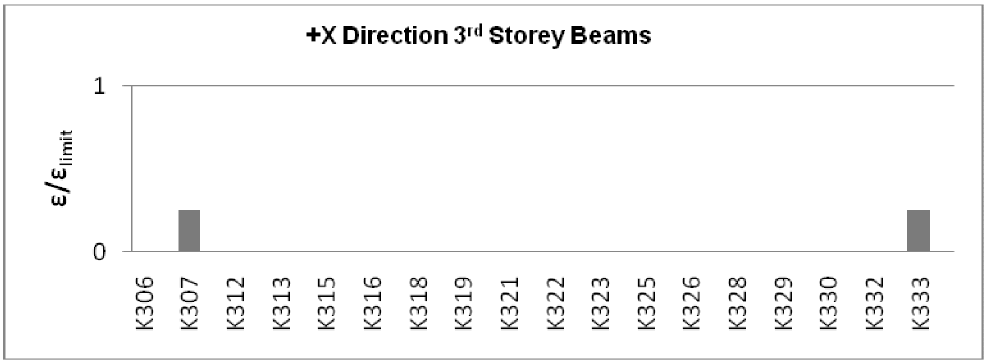


Figure C.13 ε / ε_{limit} for Beams (+X Direction)

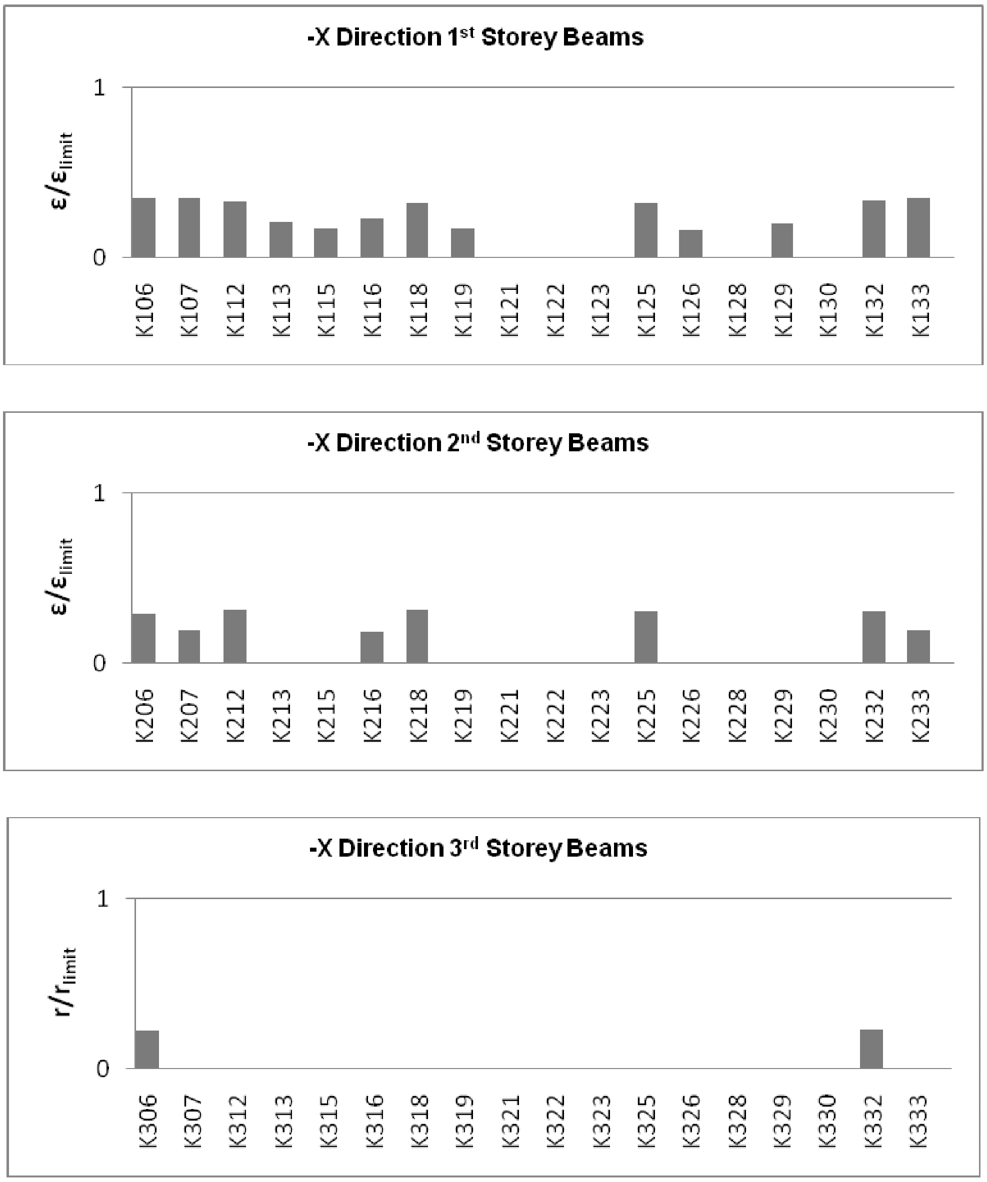
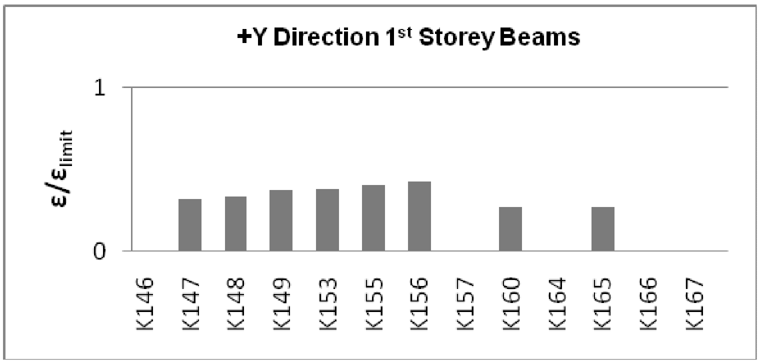
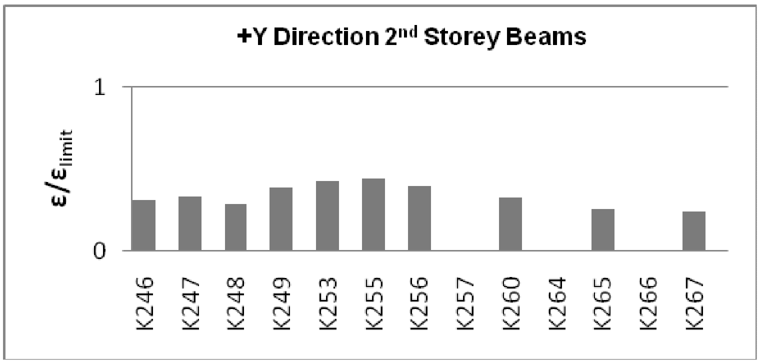


Figure C.14 ε / ε_{limit} for Beams (-X Direction)





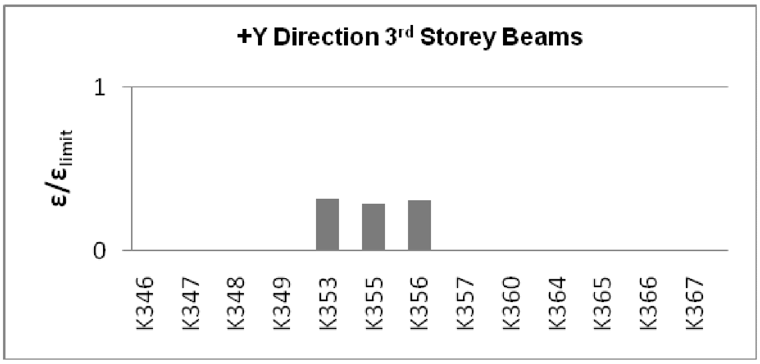
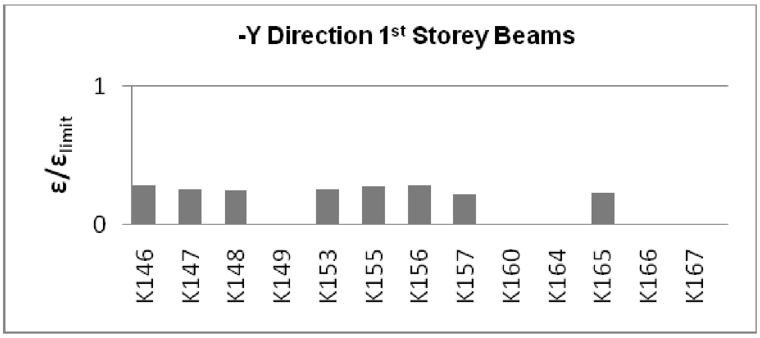
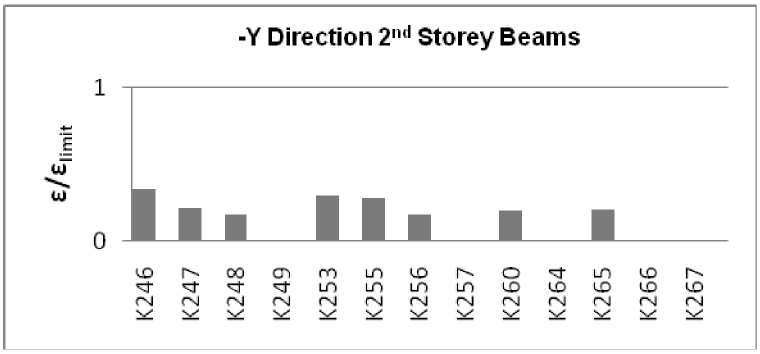


Figure C.15 ε / ε_{limit} for Beams (+Y Direction)





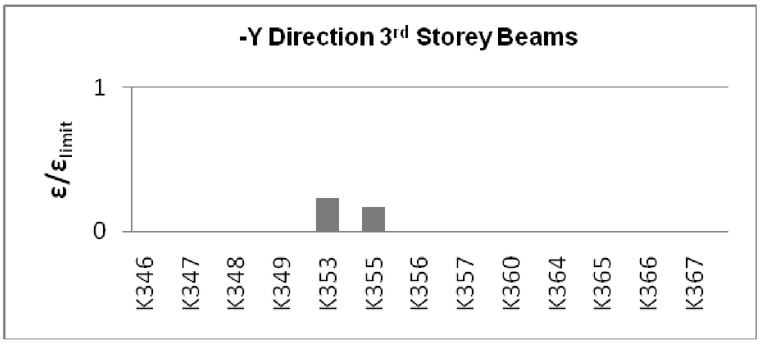


Figure C.16 ε / ε_{limit} for Beams (-Y Direction)