NONLINEAR ANALYSIS AND RETROFIT STRATEGY SELECTION OF A HISTORICAL REINFORCED CONCRETE BUILDING ACCORDING TO TURKISH EARTHQUAKE CODE 2007

by
İsmail Emrah Kılıç
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ABSTRACT

NONLINEAR ANALYSIS AND RETROFIT STRATEGY SELECTION OF A HISTORICAL REINFORCED CONCRETE BUILDING ACCORDING TO TURKISH EARTHQUAKE CODE 2007

The necessity of estimating the actual nonlinear behavior of the buildings during the seismic activity leads to a new earthquake code in Turkey. A new procedure, to evaluate seismic performance levels of the existing buildings, performance based design is introduced with the put in force of Turkish Earthquake Code 2007. The expected earthquake will also hit the historical buildings in the Marmara region, so that a historical reinforced concrete building ‘Haldun Taner Theatre Hall’ is chosen for the cased study in terms of seismic protection of historical buildings. The aim of this study is to evaluate the seismic performance of that building and to perform convenient retrofit strategy to improve the capacity to the desired level according to Turkish Earthquake Code 2007.

In this study; the material models which are used during the performance based design procedure, plastic hinge hypothesis, basis of ductility concept and the general statements and theory for the performance based design are explained to define the theoretical background of nonlinear analysis. Also to clarify the retrofit strategy selection procedure, the related factors and different retrofit strategies are explained.

In the case study, to determine the seismic demand and the target displacements with pushover analysis method; the capacity curve of the model is obtained; then it is converted to spectral acceleration-spectral displacement diagram (ADRS format) and it is compared with the earthquake demand in capacity spectrum method in the same diagram. After that the seismic performance evaluation of the building is done for two directions under two target performance levels (2% 50 years and 10% 50 years) according to TEC 2007. In the incremental equivalent seismic load method (push over analysis); the internal forces and deformation demand during the seismic activity and the resulted member damage levels

are specified directly. As a conclusion the building can not satisfy the *Immediate Occupancy Level* for the design earthquake (10% 50 years) and can not satisfy the *Life Safety Level* for the biggest earthquake (2% 50 years), also the building is in the collapse region for that target performance level.

The retrofit study of the building with shear walls and braced frames are explained respectively. There is not any section in the current code TEC2007 that exactly describes the retrofit strategies but this study gives an idea to estimate the effectiveness of the strategies to provide the desired building performance levels. There is not an exact comparison of these retrofit strategies in that study but a simple cost and construction time analysis is introduced in the last chapter. Both adding of shear walls and braced frames increase the lateral capacity and stiffness significantly which make the building reach the desired performance levels.

ÖZET

TARİHİ BETONARME BİR YAPININ TDY 2007 UYARINCA DOĞRUSAL OLMAYAN TAHLİLİ VE GÜÇLENDİRME YÖNTEMİNİN SEÇİMİ

Binanın deprem boyunca doğrusal olmayan gerçek davranışını tahmin edebilme gerekliliği ülkemizde yeni bir deprem yönetmeliğini gündeme getirmiştir. Bu yeni deprem yönetmeliğinin, Türk Deprem Yönetmeliği 2007 yürürlüğe girmesiyle binaların performans seviyelerini değerlendirmek için yeni bir yöntem olan performans esaslı tasarım tanıtılmıştır. Beklenen deprem Marmara Bölgesindeki tarihi eserleri de etkileyeceğinden, tez çalışması için tarihi binaların depreme karşı korunması başlığı altında 'Haldun Taner Sahnesi' seçilmiştir. Bu çalışmanın amacı Türk Deprem Yönetmeliğine göre binanın sismik performansını belirlemek ve binayı istenen sismik kapasiteye çıkaran uygun güçlendirme yöntemlerini belirlemektir.

Bu çalışmada performans esaslı tasarım sırasında kullanılan malzeme modelleri ve plastik mafsal hipotezi, süneklik kavramı ve performans esaslı tasarımın temel ilkeleri; doğrusal olmayan davranışı tariflendirmek için açıklanmıştır. Ayrıca farklı güçlendirme metotları ve bu metotların seçiminde kullanılan etmenler açıklanmıştır.

Performans esaslı tasarım yönteminde, deprem talebinin ve modal yerdeğiştime isteminin belirlenebilmesi; binanın kapasite eğrisinin elde edilmesi, bu eğrinin modal yerdeğiştirme- modal ivme eğrisine dönüştürülerek (ADRS formatı) depremin elastik davranış spektrumuyla karşılaştırılması sonucunda mümkün olur. Binanın sismik performansı iki ayrı deprem doğrultusunda ve iki ayrı performans düzeyi (50 yıl içinde aşılma olasılığı %2 ve %10 olan depremler) için belirlenir. Artırımsal eşdeğer deprem yükü metodunda, binada oluşan iç kuvvetler şekil değiştirmeler ve bunlardan kaynaklanan kesit hasarları doğrudan belirlenir.

Sonuç olarak bina 50 yılda aşılma olasılığı %10 olan deprem için *Hemen Kullanım* performans düzeyini ve 50 yılda aşılma olasılığı %2 olan deprem için de *Can Güvenliği* performans düzeyini sağlamamaktadır ve bu performans düzeyinde *Göçme* durumundadır.

Binanın perdelerle ve çelik çaprazlarla güçlendirilmesi sırasıyla anlatılmıştır. TDY 2007'de güçlendirme yöntemleriyle ilgili tam bir bölüm bulunmamaktadır fakat bu çalışma istenen performans düzeylerinin sağlanması için güçlendirme yöntemlerinin ne kadar etkin oldukları hakkında bir fikir vermektedir. Bu güçlendirme yöntemleri hakkında tam anlamıyla bir karşılaştırma yapılmamıştır fakat son bölümde basit bir maliyet ve zaman analizi yapılmıştır. Hem perdelerin hem de çelik çaprazların eklenmesi sistemin yanal kapasitesini ve rijitliğini önemli ölçüde artırarak binanın istenen performans seviyelerine gelmesini sağlamıştır.

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LIST OF SYMBOLS / ABBREVIATIONS

$A(T_1)$	Spectral acceleration coefficient calculated for T_1 period
A_0	Effective ground acceleration coefficient
A_e	Effective shear area
A_g	Effective shear wall area in a single storey
A_k	Effective wall area in a single storey
A_{sw}	Sectional area of the transverse reinforcement steel
A_w	Effective column area in a single storey
A_c	Gross section area of column
A_s	Cross sectional area of longitudinal steel reinforcement
A_{s1}, A_{s2}	Total area of tension reinforcement placed on one side of the beam-column joint at the top to resist the negative beam moment
a_1	Modal acceleration corresponding to first mode
$a_1^{(i)}$	Modal acceleration corresponding to first mode (in considered earthquake direction) in ith pushover step
a	Modal displacement, thickness of the weld
b	Width of section
b_w	Width of beam
C_{R1}	Spectral displacement ratio corresponding to first mode
d	Effective height of beam and column, modal displacement
d_i	Displacement calculated at ith storey of building under design seismic loads
d_1	Modal displacement corresponding to first mode
$d_1^{(i)}$	Modal displacement corresponding to first mode (in considered earthquake direction) in ith pushover step
E	Modulus of elasticity, earthquake load
E_c	Modulus of elasticity corresponding to concrete
E_s	Modulus of elasticity corresponding to steel

EI_0	Section stiffness for cracked section
e	Spacing between the centers of bolts
e_1	Spacing between the center of the bolt and end of plate
F_b	Buckling load on the brace
F_i	Equivalent earthquake loads acting to the floors
F_y	Axial load due to the yield stresses on the brace
f_{cc}	Compressive strength of confined concrete
f_{ck}	Characteristic compressive strength of concrete
f_{cm}	Existing compressive strength of concrete
f_{c0}	Compressive strength of unconfined concrete
f_{ctk}	Characteristic tensile strength of concrete
f_{ctm}	Existing tensile strength of concrete
f_{su}	Ultimate strength of reinforcing steel
f_{sy}	Yield strength of reinforcing steel
f_{yk}	Characteristic yield strength of reinforcing steel
g	Dead load
g_i	Total dead load at i^{th} storey of building
Hi	Height of i^{th} storey of building measured from the top foundation level
h	Effective height of the section
h_i	Height of i^{th} storey of building
I	Moment of inertia of the section, building importance factor
i	Radius of gyration
l	Total length of the brace, length of the weld
l_p	Length of plastic hinge
M	Bending moment
M_1	Modal mass corresponding to first natural vibration mode
M_p	Plastic moment capacity of the section
M_p	Yield moment capacity of the section

m_i	Mass of i^th storey of building
N	Normal force, total number of stories of building from the foundation level
n	Live load participation factor, number of bolts in a bolted connection
P	Normal force
q	Live load
q_i	Total live load at i^th storey of building
R	Structural behavior factor
R_a	Seismic load reduction factor
$R_a(T_1)$	Seismic load reduction factor corresponding to T_1 period
S_a	Spectral acceleration
S_{ae1}	Linear elastic spectral acceleration corresponding to first mode
S_d	Spectral displacement
S_{de1}	Linear elastic spectral displacement corresponding to first mode
S_{di1}	Nonlinear spectral displacement corresponding to first mode
$S(T_1)$	Spectrum coefficient
s	Buckling length
T_1	First natural vibration period of building
$T_1^{(1)}$	First natural vibration period of building corresponding to first mode
T_{1x}	First natural vibration period of building corresponding to first mode of building in the x earthquake direction considered
T_{1y}	First natural vibration period of building corresponding to first mode of building in the y earthquake direction considered
T_0	Spectrum characteristic period
T_A, T_B	Spectrum characteristic periods
t	Thickness of the section
$u_{xN1}{}^{(i)}$	Roof displacement in i^th pushover step along x direction corresponding to first mode
$u_{yN1}{}^{(i)}$	Roof displacement in i^th pushover step along y direction corresponding to first mode

V	Shear force
V_b	Base shear force
V_e	Shear force taken into account for the calculation of transverse reinforcement of column or beam
V_{kol}	Smaller of the shear forces at above and below the joint calculated
V_t	In the Equivalent Seismic Load Method, total equivalent seismic load acting on the building (base shear) in the earthquake direction considered
$V_{xN1}{}^{(i)}$	Base shear force in i^{th} pushover step along x direction corresponding to first mode
$V_{yN1}{}^{(i)}$	Base shear force in i^{th} pushover step along y direction corresponding to first mode
W	Total building weight, Elastic section modulus
W_i	Total weight of i^{th} story of the building including live loads multiplied by related participation factors
w	Buckling coefficient
w_i	Total weight of i^{th} story of the building
ΔF_N	Additional equivalent seismic load
Δi	Interstorey drift value
Δ_T	Axial deformation
δ	Lateral displacement
ε	Unit deformation
ε_c	Concrete strain
ε_{c0}	Compression strain of unconfined concrete
ε_{cc}	Compression strain of confined concrete
ε_{cg}	Compression strain for confined region of concrete
ε_{cu}	Ultimate compression strain of concrete
ε_s	Strain for reinforcing steel
ε_{sh}	Strain for reinforcing steel at the beginning of strain hardening
ε_{su}	Ultimate strain for reinforcing steel

ε_{sy}	Yield strain for reinforcing steel
ϕ_p	Plastic curvature demand
ϕ_t	Total curvature demand
ϕ_u	Ultimate curvature demand
ϕ_y	Equivalent yield curvature
Φ_{xN1}	Mode shape of first mode corresponding to N^{th} story in x direction
Φ_{yN1}	Mode shape of first mode corresponding to N^{th} story in y direction
Γ_{xN1}	Modal contribution coefficient corresponding to first mode in x direction
Γ_{yN1}	Modal contribution coefficient corresponding to first mode in y direction
σ_y	Yield stress of the steel section
σ_{all}	Allowable compression/tension stress for steel sections
τ_{all}	Allowable shear stress for steel sections
η_{bi}	Torsional irregularity factor defined at i^{th} storey of building
η_{ci}	Strength irregularity factor defined at i^{th} storey of building
η_{ki}	Stiffness irregularity factor defined at i^{th} storey of building
λ	Slenderness ratio of steel columns
θ_p	Plastic rotation demand
μ	Ductility factor
ρ_s	Volumetric ratio of existing transverse reinforcement steel
ρ_{sm}	Volumetric ratio of transverse reinforcement that is required for design of a new building
$w_1^{(1)}$	Angular frequency corresponding to the first mode in first step (i=1) of pushover analysis
w_B	Angular frequency corresponding to the characteristic period of acceleration spectrum

ATC	American Technology Council
ASD	Allowable Stress Design
BSSC	Building Seismic Safety Council
CG	Life Safety Level
CR	Collapse Region
GÇ	Collapse Level
GÖ	Collapse Prevention Level
FEMA	Federal Emergency Management Agency
HK	Immediate Occupancy Level
HZ	Loading type with vertical and lateral load effects
MDR	Minimum Damage Region
RC	Reinforced Concrete
SAP	Structural Analysis Program
SDOF	Single Degree of Freedom
SDR	Significant Damage Region
SEAOC	Structural Engineers Association of California
TS-498	Design Loads for Buildings
TS-500	Requirements for Design and Construction of Reinforced Concrete Structures
TEC 2007	Turkish Earthquake Code 2007
XTRACT	Cross Section Analysis Program for Structural Engineers
VDR	Visible Damage Region

1. INTRODUCTION

1.1. General Concepts

In the recent years Turkey has experienced disastrous earthquakes which cause a big human tragedy, as Erzincan Earthquake (1992), Afyon-Dinar Earthquake (1995), Adana-Ceyhan Earthquake (1998), Marmara Earthquake (1999) and Bolu-Düzce Earthquake (1999). These earthquakes cause thousands of fatalities and injuries also the Marmara Earthquake that occurs in the densely populated and industrialized region cause huge economic losses. These earthquakes also cause high damage on the historical heritage of the country.

As the results of these earthquakes are compared to their equivalents in the developed countries, it can be seen that in our country earthquakes results in much more damage and economic loss. This low seismic performance of buildings can be reasoned by the usage of low quality material, design not in accordance with code or design not in accordance with engineering principles and insufficient inspection during design and construction. Because of the facts above and also in consideration the statistical data that there will be a destructive earthquake in the Marmara Region in the following 30 years; the existing buildings must be investigated to evaluate their seismic performance levels and the required retrofit strategies must be performed as far as possible.

Other than Turkey, destructive earthquakes 1971 San Fernando, 1989 Loma Prieta, 1994 Northridge cause high economic loses with afflictive fatalities and damages. These earthquakes also showed that the current code designs are insufficient in the case of seismic resistance of buildings and estimating the behaviors of the structures during the earthquake. This condition leads to develop new aspects for aseismic design of structures and lots of projects have been developed till today. The projects which can be agreed as the most important for the development of a new model ‘Performance Based Design’ are Vision 2000(1995) a project of the Structural Engineers Association of California (SEAOC), ATC-40 (1996) was prepared by Applied Technology Council, FEMA 273-274 (1997) which was issued by Building Seismic Safety Council (BSSC). Also in Turkey, the most important innovation of the new Turkish Earthquake Code is the addition of thes performance based design procedure. In the Turkish Earthquake Code which was published in 06.03.2007 and revised in 03.05.2007, both the linear and nonlinear approaches to evaluate the seismic performance of the buildings take place.

Performance based design is based on non linear response as the building responds the ground motion in a non linear manner. The existing buildings were designed regarding the force based design but the evaluation of the seismic performances must be with performance based design to predict more realistic non linear behavior of the building during the seismic activity. Another advantage of the performance based design is the variation of desired performance levels. According to the occupancy or importance of the building the performance goals can be changed and more realistic and economic designs or retrofit studies can be performed.

1.2. Scope and Objective

In this study, the building which was designed according to the force based approaches is evaluated with the performance based design according to TEC 2007 for determining the seismic performance level. The case study building is ‘Haldun Taner Theater Hall’ which has a historical importance and which is one of the symbol structures of Kadıköy, İstanbul. This case study is important because it is an example for the seismic protection of historical heritage. Also it is a sample to investigate buildings with limited information levels and with limited information about the reinforced concrete design and engineering aspects of the term that building was constructed.

‘Haldun Taner Theatre Hall’ which can be accepted as a monument for Kadıköy was designed by the Italian architect Ferrari and it was constructed in 1927. The building which was a survival building of the ‘First National Architectural Style’ was constructed as fruit and vegetable storage and also served as fire department building. Recently, this old building is again on the agenda because of its demolition decision.

In the second chapter of this study; the material models which are used during the performance based design procedure, plastic hinge hypothesis, basis of ductility concept and the general statements and theory for the performance based design are explained. In the next chapter, seismic performance evaluation methodology for existing buildings according to Turkish Earthquake Code 2007 is explained.

The fourth chapter is about the definition of retrofit systems, different retrofit strategies, the factors that affects the selection of retrofit strategy. In the case study the irregularities of the example building is investigated, then the dynamic properties and the plastic hinge formations are defined to evaluate seismic performance of the existing building. For this evaluation incremental equivalent seismic load method (push over analysis) is used by which the internal forces and deformation demand during the seismic activity and the resulted member damage levels are specified directly. To determine the seismic demand and the target displacements with pushover analysis method; the capacity curve of the model is obtained, then it is converted to spectral acceleration-spectral displacement diagram (ADRS format) and it is compared with the earthquake demand in capacity spectrum method in the same diagram. After that the seismic performance evaluation of the building is done for two directions under two target performance levels according to TEC 2007. The nonlinear applications of the SAP 2000 V11.0.0 structural analysis program and the section analysis program XTRACT are used for these analyses.

In the next two chapters of this study, the retrofit study of the building with shear walls and braced frames are explained respectively. There is not any section in the current code TEC2007 that exactly describes the retrofit strategies but these two sections give an idea to estimate the effectiveness of the strategies to provide the desired building performance levels under different target performance demands with the incremental equivalent seismic load method. There is not an exact comparison of these retrofit strategies but a simple cost and time analysis is introduced in the last chapter and the results of the performance levels of the retrofitted models, give an idea about the abilities of these two strengthening and stiffening type retrofit strategies.

2. CONCEPTS AND THEORY FOR THE PERFORMANCE BASED DESIGN

2.1. Material Models and Stress-Strain Relationships

In the following sections, material models and stress-strain relationship for concrete and steel reinforcements are explained in details.

2.1.1. Material Modeling Concept for Concrete

A stress-strain model is developed for concrete subjected to uniaxial compressive loading and confined by transverse reinforcement. The concrete section may contain any general type of confining steel; either spiral of circular hoops; or rectangular hoops with or without supplementary cross ties. These cross ties can have either equal or unequal confining stresses along each of the transverse axes. A single equation is used for the stress-strain equation. The influence of various types of confinement is taken into account by defining an effective lateral confining stress, which is dependent on the configuration of the transverse and longitudinal reinforcement. An energy balance approach is used to predict the longitudinal compressive strain in the concrete corresponding to first fracture of the transverse reinforcement by equating the strain energy capacity of the transverse reinforcement to the strain energy stored in the concrete as a result of the confinement [1]. In TEC 2007 the stress strain relationship is defined for the confined concrete and unconfined concrete, to evaluate the building by performance based design the code is said that the 'Mander Concrete Model' will be an alternative for modeling the material properties. Figure 2.1 which is referred in the current code; ε_{co}, defines the diagram for the strain of unconfined concrete under the maximum compression stress, ε_{cc}, the strain of confined concrete under the maximum compression stress, ε_{cu}, the maximum strain value of the confined concrete, f_{co}, compression strength for the unconfined concrete and f_{cc}, compression strength for the confined concrete.

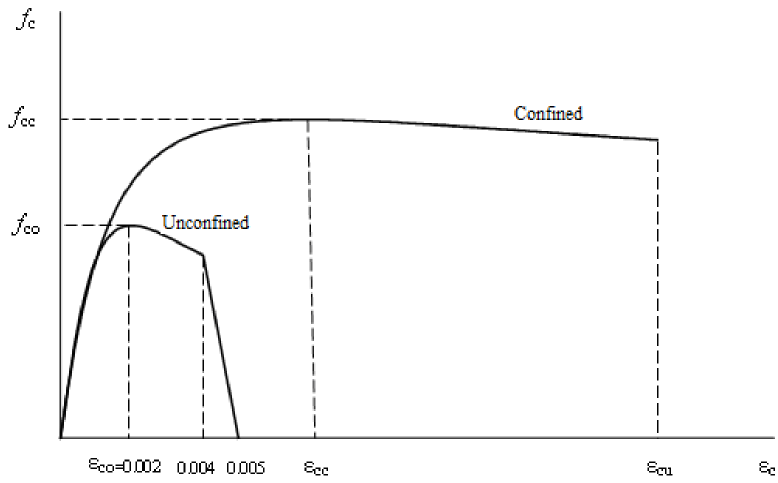

Figure 2.1. Stress-strain relationship of concrete [13]

2.1.2. Material Modeling Concept for Reinforcement

Figure 2.2 shows the diagram of axial stress-strain relationship of reinforcement regarding the definition in TEC 2007. According to that diagram stress-strain relationship consists of three regions. These regions can be named as elastic region, plastic plateau region and strain hardening region. In the diagram ε_{sy}, ε_{sh}, ε_{su}, f_{sy} and f_{su} are referring respectively yielding strain of steel reinforcement, strain of at the beginning of strain hardening, the ultimate strain of reinforcement, yielding strength of steel reinforcement and the ultimate strength of reinforcement. And the related strain and strength values for S220 steel and S420 steel can be seen in Table 2.1 regarding TEC 2007.

Table 2.1. Strain and strength values for S220 and S420 [13]

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

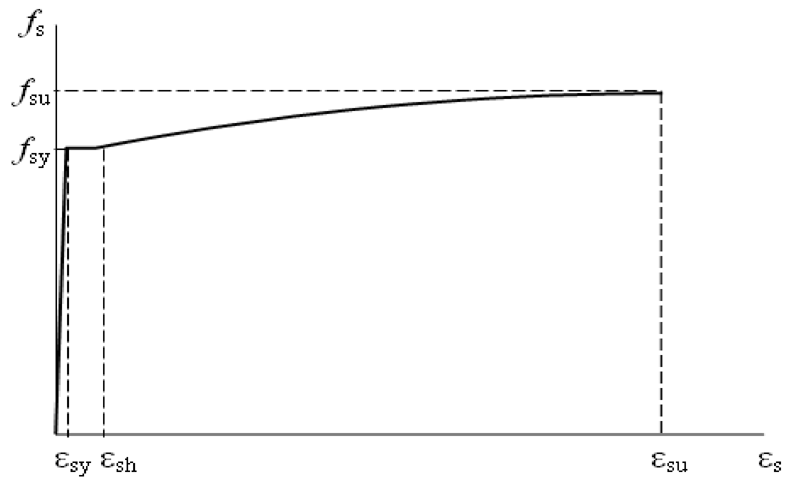

Figure 2.2. Stress-strain relationship of reinforcement [13]

2.2. Theory of The Plastic Hinge Hypothesis

In the frame elements a curvature forms because of the moment effect. This curvature is accepted as elastically proportional to the strain values in Figure 2.2 until it reaches its yield capacity. After the yield capacity of reinforcement; the reinforced concrete section can rotate to an allowable limit regarding material properties, with little increases in the moment value. It is obvious that this yielding moment value occurs not at a specific point but along a specific length and this statement points out that the above behavior which can be named as plastification occurs between any two points, A and B. The rotation of these points with respect to each other can be calculated by Equation 2.1 as the integration of the curvature along the beam.

$$\theta_{BA} = \int_A^B \phi_t \; dx = \int_A^B (\phi_{elastic} + \phi_{plastic}) \; dx = \int_A^B \phi_{elastic} \; dx + \int_A^B \phi_{plastic} \; dx \tag{2.1}$$

In most of the cases the plastic curvature is much bigger as compared to elastic curvature only the plastic curvature can be taken into account. The plastic hinge length in the plastified region can be calculated by dividing the calculated biggest plastic curvature value by the total curvature value as it can be seen in Equation 2.2 [2].

$$L_p = \theta_{BA} = \frac{\theta_{BA\ plastik}}{\phi_{p\ \max}} = \frac{1}{\phi_{p\ \max}} \int_A^B \phi_{plastik}\ dx \tag{2.2}$$

Determining the plastic hinge length with the Equation 2.2, dissipates the simplicity of the plastic hinge idealization. In TEC 2007, the length of plastic deformation region named as plastic hinge length is determined by the half of the section dimension along the performed direction.

The following assumptions can be done to represent the mathematical model of the nonlinear behavior of the material [3].

- Plastic deformations are represented at the end points of frames which are plastic regions without length.
- Reinforcement steel is assumed as elasto-plastic without any strain hardening case.
- The effect of shear stresses and the normal stresses orthogonal to frame axes, on plastic deformations are neglected.
- As a result of the above statements; the axial load and bending load that start the plastic rotation, cause the transition of the plastic region from elastic case to plastic case suddenly.
- Plastic deformations occur orthogonal to the yield surface of section.

2.3. Basis of Ductility

The most agreed opinion for the aseismic design of structures all over the world is that, during a strong ground motion the non elastic behavior of the system gains importance. Through this plastic behavior which can be named as *ductility*, the energy occurred by the ground motion is absorbed by high amplitude vibrations. The numerical definition of the ductility can be made by the ratio of ultimate load limit to the elastic strain limit as it can be seen in Equation 2.3.

$$\mu = \frac{\delta_u}{\delta_y} \tag{2.3}$$

Ductility can be defined as an effect and its opposing strain and it can be classified as the section based ductility and the system based ductility [2].

2.3.1. Section Based Ductility

In a reinforced concrete section with bending, the ratio of the biggest curvature to yield curvature without an important strength decrease for the section can be named as curvature ductility and it can be calculated by Equation 2.4

$$\mu = \frac{\phi_u}{\phi_y} = \frac{\varepsilon_{cu} / x_u}{\varepsilon_y /(d - x_y)} = \frac{\varepsilon_{cu}\ (d - x_y)}{\varepsilon_y\ x_u} \tag{2.4}$$

For the Equation 2.4; ε_{cu}, means the maximum compression strain for confined concrete; ε_y, defines the yielding strain of the steel reinforcement; d, defines the available height of the section; x_y, means the neutral axis height at yielding; x_u, means the neutral axis height at the ultimate point when the most outer concrete fiber reaches its maximum shortening.

Through the same equation; values for $\varepsilon_{cu}, \varepsilon_y, d$ does not change but due to an increase in compression x_y and x_u values also increase which cause a decrease in ductility. Also by defining an upper limit for the tension reinforcement of the section, the ductility can be provided as ensuring the yielding of reinforcement before the concrete is crushed under compression. Curvature ductility of the section can be found by curvature values that coincide with the moment values changing under different axial loads [4].

2.3.2. System Based Ductility

The structural system responses the demand displacement of the ground motion through a specific damage formation. This target displacement demanded by the earthquake is approached by the plastic rotations of the sections. This condition can be named as the system lateral displacement ductility. The sections help one another by means of ductility and in the further steps of the displacement; sections exceed the elastic limit absorb energy by plastic deformations and the internal loads are transferred to the less strained sections [3]. Because new hinge formations take place during the system behaves in a ductile manner, static indeterminacy degree of the system must be high enough. The system can behave ductile to the collapse limit regarding the above statement. The ultimate load capacity, at the target displacement demanded by ground motion, of the members and the structural system must be reached in a ductile way to evaluate the required performance level of the building.

2.4. General Statements for the Performance Based Design

Civil engineers all around the world are, by tradition, trained as “linear engineers”. Consequently, the “nonlinear” seismic evaluation or design process is “linearized” basically in two ways: Either engineers are told to run linear analysis under somehow reduced “equivalent seismic loads” as in the traditional seismic design code applications, or alternative procedures are offered where the seismic behavior is again “linearized” in a “dynamic sense” by employing the “secant stiffness” and an “equivalent damping ratio” of a single-degree-of-freedom (SDOF) “substitute structure”.

On the other hand, pushover-based seismic evaluation and design methods appear to provide a great opportunity for engineers to consider and actually calculate and evaluate the consequences of nonlinear response in a direct manner. It is most likely that those methods will eventually prove to be superior over the so called “direct displacement based design” procedure based on artificially linearized “substitute structures”. As a matter of fact, pushover-based methods created a great deal of enthusiasm in engineering community when they were re-invented in the last decade for the purpose of estimating seismic deformation demands. Yet, they have not found a widespread acceptance in practice. Engineers still treat seismic evaluation or design efforts through such methods as advanced applications and think of pushover analysis itself as a complex and difficult to understand procedure. Generally, they are not well-informed on how the procedure is actually handled in computer programs. Pushover analysis is still treated as a “black box” [5].

Performance based design appears to be the future direction of seismic design codes. In the newly developed performance based seismic design approach, nonlinear analysis procedures become important in identifying the patterns and levels of damage for assessing a structure's inelastic behavior and for understanding the failure modes of the structure during severe seismic events. Pushover analysis is a simplified, static, nonlinear procedure in which a predefined pattern of earthquake loads is applied incrementally to framework structures until a plastic collapse mechanism is reached. This analysis method generally adopts a lumped plasticity approach that tracks the spread of inelasticity through the formation of nonlinear plastic hinges [6].

As graphically presented in Figure 2.3, the nonlinear static analysis procedure requires determination of three primary elements: capacity, demand and performance. The capacity spectrum can be obtained through the pushover analysis, which is generally produced based on the first mode response of the structure assuming that the fundamental mode of vibration is the predominant response of the structure. This pushover capacity curve approximates how a structure behaves beyond the elastic limit under seismic loadings. The demand spectrum curve is normally estimated by reducing the standard elastic 5% damped design spectrum by the spectral reduction method. The intersection of the pushover capacity and demand spectrum curves defines the "performance point" as shown in Figure 2.3. At the performance point, the resulting responses of the building should then be checked using certain acceptability criteria. The responses can be checked against acceptability limits on both global system levels (such as the lateral load stability and the inter story drift) and local element levels (such as the element strength and the sectional plastic rotation) [7]. When the responses of a structure do not meet the targeted performance level, the structure needs to be resized and the design process repeated until a solution for the desired performance level is reached. In general, the determination of the satisfactory performance response that fulfills both the system level response and element level response requires a highly iterative trial and error design procedure even with the aid of today's engineering computer software.

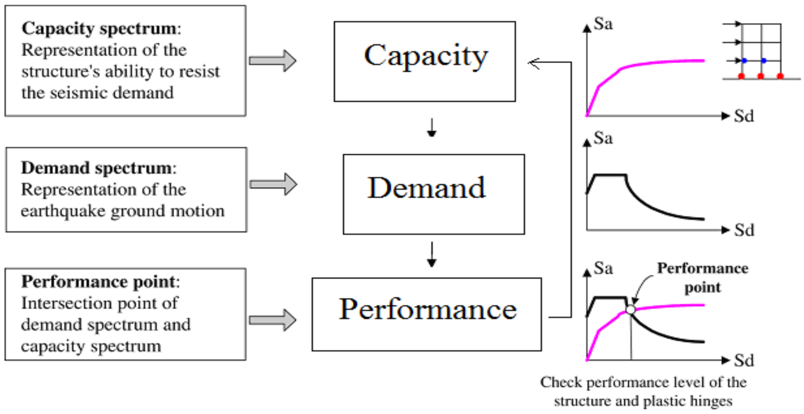

Figure 2.3. Nonlinear analysis procedure [7]

It has been recognized that the inter story drift performance of a multistory building is an important measure of structural and non-structural damage of the building under various levels of earthquake motion [8]. In performance based design, inter story drift performance has become a principal design consideration. The system performance levels of a multistory building are evaluated on the basis of the inter story drift values along the height of the building under different levels of earthquake motion. The control of inter story drift can also be considered as a means to provide uniform ductility over all stories of the building. A large story drift may result in the occurrence of a weak story that may cause catastrophic building collapse in a seismic event. Therefore, uniform story ductility over all stories for a multistory building is usually desired in seismic design [9].

Although lateral drift performance is a principal concern in the seismic design of structures, economically designing elements of building structures for various levels of elastic and inelastic lateral drift performance under multiple levels of earthquake load is generally a rather difficult and challenging task [10]. Lateral drift design requires the consideration of a proper distribution of the stiffness of all structural elements and, in a severe seismic event, also the occurrence and redistribution of plasticity in the structural elements.

Performance based design concept in which the performance is the measure of the damage that the building sustain, define the multiple target performance levels which are expected to be achieved, when the structure is subjected to specified earthquake ground motion. Those damage levels are determined by engineers together with users and the architects based on the consequences of the damage faced by the user community. Here, damage is expressed in terms of inelastic deformation limits for various structural components of the building. Instead of comparing forces, nonlinear static procedures use displacement of the components to compare seismic demand with the capacity of a structure.

A performance level is defined as the maximum desired extent of probable damage of a building when confronted by a specified ground motion. Building performance levels are expressed in two but related ways [11]:

- In technical terms useful for engineering design and evaluation including the extent of deterioration and degradation permissible to individual structural and non structural elements, as well as overall global behavior of the structural systems.
- In qualitative terms meaningful to lay public including owners, occupants and the policy makers.

As a conclusion, this new analysis can estimate the actual behavior of the building more accurately during a specific ground motion and results in a more effective and cost efficient retrofit.

It has been long recognized that the structural behavior and damageability of structures during earthquakes is essentially controlled by the inelastic deformation capacities of the ductile structural elements. This eventually led to a notion that the seismic evaluation and design of structures should be based on displacements (or more correctly on deformations) demanded by the earthquake action, not on the stresses induced by the assumed equivalent seismic forces. In spite of this recognition the current seismic design practice is still governed by the force based design principles. Nevertheless significant attempts have been made in the last decade to incorporate the displacement based evaluation and design concept into the seismic engineering practice. Those attempts are developed in two interrelated but different directions yielding the displacement based design methods aiming at direct design of new structures and the displacement-based evaluation methods dealing with the seismic performance evaluation of pre-designed or existing structures [12].

3. TEC 2007 APPROACH TO THE PERFORMANCE BASED DESIGN

Performance based design helps describing the inelastic behavior of the structural component of a building. By this approach the actual behavior of a building can be estimated more accurately during a specified ground motion. Since all the structural members are examined individually in performance design procedures it is easy to see which member or member group does not satisfy the desired performance level.

This design technique has two main parameters one is the demand which represents the ground shaking motion that affects to the structure; the other is the behavior of the structure under this ground shaking motion which can be named as capacity of the structure.

It is essential to determine the seismic performance levels of existing buildings and the case that they can not reach the desired performance levels and retrofit strategies must be decided according to the performance based design procedure. In United States this design procedure is improved in recent years to determine the actual seismic performances of the buildings according to their codes, ATC40 and FEMA 356 which explain the performance levels, capacity and demand issues.

After 1999 Marmara Earthquake, it is seen obviously that the existing earthquake code is not adequate to figure out the seismic performance of the existing buildings. To understand the performance levels of the buildings and to take precautions for the earthquake in terms of retrofit strategies the earthquake code was revised. The new code which includes the performance based design was published at 06.03.2007 and also partially revised at 03.05.2007. This section of the study states the information of the performance based design according to TEC 2007 [13].

3.1. The Scope of Data Collected From Buildings

The dimensions and details of members, data about structural system geometry and material properties to be used in determining the capacity of the structural members of existing buildings and evaluating the seismic performance of them shall be obtained from projects and reports of the buildings, observations and measurements to be made in the building and experiments to be applied to the material samples obtained from the buildings.

The processes to be performed in the scope of the obtaining data from buildings are defining the structural system, determination of foundation system, geometric properties and soil properties of the building, determination of the existing damage and changes and / or repairs performed, measurements of dimension of the members, determination of the material properties, controlling of whether all data obtained in field are in compliance with the buildings' project.

3.2. Information Levels

Information levels shall be classified as *limited, medium* and *detailed information levels* respectively. Information levels obtained affect the choice of the design method to be used in determining the seismic performances of the buildings.

- *Limited Information Level*, structural working drawings of the building do not exist. Structural system properties are determined by the measurements taken from the building.
- *Medium Information Level*, if structural working drawings of the building do not exist, more measurements are performed with respect to limited information level. If projects of the building exist, measurements defined in limited information level are performed and the project is verified.
- *Detailed Information Level*, structural working drawings of the building exist. Necessary measurements are taken to confirm the information about the project.

3.2.1. Existing Material Strength

Existing material properties to be used in determining the capacities of the structural members is defined as *existing material strength.*

3.2.2. Limited Information Level in Reinforced Concrete Buildings

Building Geometry: The plan of the structural system is prepared by performing a site survey. In case the architectural projects exist, they are used as assistance for the site surveys. The information obtained must include locations, spans, heights and dimensions of all reinforced concrete members and partitions and must be sufficient for modeling of the building. Foundation system is observed through a control pit to be opened at corner of the building. Short columns and similar irregularities in the building are marked on the floor plan. Relation with neighbor buildings (separated, adjacent, joint exists / does not exist) is determined.

Member Details: Structural working drawings or shop drawings of building do not exist. Details of the reinforced concrete members are assumed to be in compliance with the minimum reinforcement requirements at the time of construction of the building. In order to verify this assumption or in order to determine the rate of accuracy of it, reinforcement control is carried out on 10 percent of the columns and 5 percent of the beams on each storey not being less than one, by peeling off the concrete cover shares on two adjacent faces. The peeling off operation shall be carried out on the one third parts in the middle of the span of the length of columns and beams. Surfaces that are peeled off shall be filled later with high–strength repairing plaster.

Material Characteristics: In each story, at least two concrete samples shall be taken from columns or shear walls in compliance with the conditions determined in TS – 10465. The experiment shall be made and existing concrete compressive strength is obtained by using standard deviation and average formulae. Reinforcing steel class shall be determined by visual inspection performed on the surfaces peeled. Characteristic yield strength of the steel in this class shall be taken as existing steel yield strength. In this investigation, the members in whose reinforcement steel corrosion is observed shall be marked on the plan and this situation shall be taken into consideration in determining the capacities of the members.

3.2.3. Medium Information Level in Reinforced Concrete Buildings

Building Geometry: In case structural working drawings of the building exist, the compliance of the existing geometry with the architectural plan and the structural working drawings is checked through measurements to be performed in the building. In case structural working drawings do not exist, the structural system and plan of the building is prepared by site survey. The information obtained shall include the location, span, height and dimensions of all reinforced concrete members. The building geometry information must include the details necessary for the accurate definition of the mass of the building. Short columns in the building are marked on the floor plan. The relation of the building with neighbor buildings (separate, adjacent, joint exists / does not exist) is determined. Foundation system is observed through a control pit to be opened at corner of the building.

Member Details: In case structural working drawings or shop drawings do not exist, the requirements given in 3.2.2 is applied, but amount of the columns and beams to be reinforcement control will be carried out shall not be less than 20 percent of the columns and 10 percent of the beams, and two each in every story at least. In case structural working drawings or shop drawings exist, the operations indicated in 3.2.2 for reinforcement control shall be applied on the reinforced concrete member in the same amount. In case of incompliance between the project and application, the “reinforcement accuracy coefficient” indicating the ratio of the existing reinforcement to the reinforcement in the project shall be determined for beams and columns separately. This coefficient, which is used in the determination of the members’ capacities, may not be greater than one. Reinforcement ratios shall be determined by applying this coefficient to all other members whose reinforcement determination was not performed.

Material Properties: In each story, at least three concrete samples and minimum nine samples in the building as one for each 400 m2 shall be taken from columns or shear walls in compliance with the conditions determined in TS – 10465 and the experiment shall be made. Values (average – standard deviation) obtained from the samples shall be taken as existing concrete compressive strength in calculation of the capacities of members. The distribution of the concrete compressive strength in the building shall be controlled with concrete hammer readings arranged with core sample experiment results or similar non-destructing examination tools calibrated with concrete sample test results. Reinforcement steel class shall be determined with visual inspection carried out on the surfaces that are peeled off. The design strength of the reinforcement of this class shall be taken as basis in the member capacity calculations. The members, on the reinforcements of which corrosion is observed, shall be marked on the plan and this condition shall be taken into consideration by an appropriate method in determining the capacities of the members.

3.2.4. Detailed Information Level in Reinforced Concrete Buildings

Building Geometry: Structural working drawings of the building exist. The compliance of the existing geometry with projects shall be determined through the measurements to be performed in the building. If there are important differences between projects and measurements, the projects are ignored and the building is investigated in compliance with average information level. Short columns and similar irregularities are marked on the floor plan. Relation with neighbor buildings (separate, adjacent, joint exists / does not exist) is determined. Building geometry information must include the details necessary for the accurate definition of the building mass. Foundation system is observed through a control pit to be opened at corner of the building.

Member Details: Structural working drawings of the building exist. In order to control the compliance of the reinforcement with the project, the operations indicated in 3.2.2 are applied on the reinforced concrete members in the same amount. In case of incompliance between the project and the application, the “reinforcement accuracy coefficient” giving the ratio of the existing reinforcement to the reinforcement in the project is determined for beams and columns separately. This coefficient, which is used in the determination of the members’ capacities, may not be greater than one. Reinforcement ratios shall be determined by applying this coefficient to all other members whose reinforcement determination was not performed.

Material Properties: In each story, at least three concrete samples and minimum nine samples in the building as one for each 200 m2 shall be taken from columns or shear walls in compliance with the conditions determined in TS – 10465 and the experiment shall be made. Values (average – standard deviation) obtained from the samples shall be taken as existing concrete compressive strength in calculation of the capacities of members. The distribution of the concrete compressive strength in the building shall be controlled with concrete hammer readings arranged with core sample experiment results or similar non-destructing examination tools related with concrete sample test results. Reinforcement steel class shall be determined with visual inspection in the surfaces peeled off. Experiments shall be made for the steels in each class (S220, S420, etc.) by getting one sample of each; the yield and rupture strengths of the reinforcement shall be determined and their compliance with the project will be checked. If compliant, the yield strength of the reinforcement used in the project shall be taken as basis in determining the capacities of the members. In case of incompliance, at least three more samples shall be taken and tested and the minimum yield strength obtained shall be taken as basis in determining the capacities of the members. The members, on the reinforcements of which corrosion is observed, shall be marked on the plan and this condition shall be taken into consideration in determining the capacities of the members.

The information level factors, that is applied to the capacities of the structural members according to information levels after the investigation of buildings can be seen in Table 3.1

Table 3.1. Information level factors [13]

Information Level	Information Level Factors
Limited	0.75
Medium	0.90
Detailed	1.00

3.3. Member Performance Regions

Structural members whose member damage levels are less than MN limit shall be considered in *Minimum Damage Region* (Minimum Hasar Bölgesi), between MN and GV limits shall be considered in *Visible Damage Region* (Belirgin Hasar Bölgesi), between GV and GÇ limits shall be considered in *Significant Damage Region* (İleri Hasar Bölgesi). Structural members of whose member damage levels are greater than GÇ limit are defined as in Collapse Region (Göçme Bölgesi). These regions are illustrated in Figure 3.1.

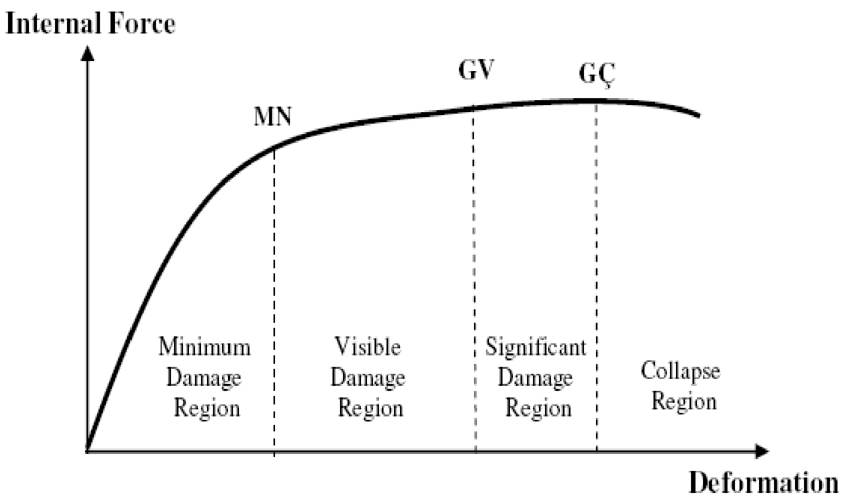

Figure 3.1. Member damage levels and member performance regions on capacity curve [13]

3.4. Building Performance Levels

Seismic safety of the buildings is related to the damage level possibly to occur in the structure under effect of the seismic load applied. Four building performance levels are defined.

3.4.1. Immediate Occupancy Level (HK)

At any story, as a result of the calculations performed at each seismic load direction, maximum 10 percent of the beams can be in visible damage region, but all other structural members shall be in minimum damage region. In this case, the building can be considered as in *Immediate Occupancy Level* and it is not required to be retrofitted.

3.4.2. Life Safety Level (CG)

At any floor, as a result of the calculations performed at each seismic load direction, maximum 30 percent of the beams and some of the columns can be in significant damage region. But, contribution of the columns in significant damage region must be less than 20 percent of the total shear force carried by columns. All other structural members shall be in minimum damage region or visible damage region. In this case, the building can be considered as in *Life Safety Level*. To comply with life safety level, the ratio of the shear force carried by the columns, exceeding the minimum damage limit in both upper and lower end sections at any story, to the shear force carried by all columns at the related story ratio must be less than 30%. The ratio of total shear force of the vertical components in significant damage region at roof story to total shear force of the columns at the related story ratio can not be more than 40%. Decision on rehabilitation should be made based on the number and distribution of members that exceed the safety limit. Structural members which have brittle damages *must* be retrofitted to ensure this performance level.

3.4.3. Collapse Prevention Level (GÖ)

At any floor, as a result of the calculations performed at each seismic load direction, maximum 20 percent of the beams and some of the columns can be in collapse region. But, contribution of the columns in collapse region must be less than 20 percent of the total shear force carried by columns. Columns in the collapse region at any story shall not create a collapse mechanism at that story. All other structural members shall be in minimum damage region, visible damage region or significant damage region. In this case, the building can be considered as in *Collapse Prevention Level*. To comply with collapse prevention level, the ratio of the shear force carried by the columns whose minimum damage limits are exceeded in both upper and lower end sections at any story to the shear force carried by all columns at the related story ratio must be less than 30%. The structural members which have brittle damages are accepted as they are in the *Collapse Region* for their member performance levels.

3.4.4. Collapse Level (GÇ)

If the building does not provide the conditions of collapse prevention level, it can be considered as in Collapse Level. The usage of the building in existing condition is not permitted.

3.5. Target Performance Levels for Buildings

Three types of ground shaking are defined to be taken into consideration in performance based design and evaluation. These ground shakings are explained by having probabilities to be exceeded in 50 years.

- *Service (Usage) Ground Shaking*: It is defined as ground shaking having a 50% probability to be exceeded in 50 years. Return period of this ground shaking is approximately 72 years. The effect of this ground shaking (spectral acceleration) is half of the effect of ground shaking defined below.
- *Design Ground Shaking:* It is defined as ground shaking having a 10% probability to be exceeded in 50 years. Return period of this ground shaking is approximately 475 years. This ground shaking is used in the Turkish Earthquake Codes 1998 and 2007.
- *The Biggest Ground Shaking*: It is defined as ground shaking having a 2% probability to be exceeded in 50 years. Return period of this ground shaking is approximately 2475 years. The effect of this ground shaking is 1.5 times of the effect of design ground shaking.

Table 3.2. Target performance levels for buildings under different earthquake effects [13]

Purpose of Occupancy and Type of Building	Probability to be exceeded		
	50% in 50 years	**10% in 50 years**	**2% in 50 years**
The buildings to be utilized immediately after the earthquake: Hospitals, health facilities, fire fighting buildings, communication and power facilities, transportation stations, governorate, county and municipality administration buildings, first aid and emergency planning stations	_	**HK**	**CG**
Intensively and long-term occupied Buildings: Schools, dormitories, boarding houses, military barracks, prisons, museums, etc.	_	**HK**	**CG**
Intensively and short-term occupied Buildings: Cinema, theatre, concert saloons, cultural centers, sport facilities.	**HK**	**CG**	_
Buildings containing hazardous materials: Buildings containing or storing toxic, explosive and flammable materials, etc.	_	**HK**	**GÖ**
Other buildings: Buildings after than above defined buildings (Residential and office buildings, hotels, touristic facilities, industrial structures, etc.)	_	**CG**	_

3.6. General Principles about the Performance Based Design

The aim of the earthquake calculations is to evaluate performance levels of the existing or retrofitted buildings. Linear analysis methods or nonlinear analysis methods both can be used to evaluate the seismic performances of the structural systems but these two theoretically different methods may not give the identical results. The following principles are acceptable for both of these methods.

- In the definition of the earthquake effects, elastic (non reduced) acceleration spectrum will be used but for the different exceedance probabilities it will be modified according to the *target performance levels* for buildings. The building importance factor will not be put into practice.
- The seismic performance of the building will be considered under the combined effects of the vertical loads and earthquake effects. The vertical dead loads will be

described compatible with the masses which are taken into consideration under the seismic calculations.

- Lateral seismic loads will be applied to the building separately in two sides and in two directions.
- The structural system model has to be prepared accurately to calculate the internal forces, displacements and deformations that will be occurred in the structural members under the vertical loads and seismic effects.
- The buildings in which the slabs are considered as rigid diaphragms laterally; two lateral displacement degrees and one rotational degree of freedom will be taken into account. The storey's degrees of freedom will be defined at the center of mass of the storey and no additional eccentricity effects will take place.
- The uncertainties about the structural systems of the existing buildings will be reflected according to the *information level factors* to the evaluation methods.
- The description of the yield surfaces of reinforced concrete sections under uniaxial or biaxial bending and axial load, will be described according to the following statements.
- The existing material properties modified with the information level factors will be regarded for the material properties of concrete and reinforcement.
- The maximum strain capacity of the concrete will be taken as 0.003 and for the steel reinforcement it will be considered as 0.01.
- The yield surface can be modeled as yield lines and yield planes for two dimensional and three dimensional behavior conditions respectively.
- For the section definitions of reinforced concrete structural members, the connection regions will be regarded as infinitely rigid end zones.
- *Section stiffness for cracked sections* $(EI)_e$, shall be used for linear behavior of reinforced concrete members under the effect of bending before yielding. Till more certain calculation is performed, following values shall be used for section stiffness corresponding to cracked sections:

For beams: $\left(EI\right)_e = 0.40\left(EI\right)_0$ (3.1)

For columns and shear walls:
$$\frac{N_D}{(A_c.f_{cm})} \leq 0.10 \;;\; \left(EI\right)_e = 0.40\left(EI\right)_0$$
$$\frac{N_D}{(A_c.f_{cm})} \geq 0.40 \;;\; \left(EI\right)_e = 0.80\left(EI\right)_0 \tag{3.2}$$

For the intermediate values of the axial compression load N_D linear interpolation will be assessed. The axial compression load, N_D, will be defined according to the pre-vertical load calculations in which uncracked section stiffness $\left(EI\right)_0$ and the vertical loads compatible with the masses used in seismic calculations, are used.

- In the calculations of the positive and negative plastic moments of reinforced concrete T-beams, the concrete slab and the reinforcement in it will be considered.
- When the anchorage and lap lengths are not sufficient for the reinforced concrete members, the yield strength of the related reinforcement will be reduced according to the lap length insufficiency ratio during the calculation of the sectional moment capacity.

3.7. Nonlinear Analysis Methods

3.7.1. Definition of Nonlinear Analysis Method

The aim of the non–linear analysis methods to be used in determination of structural performances and retrofitting analysis of existing buildings under the effect of the seismic loads, is calculating the plastic rotation demands of ductile behavior and the demand for internal forces of brittle behavior for a given earthquake. Then, these demand values are compared with deformation capacities defined in this section. Evaluation of the structural performance is done for the performance level of the member and the building.

Some of the main nonlinear analysis methods given in the Turkish Earthquake Code 2007 are *Incremental Equivalent Seismic Load Method, Incremental Mode Superposition Method* and *Analysis Methods in Time Domain*.

3.7.2. Incremental Equivalent Seismic Load Method (Pushover Analysis Method)

This method shall be applied to the buildings whose total story number is not more than 8 and torsional irregularity coefficient calculated without considering additional eccentricities is η_{bi} < 1.4. Besides, in the direction of the considered earthquake, it is necessary that the ratio of effective mass corresponding to the first natural vibration mode calculated on bases of linear behavior to total building mass (except the mass of the basement floor encircled by shear walls) must be minimum 0.70. In incremental equivalent seismic load method, nonlinear pushover analysis is performed under monotonically increasing equivalent earthquake load until performance point is reached. Performance point is also named as target modal displacement demand. Displacement, plastic deformation, increase in internal forces and related cumulative values are determined at each pushover step. Once the system reaches its performance point, total base reaction and roof displacement values are determined.

3.7.3. Incremental Mode Superposition Method

Modal capacity diagrams and modal displacement demands corresponding to all modes considered shall be obtained. On this base; internal force demands, plastic deformations (plastic rotations) and displacements occur on the structural systems shall be calculated.

3.7.4. Analysis Methods in Time Domain

Analysis Method in Time Domain is step by step integration of the movement equation of the system by considering non–linear behavior of the structural system. The displacement, deformation and internal forces occur in the system in the duration of the analysis in each time increase and the maximum equivalent values of them with respect to the seismic demand are calculated.

3.8. Methodology of Pushover Analysis Method

The steps of the methodology of the incremental pushover analysis will be defined for the nonlinear analysis method in this section.

- The plastic behavior of the structural system model will be idealized according to the principles in 3.9.
- Before the pushover analysis method, a nonlinear static analysis will be done considering the vertical loads which are compatible with the story masses. The results of this static analysis will be the preliminary conditions for the pushover analysis method.
- For the incremental equivalent seismic load method, the *modal capacity diagram* will be obtained which has the axes as the *modal displacement-modal acceleration* of the first (dominant) mode. Modal capacity diagram obtained at the end of pushover analysis and elastic response spectrum are taken into consideration together and modal displacement demand of first mode will be calculated. At the last step, displacements which refer the modal displacement demands, plastic deformations (plastic rotations) and internal force demands will be evaluated.
- From the plastic rotational demands which are calculated for the ductile sections, the plastic curvature demands will be evaluated which will handle to find the total plastic curvature demand of the member. After that, in accord with these the strain demands for the concrete and reinforcement steel will be achieved for reinforced concrete members. These strain demands will be compared with the strain limits which are specified for different damage levels so a performance level evaluation will be done in sectional for structural members in ductile manner. Also the obtained shear force demands will be compared with the shear capacity of sections to make a consideration in brittle manner.

3.9. Idealization of Plastic Behavior

In this specification, it is suggested to use "*elastic perfectly plastic hypothesis*" for nonlinear analysis. It is assumed that plastic deformations occur uniformly distributed within the plastic hinge length. In case of simple bending, length of the plastic deformation region called *plastic hinge length* (L_p) shall be taken as equal to half of member dimension in bending direction (h).

$$L_p = 0.5 \times h \tag{3.3}$$

It is required that plastic hinges are located in the exact middle of the plastic deformation region theoretically. But in practical operations, following approximate idealizations can be allowed:

- Plastic hinges shall be located at sufficient distance from the column-beam connection region. But, it must be considered that plastic hinges can occur at spans of the beams due to vertical loads.
- In reinforced concrete shear walls, plastic hinges are allowed to be assigned in bottom ends of shear walls in each story. U, T, L or box typed shear walls, must be idealized as single shear wall sections. In the case of basement floors of the buildings are encircled by rigid shear walls, plastic hinges of these shear walls going towards the upper floors must be located by starting on basement.
- Yield surfaces of the reinforced concrete members can be modeled as yield lines and yield planes for two dimensional and three dimensional behavior conditions respectively.

The following idealizations shall be considered about internal force-plastic deformation equations to be used in pushover analysis model.

- Strain hardening, in internal force-plastic deformation equations, can be ignored approximately (Figure 3.2(a)). For pushover steps after the plastification has occurred in the sections under the effect of simple or combined bending and axial load; regarding the internal forces remain over yield surface, the vector of plastic deformation must be perpendicular to the yield surface.

- In case of strain hardening is considered (Figure 3.2(b)), for pushover steps after the strain hardening has occur in the sections under the effect of simple or combined bending and axial load; the conditions that internal forces and vector of plastic deformation must be satisfied, will be defined according to a suitable strain hardening model obtained from literature.

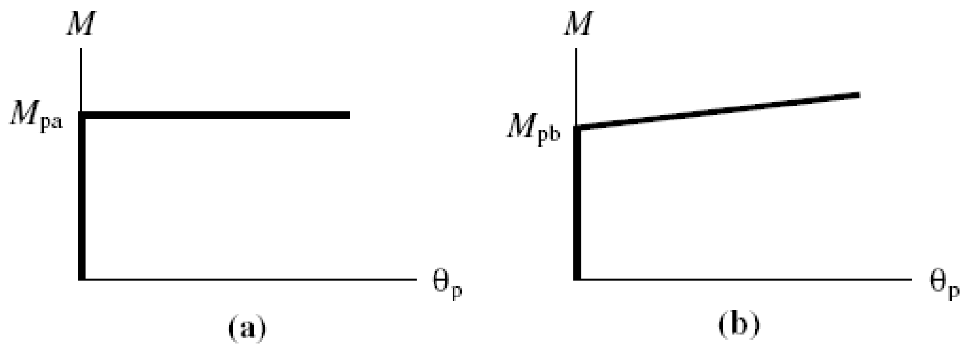

Figure 3.2. Bending moment-plastic hinge rotation relations [13]

3.10. Pushover Analysis Using Incremental Equivalent Seismic Load Method

It is required that; the effective mass calculated by considering first natural vibration mode of considered earthquake direction to total building mass shall not be less than 0.70 and torsional irregularity coefficient calculated without considering additional eccentricities is $\eta_{bi} < 1.4$. In addition, number of stories shall not be more than eight excluding basement. Otherwise Incremental Equivalent Seismic Load Method can not be applied to the structural system.

In Incremental Equivalent Seismic Load Method, performance point of building is represented with base shear-roof displacement curve and modal capacity diagram. Roof displacement is the displacement calculated in each pushover step in x earthquake direction considered at center of mass of the top storey. Base shear force is the sum of equivalent earthquake loads in each step in x earthquake direction. Structural system is calculated under vertical loads and proportionally increasing earthquake loads to obtain pushover curve until the performance point is reached.

Modal transformation applied to pushover curve and modal capacity diagram having coordinates “modal displacement – modal acceleration” shall be obtained as expressed below:

a) modal acceleration, $a_1^{(i)}$ corresponding to first mode (in considered earthquake direction) in (i)th pushover step is obtained as following way :

$$a_1^{(i)} = \frac{V_{x1}^{(i)}}{M_{x1}} \tag{3.4}$$

b) Modal displacement, $d_1^{(i)}$, corresponding to first (dominant) mode for considered earthquake direction in (i)th pushover step is obtained as following way.

$$d_1^{(i)} = \frac{u_{xN1}^{(i)}}{\Phi_{xN1}.\Gamma_{x1}} \tag{3.5}$$

c) Modal contribution coefficient corresponding to first mode in x direction Γ_{x1} is defined with the following equation;

$$\Gamma_{x1} = \frac{L_{x1}}{M_1} \tag{3.6}$$

Modal capacity diagram obtained at the end of pushover analysis and elastic response spectrum are taken into consideration together and modal displacement demand of first mode is calculated. Modal displacement demand, $d_1^{(p)}$, is equal to nonlinear spectral displacement S_{di1}.

$$d_1^{(p)} = S_{di1} \tag{3.7}$$

Nonlinear spectral displacement, S_{di1}, is obtained by using linear elastic spectral displacement, S_{de1} .According to “Equal Displacement Rule”, the maximum inelastic lateral displacement experienced by the structure behaving nonlinearly will be equal to the maximum lateral displacement experienced by the structure behaving linear elastically.

$$S_{di1} = C_{R1}.S_{de1} \tag{3.8}$$

Linear elastic spectral displacement, S_{de1}, is obtained by using linear elastic spectral acceleration, S_{ae1}, that is corresponding to the first mode of pushover analysis.

$$S_{de1} = \frac{S_{ae1}}{(w_1^{(1)})^2} \tag{3.9}$$

Spectral displacement ratio, C_{R1}, in case of initial period $T_1^{(1)}$ is equal to or greater than T_B that is the characteristic period at acceleration spectrum. ($T_1^{(1)} \geq T_B$ or $(w_1^{(1)})^2 \leq w_B{}^2$).

$$C_{R1} = 1 \tag{3.10}$$

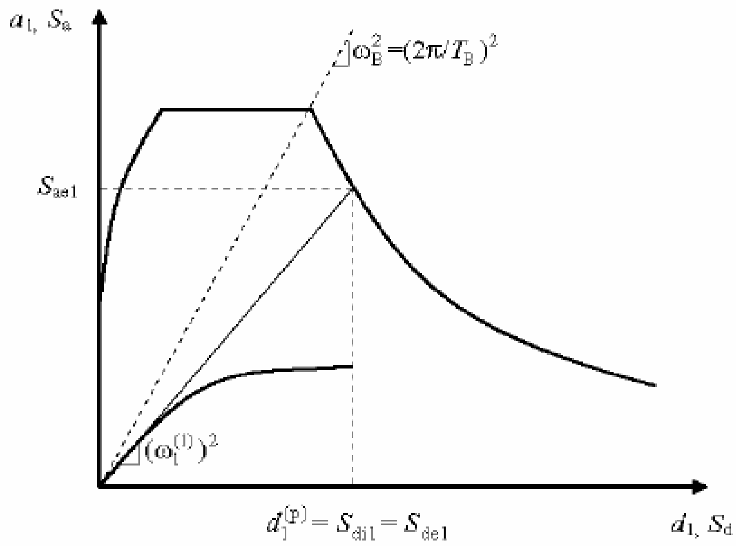

Figure 3.3. Determination of performance point $T_1^{(1)} \geq T_B$ [13]

Spectral displacement ratio, C_{R1}, in case of initial period, $T_1^{(1)}$, is less than, T_B, that is the characteristic period at acceleration spectrum ($T_1^{(1)} < T_B$ or $(w_1^{(1)})^2 > w_B{}^2$) is calculated by successive approximation method in following way.

i. Modal capacity diagram obtained at the end of pushover analysis is converted to a bi-linear diagram. In this diagram, the slope of the beginning line is taken as equivalent to value, $(w_1^{(1)})^2$, corresponding to the first mode the angle of line in first step $(i=1)$ of pushover analysis ($T_1^{(1)} = \dfrac{2\pi}{w_1^{(1)}}$) illustrated in Figure 3.4.

ii. In the first step of successive approximation method it is assumed that C_{R1}=1 and coordinates of equivalent yield point is determined by using equivalent areas rule.

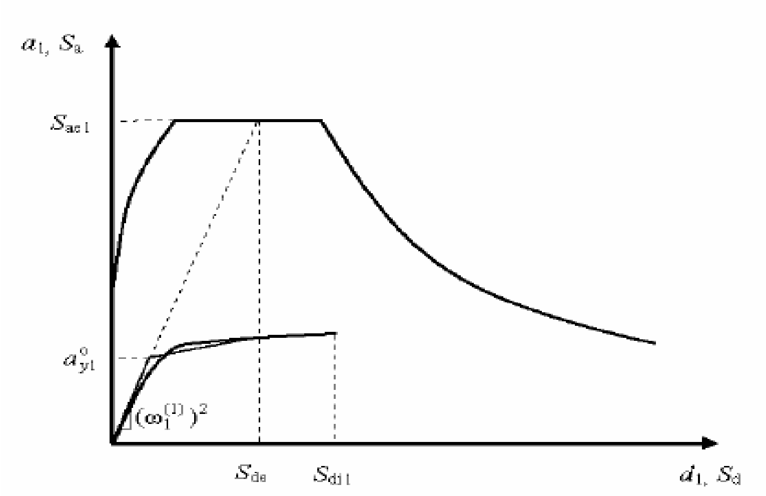

Figure 3.4. Determination of performance point $T_1^{(1)} < T_B$ [13]

Later nonlinear spectral displacement, S_{di1}, is calculated by using Equation (3.9) and C_{R1} , value is calculated by using Equation (3.11).

$$C_{R1} = \frac{1+(R_{y1}-1)T_B / T_1^{(1)}}{R_{y1}} \geq 1 \tag{3.11}$$

Strength reduction factor for first mode, R_{y1}, can be calculated by using Equation (3.12)

$$R_{y1} = \frac{S_{ae1}}{a_{y1}} \tag{3.12}$$

iii. Coordinates of equivalent yield point is determined again by using equivalent areas rule. a_{y1}, R_{y1} and C_{R1} are calculated again. Successive approximation method is completed when the results of two adjacent steps are approximately same, Figure 3.5.

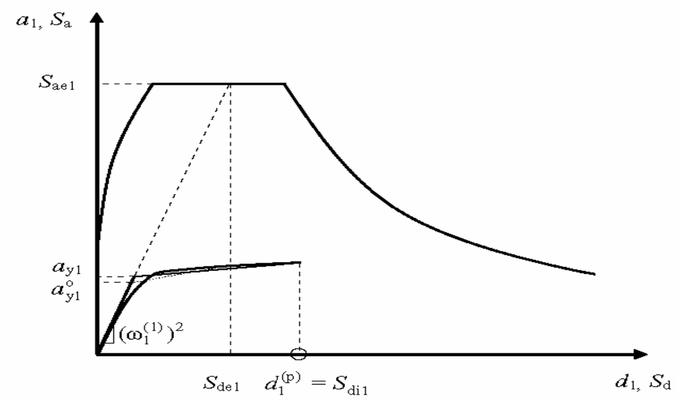

Figure 3.5. Determination of performance point $T_1^{(1)} < T_B$ [13]

Modal displacement demand obtained by using Equation (3.8) is used in Equation (3.13) and roof displacement along the x earthquake direction is calculated.

$$u_{xN1}{}^{(p)} = \Phi_{xN1}.\Gamma_{x1}.d_1{}^{(p)} \tag{3.13}$$

3.11. Determination of Strain Demands

Plastic curvature demand in structural systems which are designed according to nonlinear methods is calculated, depending on plastic rotation demand, θ_p, obtained at any section, by using the Equation (3.14)

$$\phi_p = \frac{\theta_p}{L_p} \tag{3.14}$$

Bilinear bending moment-curvature relation obtained by using concrete and reinforcing steel models defines equivalent yield curvature, ϕ_y. Total curvature demand is obtained by adding equivalent yield curvature to plastic curvature demand, ϕ_p.

$$\phi_t = \phi_y + \phi_p \tag{3.15}$$

Stress, strain demands of concrete and steel are calculated according to moment-curvature analysis defined above. Seismic demands obtained based on strain demands of concrete and steel are compared with the strain limits given below and member damage levels are determined.

3.12. Member Damage Strain Capacities of Reinforced Concrete Members

In ductile reinforced concrete structural system members, strain limits depending on their damage levels are defined below.

a. For *Minimum Damage Limit* (MN), the maximum compressive strain demand on concrete (at the extreme fiber) and the unit deformation limit for the reinforcement corresponding to that total curvature demand are given in Equation (3.16).

$$(\varepsilon_{cu})_{MN} = 0.0035 \qquad (\varepsilon_s)_{MN} = 0.010 \tag{3.16}$$

b. For *Visible Damage Limit* (GV), strain capacities of at the extreme layer of concrete (the extreme fiber at the section bounded by stirrups) and reinforcement are given in Equation (3.17).

$$(\varepsilon_{cg})_{GV} = 0.0035 + 0.01(\rho_s / \rho_{sm}) \leq 0.0135 \qquad (\varepsilon_s)_{GV} = 0.040 \tag{3.17}$$

c. For Collapse Limit (GÇ), strain capacities of at the extreme layer of concrete (the extreme fiber at the section bounded by stirrups) and reinforcement are given in Equation (3.18).

$$(\varepsilon_{cg})_{GC} = 0.004 + 0.014(\rho_s / \rho_{sm}) \leq 0.018 \quad (\varepsilon_s)_{GC} = 0.060 \tag{3.18}$$

ρ_s : Volumetric ratio of existing transverse reinforcement steel

ρ_{sm} : Volumetric ratio of transverse reinforcement that is required for design of a new building.

3.13. Shear Strength Capacities of Reinforced Concrete Members

i. The shear strength capacities of the reinforced concrete members except the beam-column joints will be determined in proportion to TS-500. For the determination of shear strength capacities, the existing material properties described with the building information levels will be used. The structural members having shear resistance smaller than the shear demand will be defined as brittle members.

ii. Shear force in beam-column joints along the earthquake direction considered (Fig.3.6) shall be calculated by Equation (3.19). V_{kol}, in the equation means the smaller of the shear forces at above and below the joint calculated.

$$V_e = 1.25 f_{ym} (A_{s1} + A_{s2}) - V_{kol} \qquad (3.19)$$

The shear force calculated by Equation (3.19) in a joint along the given earthquake direction shall not exceed the limits given below by Equation (3.20) and Equation (3.21), (Figure 3.6) .In the cases where those limits are exceeded, cross-section dimensions of column and/or beam shall be increased and the seismic analysis shall be repeated. The existing compression strength for concrete will be used in the calculations.

$$\text{In confined joints; } V_e = 0.60.b_j.h.f_{cm} \qquad (3.20)$$

$$\text{In unconfined joints; } V_e = 0.45.b_j.h.f_{cm} \qquad (3.21)$$

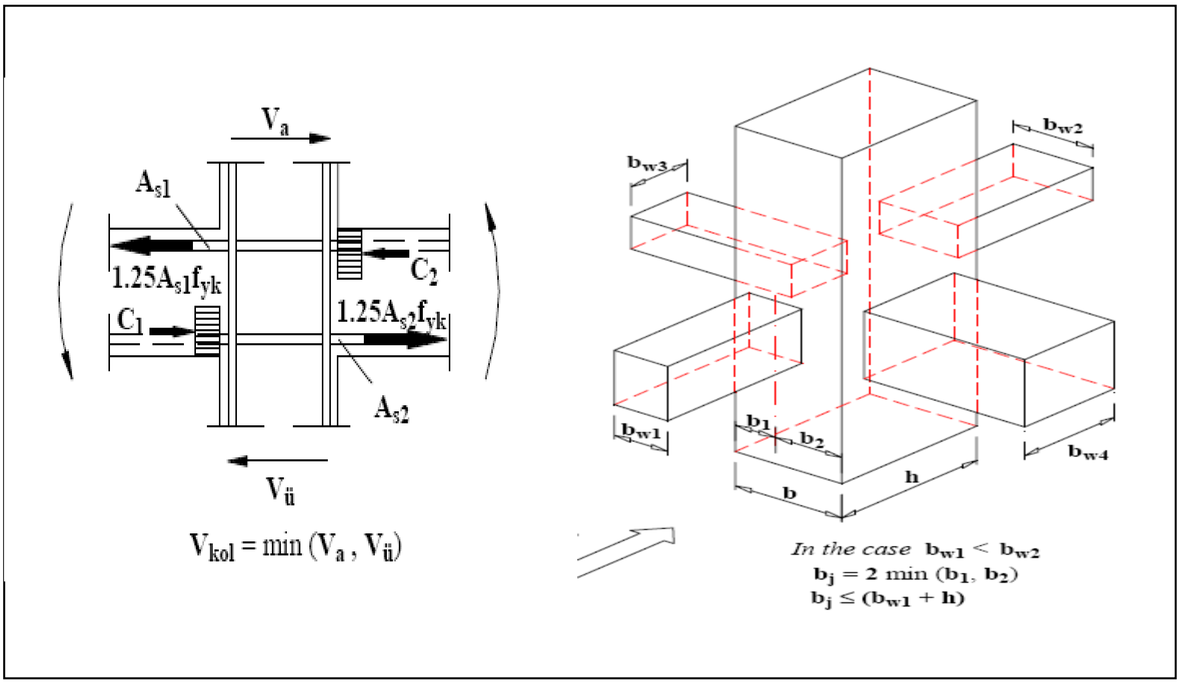

Figure 3.6. Confined joint conditions to define the shear capacity [13]

4. RETROFIT STRATEGIES AND SYSTEMS

4.1. General

For most buildings and performance objectives, a number of alternative strategies and systems may result in acceptable design solutions. . Before adopting a particular strategy, the engineer must evaluate a number of solutions for feasibility and applicability and together with the owner, should choose a strategy or a combination of strategies that seems like the most favorable solution. It is not possible to start retrofit study until the owner's performance objectives for the construction have been identified and an assessment of the building was completed. This assessment will determine if the building is capable, in its existing configuration, to provide this performance, and if not, the extent of any existing deficiencies.

The methodology indicated for evaluating different strategies to determine their applicability is also a bit complex compared to traditional code-based approaches assessing structural interventions. Often, the effort involved in evaluating several strategies to upgrade an optimal selection, may be very long and expensive. For simple structures, the deficiencies that can be easily and economically reduced with the introduction of the strength and rigidity to the lateral force resisting system, detailed evaluation of several alternative strategies is neither justified nor necessary [14].

4.2. Retrofit Strategies

Improving the probable seismic performance of the structural system or reducing the existing risk to an acceptable level can be the basic goals of the retrofit strategy. Seismic risk reduction can be achieved by both technical strategies and management strategies. Increasing building strength, correcting critical deficiencies, altering stiffness, and reducing demand are the general approaches of technical strategies. For the approaches of management strategies; change of occupancy, incremental improvement and phased construction can be mentioned.

4.3. Retrofit System

A retrofit system is the method used to achieve the selected strategy. For example, if the strategy is to increase the strength of the building, then to accomplish these strategy alternative systems that may be used as addition of new shear walls, thickening of existing shear walls, and addition of braced frames. Deflection compatibility with the existing structure is the most important consideration in the selection and design of a retrofit system. An example strategy to upgrade a building that is both weak and brittle, although being quite stiff, is to provide supplemental lateral strength.

Seismic risk reduction strategies include such approaches as increasing strength, increasing stiffness, increasing deformability, increasing damping, reducing occupancy exposure and modifying the character of the ground motion transmitted to the building. Strategies can also include combinations of these approaches. Retrofit systems are specific methods used to implement the strategy such as; the addition of shear walls or braced frames to increase stiffness and strength, the use of confinement jackets to enhance deformability [14].

4.4. Technical Strategies

Building's mass, stiffness, damping and configuration; and the characteristic of the ground motion that the building must resist are the leading aspects that affect the building's seismic performance. Technical strategies which can be divided into groups as system strengthening and stiffening, reducing earthquake demands and enhancing deformation capacity; improve seismic performance by directly performing these strategies.

4.4.1. System Strengthening and Stiffening

The most common strategies to improve the seismic performance of the buildings without the required lateral load resistance, are system strengthening and stiffening which are closely related but different. The effect of strengthening a structure is to increase the amount of total lateral force capacity required to initiate damage events within the structure. The effect of stiffening is directly related with the structure's displacement. The structure can achieve larger lateral displacements without damage, if strengthening done without stiffening.

Figure 4.1 uses a demand/capacity spectrum diagram which shows the effect of system strengthening on earthquake performance. In that figure the curve A-B-C-D represents the performance of an unstrengthened frame while the curve A-B-E-F-G-H represents the one possibility of a strengthened frame capacity curve. This figure shows the capacity curve under pure strengthening effect without any stiffening increase of the original structure so the capacity curve for the existing and strengthened structure reaches the target displacements at the same point.

Figure 4.2 shows the stiffening effects with the capacity curve A-B-C-D-E for the unstiffened structure and the improved capacity curve A-F-G-H-I of the structure after stiffening effects. System stiffening and strengthening are performed as concurrent strategies because the strengthening of the structure results in stiffening of the structure and the systems stiffen the structure also strength the structure.

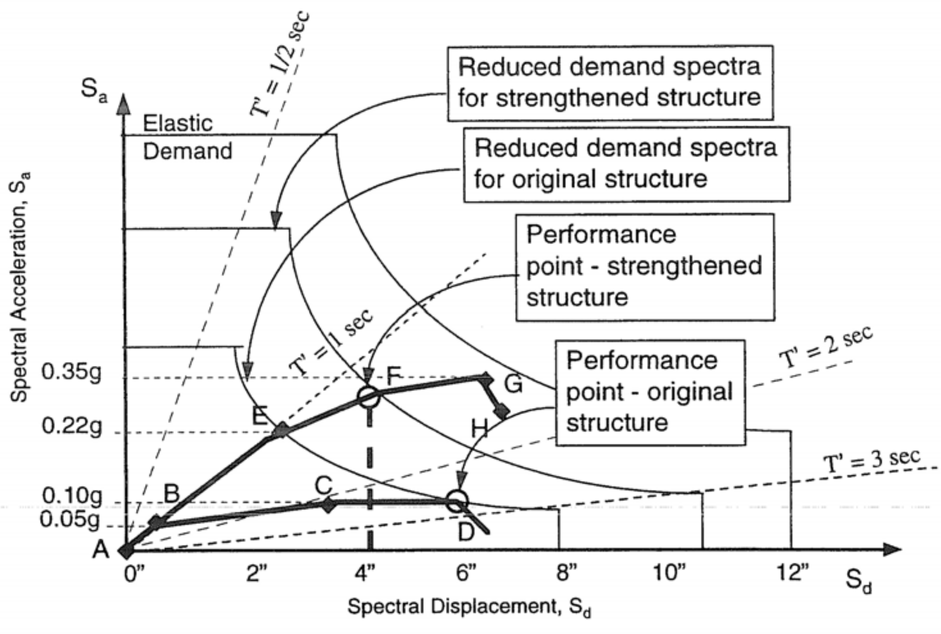

Figure 4.1. Effect of system strengthening on performance [14]

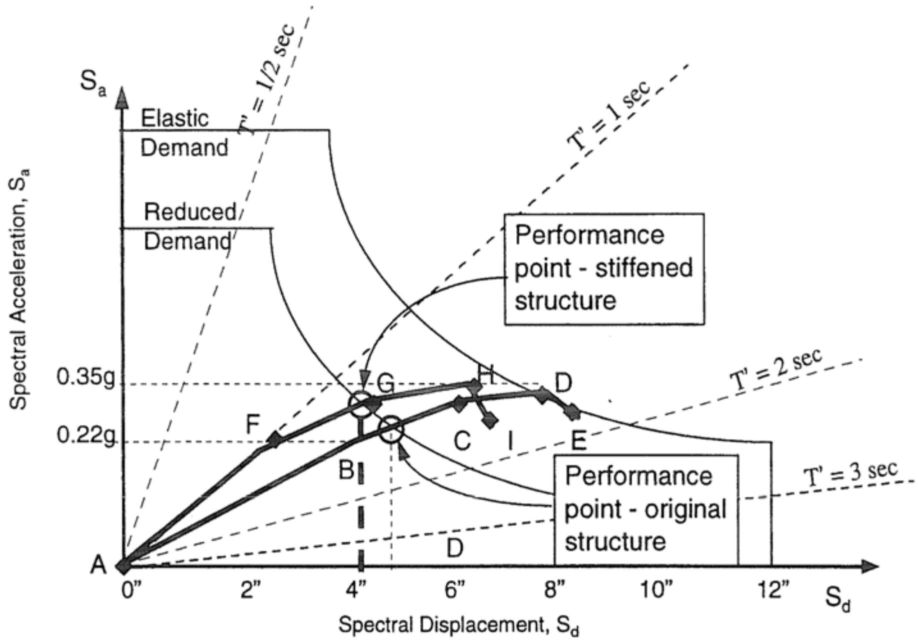

Figure 4.2. Effect of system stiffening on performance [14]

Addition of shear walls, braced frames, buttresses and moment resisting frames are the alternative systems which can be employed to achieve structural strengthening and stiffening [14].

<u>4.4.1.1. Addition of Shear Walls</u> Addition of shear walls into an existing concrete structure is one of the most common ways to upgrade the seismic performance. Shear wall addition is an effective and economic way to increase the system stiffening and strengthening and also it is readily compatible with most existing concrete structures. It changes the dynamic behavior of the whole structure during an earthquake and also causes considerable redistribution of the lateral forces between the earthquake resisting elements. And the following conditions should be considered:

- Avoiding large concentration of forces in members with small strength and/or ductility capacities by locating the shear walls uniformly throughout the structure.
- Improving the distribution of lateral force by reducing the effects of torsion and irregularities.
- Providing adequate strength in connections between the existing structure and the shear walls.

- Improving the foundations to bear the large overturning moments that the shear walls produce at their bases.
- Resulting increase in seismic forces because of the addition of shear walls results in a significant increase in building mass.

Also weak shear walls can be strengthened to improve the seismic performance because of the fact that they carry significant percent of the total overturning moments and total base shear forces. These weak shear walls generally strengthened with increasing their wall sizes. And to improve the flexural strength of the shear wall, reinforced concrete flanges are added to both ends of the wall [15].

<u>4.4.1.2. Addition of Braced Frames</u> Steel bracing of RC frames has recently been shown to be a suitable alternative to shear walls for enhancing the seismic performance of an existing building. Compared with shear walls, steel braces provide lower levels of stiffness and strength to the structure but they add far less mass to the structure than shear walls. Another advantage of steel braces is the construction with less disruption of the building which results in a less of light and smaller effect on of traffic pattern within the structure. Because relatively large forces are transferred between the existing structure and steel braces, it is the most difficult issue to attach the steel braces to the existing building.

Different bracing methods fall into two main categories, namely external bracing; and internal bracing. In the external bracing method, existing buildings are retrofitted by attaching a local or global steel bracing system to the exterior (and occasionally interior) frames. Architectural concerns and difficulties encountered when connecting the steel bracing to the RC frames are the main shortcomings of this method. In the internal bracing method, the buildings are retrofitted by positioning a bracing system inside the individual bays of the RC frames. The bracing may be attached to the RC frame either indirectly or directly. In the indirect internal bracing, a braced steel frame is positioned inside the RC frame. As a result, the transfer of load between the steel bracing and the concrete frame is carried out indirectly through the steel frame. This method of internal bracing can be costly and technical difficulties in fixing the steel frame to the RC frame can be inhibiting. The direct internal bracing method overcomes the aforementioned shortcomings of the indirect internal bracing system. In this method the steel braces are directly connected to the RC frames without the use of an intermediary steel frame. The direct internal bracing method was proposed not only as a retrofit measure for existing buildings, but also as a shear-resisting element to be used in the seismic design of new buildings [16].

A braced RC frame designed using the same force reduction factor as that of a conventional RC moment frame with moderate ductility would behave adequately during an earthquake event. The design of RC sections in a braced RC frame can be carried out using conventional RC design methods. General reinforcement detailing requirements are adequate and there is no need to use special seismic detailing. The brace members and their connections can be designed using a similar procedure to that for braces in steel structures. The uniform method proposed to predict forces transferred by brace members to beams and columns is adequate. It prevents failure from occurring in connections of the brace members and minimizes the moment at the interface of the beam and column [17].

4.4.1.3. Addition of Buttresses Buttresses are the braced frames or shear walls installed perpendicular to the exterior face of the building to provide required seismic performance by strengthening or stiffening. The advantages of adding buttresses can be mentioned as; the cost of buttresses can be quite lower than the interior systems as shear walls and braced frames because the work is performed outside of the building and also it is appropriate to for the buildings which must be occupied during construction. But the buttresses are not convenient for the historical buildings because of the significant aesthetic impact of them.

4.4.2. Enhancing Deformation Capacity

Enhancing deformation capacity is the method to improve the capacities of individual structural members to resist deformations within the building. The capacity spectrums of the non ductile reinforced concrete buildings often do not intersect the demand spectrum of the earthquake ground motion because of the failure of individual elements. For this reason improvement the seismic performances of these elements can be adequate to achieve the desired performance level.

As it can be seen in Figure 4.3, the A-B-C-D-E capacity curve of the existing building can not intersect the demand curve at a performance point because of the failure of an individual critical element. The improvement of this element makes the A-B-C-D-F-G capacity curve of the building to extend larger spectral displacements that allows the performance point intersection at a desired structural performance level.

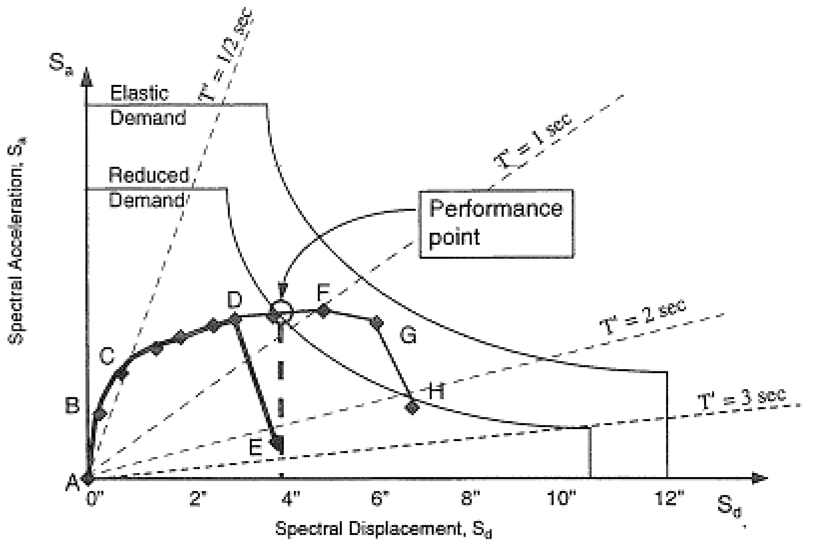

Figure 4.3. Effect of deformation enhancement on structural performance [14]

The strategy to enhance the deformation capacities of members include; adding confinement to existing members, strengthening columns and making local stiffness reductions [14].

<u>4.4.2.1. Adding Confinement</u> For the non ductile columns of the existing building, exterior confinement jacketing causes an increase in the deformation capacity. Shear strength and especially ductility is improved by confinement. The techniques that can be used for exterior confinement are steel plate jacketing, reinforced concrete jacketing and winding with fiber reinforced plastic fabrics.

The jacketing elements must also resist the compressive stresses exerted by the concrete elements, in a rigid manner. Because of the fact that, in steel profile jacketing the profiles placed around the column can not pass through the slab; the flexural column strength can not be improved to desired levels [14, 18].

4.4.2.2. Column Strengthening Column strengthening is performed when strong beam-weak column configuration occurs in older reinforced concrete frames. This condition tends to develop single story mechanism which means all the inelastic deformation demand of the ground motion occurs in the story that the mechanism takes place. When the columns are strengthened the beams become the weaker elements, which do not permit the story mechanism condition and leads more structural drifts overall.

The beam column joints must be strengthened to allow development of larger moments, in addition to column strengthening. Joints must be strengthened to respond the internal forces and joint forces and to provide adequate shear strength and sufficient beam reinforcement anchorage in joint area. Sometimes beam strengthening is needed and strengthening the beam with jacketing improves both flexural strength and shear strength by installing longitudinal and transverse reinforcement. The beam should not be too stiff than the column to insure that the potential hinging will occur in beam rather than in column.

4.4.3. Reducing Earthquake Demand

This strategy involves modification of the response of the structure by trying to reduce the impact of the earthquake itself rather than modifying the capacity of structure by strengthening and enhancing deformation capacity. Base isolation, energy dissipation systems and reducing the building's mass are the common methods to achieve reducing the demand of the seismic activity.

This strategy needs larger investment then the conventional approaches to modify the behavior of the structure. It can be convenient for buildings housing critical occupancies with sensitive equipment or need to attain rapid post earthquake functionality. This may also be appropriate for the historical structures because this strategy does not need extensive invasive construction in the historic spaces [14].

4.4.3.1. Base Isolation Among the structural control schemes developed, seismic base isolation is one of the most promising alternatives to enhance structural safety and integrity against severe earthquakes. It can be adopted for new structures as well as the retrofit of existing buildings and bridges. Strategies to achieve seismic isolation include:

1) Period shifting of structures
2) Cutting off load transmission path

The spring like isolation bearings with considerable lateral flexibility help in reducing the earthquake forces by changing the structure's fundamental period to avoid resonance with the predominant frequency content's of earthquakes. Whereas the sliding type isolation bearings filter out earthquake forces via the discontinuous sliding interfaces, between which the forces transmitted to the superstructure are limited by the maximum friction forces, regardless of earthquake intensity.

Ongoing research has improved the effectiveness of isolators in decreasing problems of stability, roll-out, failure of the isolators, or unexpected response and the trend away from add-on mechanical dampers. Additionally, the difficulties of manufacturing large isolators have diminished. It is now possible to make bearings as large as 1.5 m. in diameter.

<u>4.4.3.2. Energy Dissipation Systems</u> Energy dissipation systems improve the ability of the structure to dampen earthquake response in a benign manner, through both viscous and hysteretic damping. Energy absorbing elements within the bracing system is an energy dissipation system which may be used mainly for its damping effect on dynamic vibrations of the structure, and instead of swinging back and forth repetitively; the structure comes to a stop eliminating some of the effects of rapid stress reversal.

This system becomes most effective performing on buildings with large deformation capacity because in relatively rigid buildings the system can not dampen the building response before damage has occurred in the building [19].

<u>4.4.3.3. Mass Reduction</u> Building mass reduction results in a reduction of the seismic forces on the structural elements. It also reduces the building's natural period, the amount

of internal force that develops during building's response and the total displacement demand of the structure. Mass can be reduced by removing the upper stories of the building may be by changing the use of the building, removing a heavy roof system, removing heavy non structural elements such as cladding, water tanks and storage.

4.5. Management Strategies

Management strategies in which the decision of building owner is more important than the engineer, affect the feasibility and implementation of technical strategies. There are two approaches for the management strategies, it can affect the agreement on the building's probable performance and it can regulate the alternative implementations of technical strategies. Management strategies include some decisions about changing building occupancy, demolishing the building and replacing with an alternative facility, interior or exterior retrofit scheme, retrofit the building during occupancy or retrofit of vacant building.

4.6. Strategy Selection

The first necessity of strategy selection is establishing the basic performance objectives desired for the building and the existing deficiencies relative to those performance objectives. After the determination of these objectives, evaluation of various strategies comes to decide whether it meets the improvements of deficiencies in technical and practical manner. The factors for the selection of the retrofit strategy are seismic performance, project cost and schedule, aesthetics and architectural constraints code requirements and occupancy disruptions during retrofit work.

The most important design constraint is the project performance, for the engineers. Each of these constraints will apply in some degree to every project and their relative importance on a given project will not be the same, however, and it will vary from one project to another. For example, historical buildings may require retrofit system designed to preserve historical structural or non structural features in the most inconspicuous manner. In some cases, when permissible, removal and replication of these features during the retrofit work may be more cost effective than preservation or restoration.

Importance factors of these constraints are determined according to building conditions, engineering aspects and the owner's decisions. Many strategies may be eliminated without detailed study and constraints of the remaining strategies should be estimated, after understanding all those factors. After that, in the preliminary design phase, approximate sizes and preferred location for all elements of the related retrofit strategy should be determined and the evaluation of the retrofit strategy should be made; whether it achieves the desired performance level or not [14].

5. CASE STUDY

The “Haldun Taner Theatre Hall” (1927) is selected to investigate the seismic performance of a special structure according to its historical importance and structurally old design techniques for reinforced concrete building aspects.

5.1. Introduction

The building consists of four singular buildings, each of them is separated by dilatation joints and these are buildings in which seismic loads are fully resisted by frames (systems of nominal ductility level). The parts of the building can be seen in Figure 5.1 as Haldun Taner Theatre Hall, Building of Istanbul University School of Music and Drama, The Left Side Building and The Right Side Building.

The original structural and architectural design works which were prepared in 1927 can not be found. The architectural draft plan of the building is prepared and the location of structural members, dimensions of structural members, the location grid lines and the location dilatation joints are all found considering this building survey. The existing concrete compressive strength and the yield strength of the reinforcement are taken according to the material report. The compressive strength of the concrete is determined in line with the concrete hammer readings and core sample experiment results. The steel class is determined with visual inspection carried on the surfaces that are peeled off. The reinforcement application plans can not be found so the reinforcement is modified according to the minimum reinforcement ratio and the ultimate load design procedure, the reinforcement ratios are consistent with the results of ferroscan examination and core specimen results.

Because of the fact that; some of indefinite approaches are used for the building geometry, member details and material properties during all evaluations for the building limited information level in considered (TEC 2007 section 2.7).

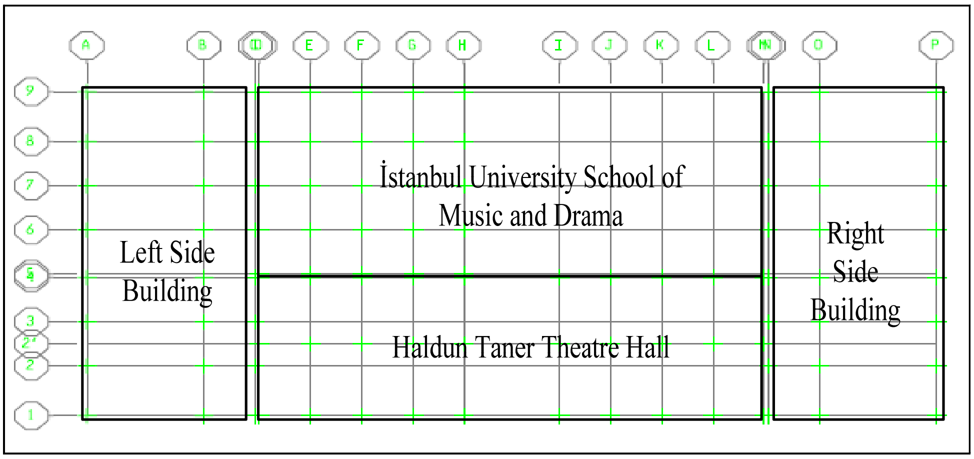

Figure 5.1. Haldun Taner Theatre Hall layout plan

During the investigation just one block of the four parts which is now being used as Building of Istanbul University School of Music and Drama is taken as an example.

In the following sections the building will be investigated for the performance based design process according to the "incremental pushover analysis" which is explained in the current design code TEC 2007. Then, if the building is decided as incapable to perform the desired performance levels or to resist the earthquake ground motion, it will be strengthened by different methods and these strategies will be compared according to their results.

5.1.1. General Information about the Building

This building consists of two floors; first floor has a height of 3.25 m, second floor has a height of 3.4 m and one top floor which has a lightweight steel roof on top of it and has a height of 2.6 m .The total height of the building is 9.25m and it consists of three floors. In the X direction it has 10 grid lines and has a length of 38m.while it has 3 grid lines along the Y direction and has a length of 12.5m. The floor plans of the building can be seen in Figure 5.2, Figure 5.3 and Figure 5.4.

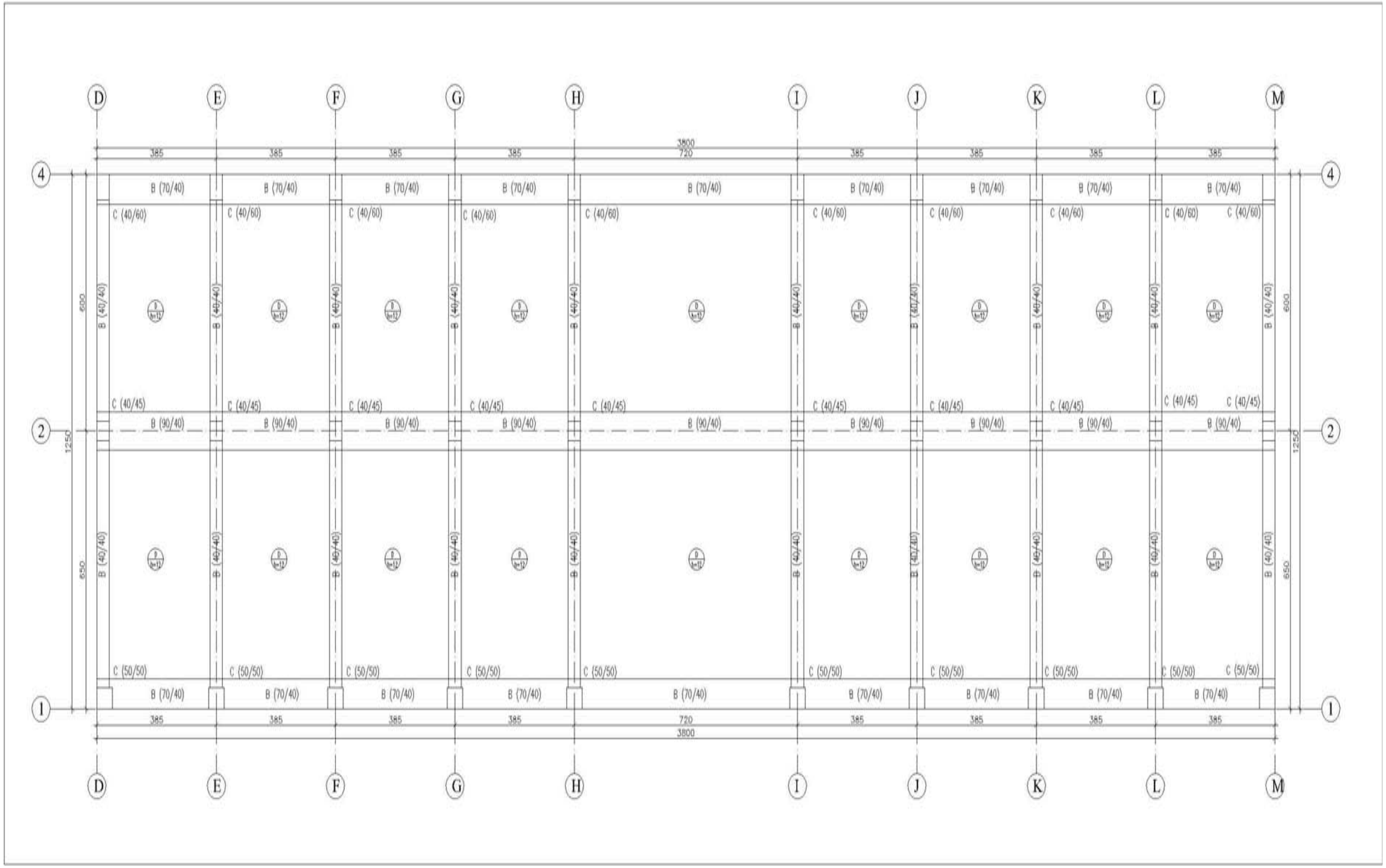

Figure 5.2. Plan view of the first storey

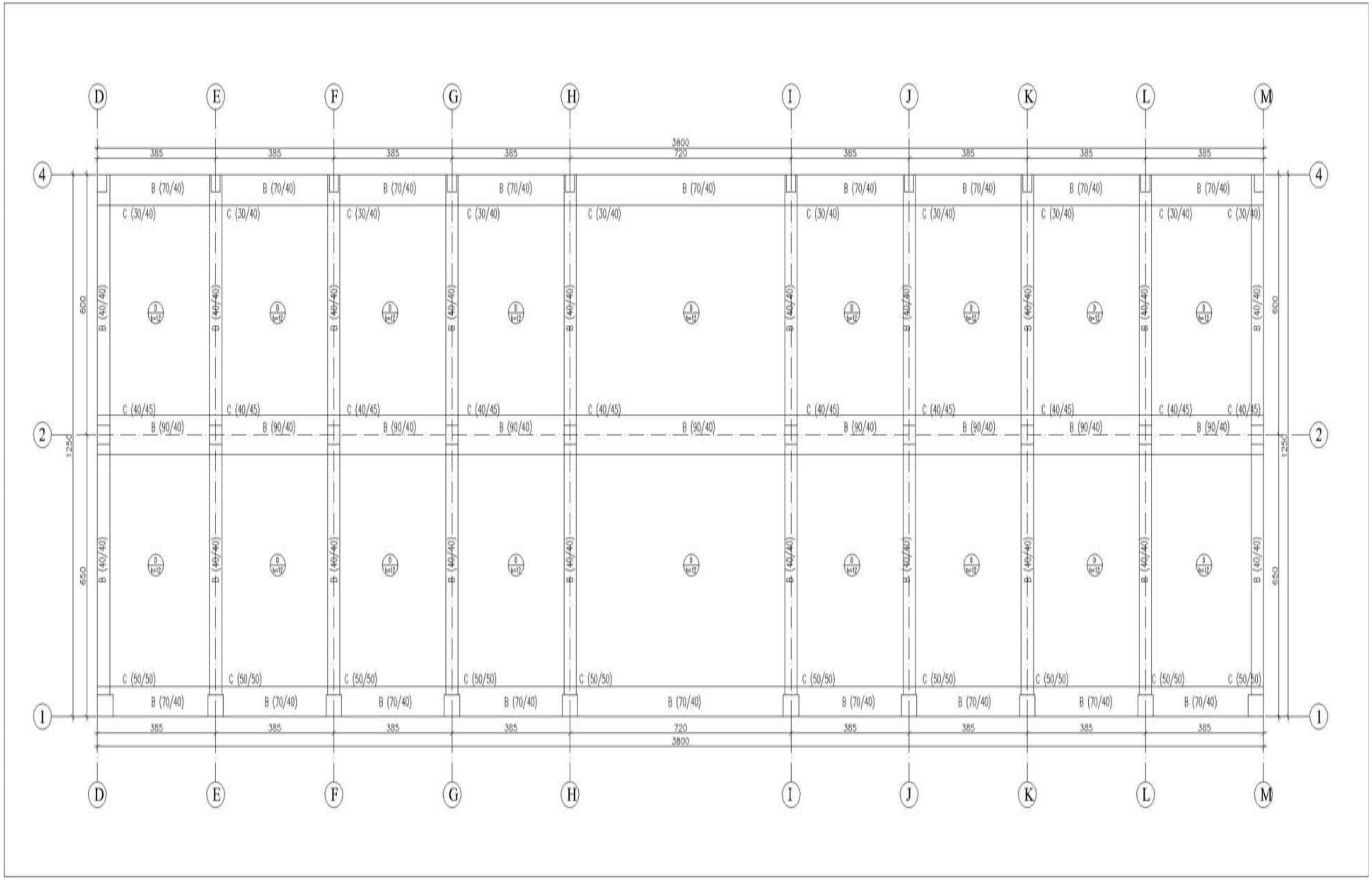

Figure 5.3. Plan view of the second storey

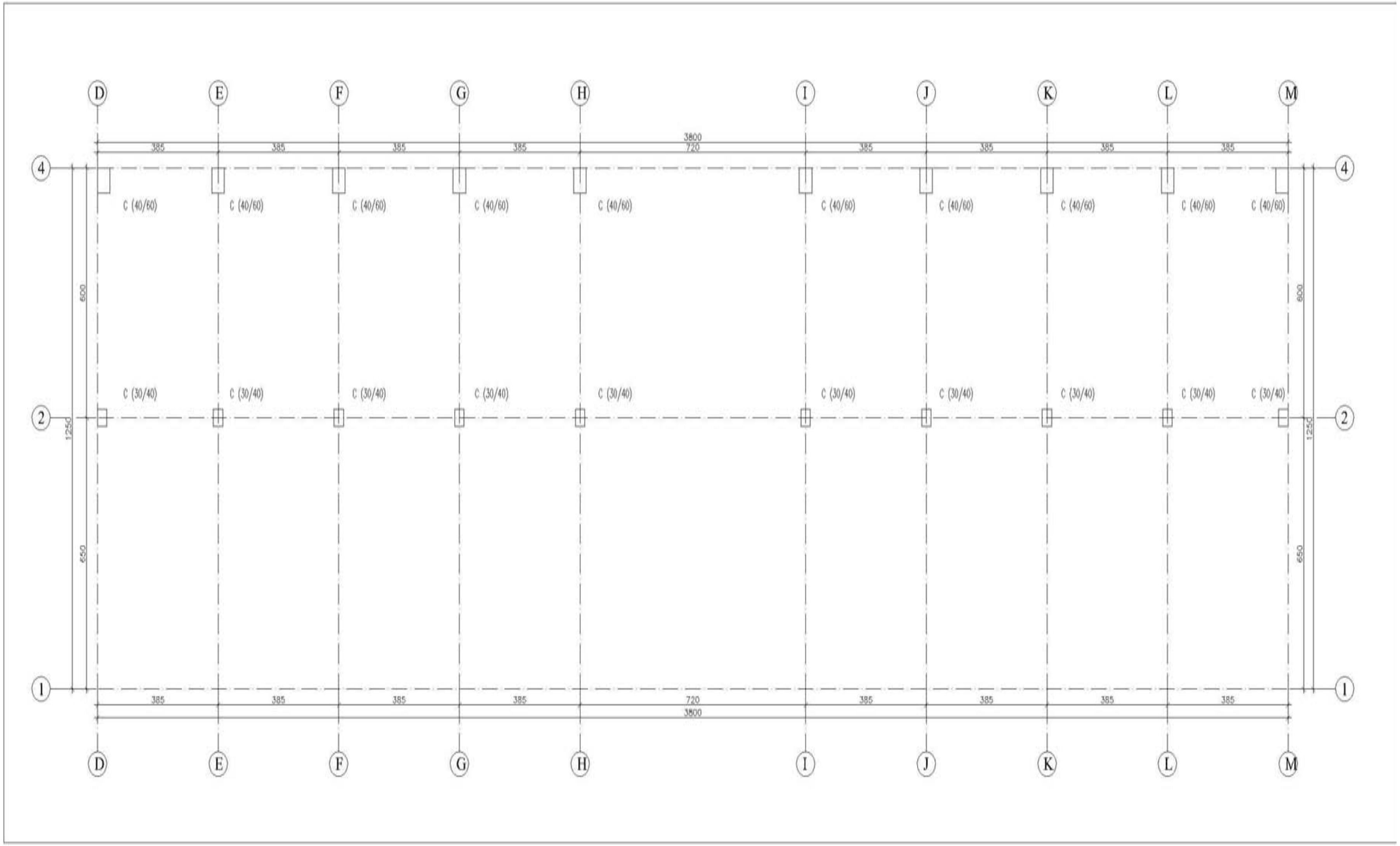

Figure 5.4. Plan view of the third storey

The building has four types of column sections which are 30x40, 40x45, 50x50 and 40x60, all of them have %1 reinforcement area and the same longitudinal reinforcement is used in all three stories. The reinforcement details of the columns can be seen in Figure 5.5. The stirrups are Ø8 at a spacing of 250 mm along the entire length of the column. For the concrete cover; the clean cover is 20 mm and cover to the center of longitudinal reinforcement is taken as approximately 35 mm for all columns. Because of the fact that the building has a moderate information level according to TEC 2007 (7.2.5) all these reinforcement details are assumed as they are consistent with the results of ferroscan examination and core specimen results.

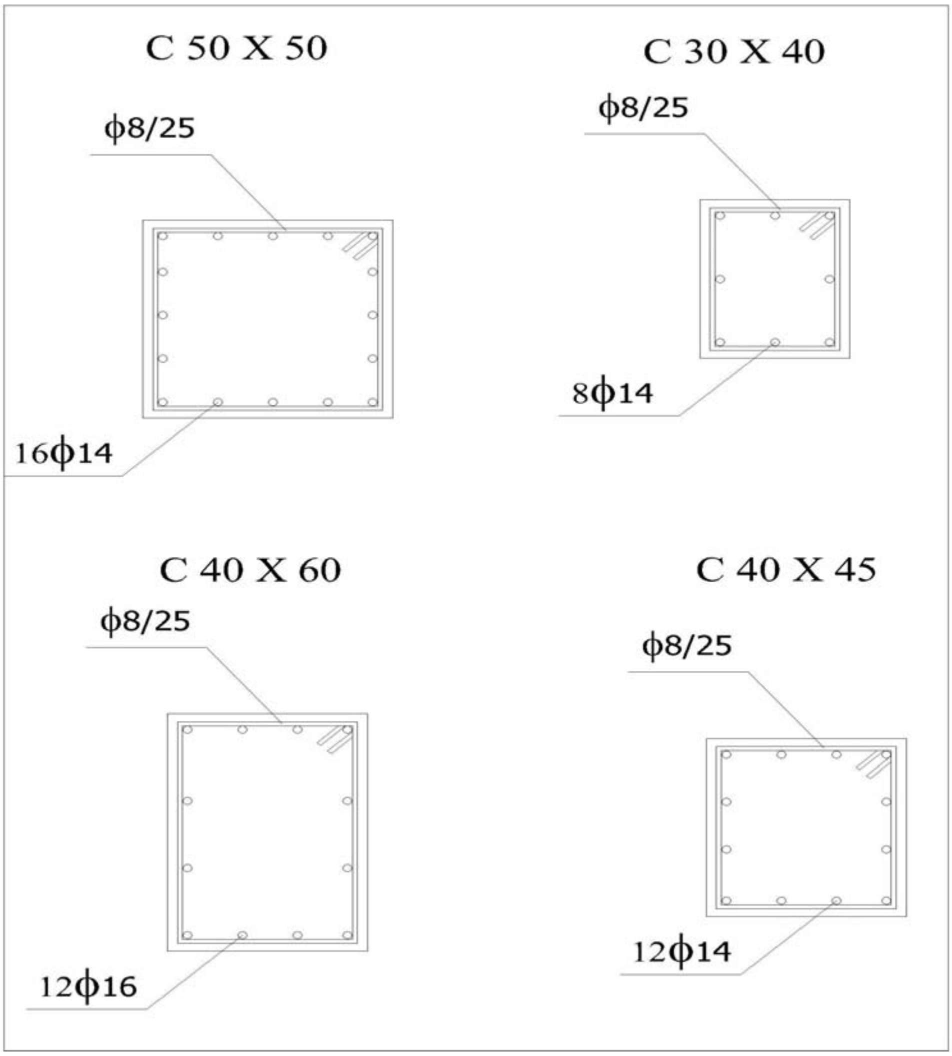

Figure 5.5. Reinforcement details of columns

There are two types of beams in the X direction 90x40 and 70x40, one type of beam in the Y direction 40x40 and these are plane girders dissimilar from today's deep beam concept. The beams are grouped into eight for 90x40 and 70x40 and grouped into 3 for 40x40 beams in the Y direction according to their reinforcement details. The reinforcement details of the beam groups can be seen in Table 5.1 and the location of these beam types in the floor plan can be seen in Figure 5.6.

Table 5.1. Beam types

Beam Type	Top Reinf.	Bottom Reinf.	Section
1a	2ø14+4ø14	4ø12	70x40
2a	4ø14+2ø16	3ø14	70x40
2b	2ø16+3ø20	3ø14	70x40
3a	4ø16+3ø20	4ø20	70x40
4a	2ø16+4ø20	3ø20	90x40
5a	2ø20+4ø20	2ø20	90x40
5b	2ø20+7ø20	2ø20+2ø16	90x40
6a	3ø20+7ø20	7ø20+2ø16	90x40
7a	3ø16+2ø14	3ø16	40x40
8a	3ø16+2ø14	3ø16	40x40
9a	3ø20+2ø16	3ø20	40x40

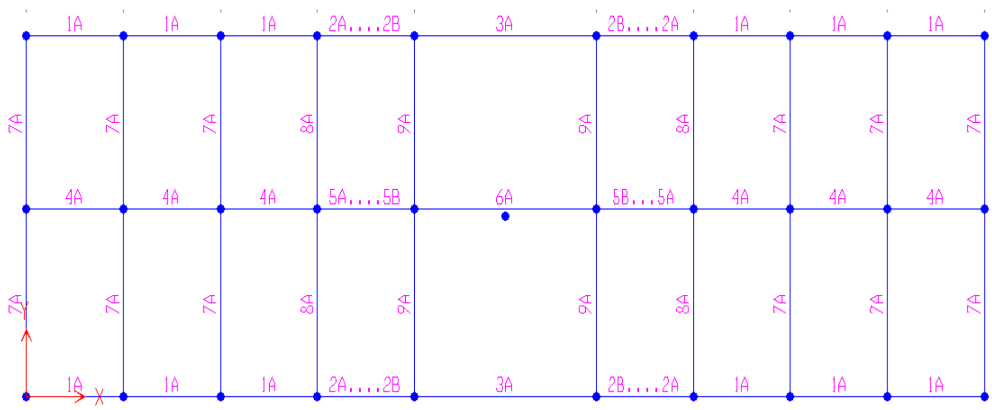

Figure 5.6. Location of beam types

The slabs have a thickness of 12 cm and they have 20x40 girders in the Y direction and these girders create a one way slab and they translate the vertical loads to the 90x40 and 70x40 beams in X direction. The 40x40 beams in the Y direction provide a small portion of the vertical loads because of that design concept. During the evaluation of the building the 20x40 girders and their member performance works are not considered because they are subsidiary structural members.

5.1.2. Structural System Models

The structural model of the system is prepared by the SAP 2000 v11.0.0 Structural Analysis Program. The structural members are modeled via frame elements. The vertical loads on the slabs are distributed to beams according to one way working slab procedure. The slabs for the first and second story defined as rigid diaphragm and the roof is assumed as, it has full rigid connections to the column ends. Concrete class and reinforcing steel class are chosen as C10 (compressive strength of concrete $f_{ck} = 10 MPa$) and S220 ($f_{yk} = 220 MPa$) respectively. It is assumed that building is located on first seismic zone and Z3 soil type. The 3D model of the building prepared via SAP2000 can be seen in Figure 5.7

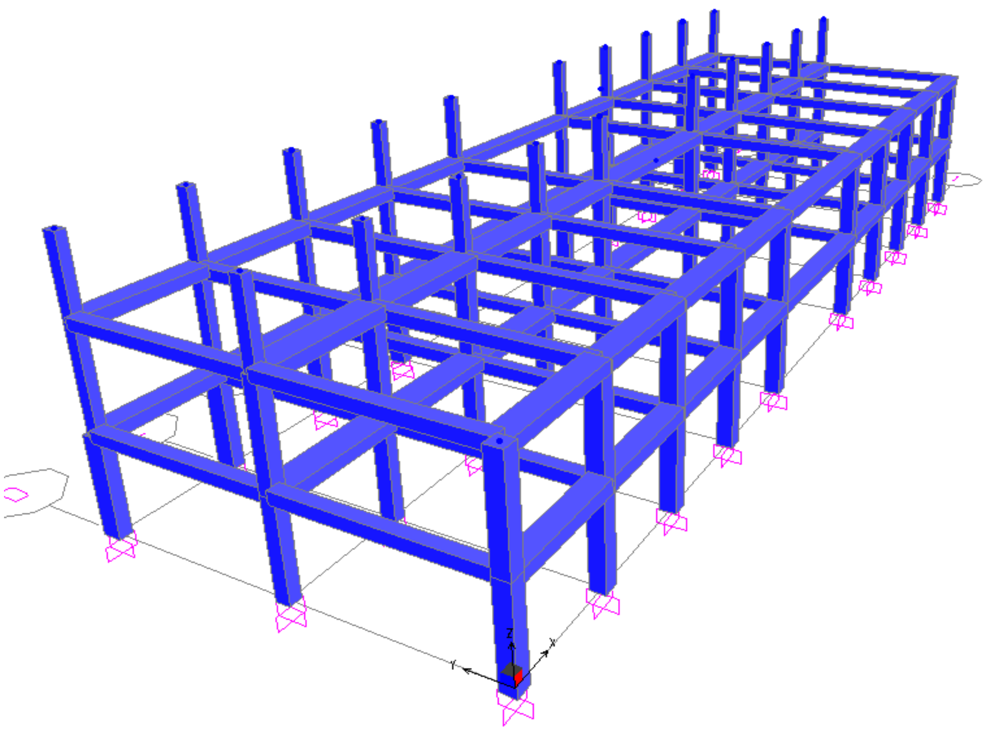

Figure 5.7. The 3D computer model of the structure

5.2. Story Drift Limitation and Irregularity Control under Elastic Earthquake Loading

Before the seismic evaluation of the building under nonlinear analysis, the story drift limitations and irregularities in plan and in elevation will be controlled under elastic earthquake loads and equivalent seismic load method will be used for lateral load concept according to current design code TEC 2007.

5.2.1. Irregularities in Plan (A)

5.2.1.1. Torsional Irregularity (A1) The case where '*Torsional Irregularity Factor'* η_{bi}, which is defined as the ratio of the maximum storey drift at any storey to the average storey drift at the same storey in the same direction for any two orthogonal earthquake directions, is greater than 1.2

$$\eta_{bi} = \frac{(\Delta_i)_{\max}}{(\Delta_i)_{ort}} > 1.2 \tag{5.1}$$

5.2.1.2. Floor Discontinuities (A2) In any floor, below cases determine the floor discontinuities:

- The case where the total area of the openings including those of stairs and elevator shafts exceeds 1/3 of the gross floor area,
- The cases where local floor openings make it difficult the safe transfer of seismic loads to vertical structural elements,
- The cases of abrupt reductions in the in-plane stiffness and strength of floors.

5.2.1.3. Projections in Plan (A3) The cases where projections beyond the re-entrant corners in both of the two principal directions in plan exceed the total plan dimensions of the building in the respective directions by more than 20%.

5.2.2. Irregularities in Elevation (B)

5.2.2.1. Interstorey Strength Irregularity (Weak Storey) (B1) In reinforced concrete buildings, the case where '*Strength Irregularity Factor*', η_{ci}, which is defined as the ratio of the effective shear area of any storey to the effective shear area of the storey immediately above for any two orthogonal earthquake directions, is less than 0.80.

$$\eta_{ci} = \frac{(\Sigma A_e)_i}{(\Sigma A_e)_{i+1}} < 0.80 \tag{5.2}$$

Definition of effective shear area in any storey:

$$\Sigma A_e = \Sigma A_w + \Sigma A_g + 0.15 \times \Sigma A_k \tag{5.3}$$

5.2.2.2. Interstorey Stiffness Irregularity (Soft Storey) (B2) The case where in each of the two orthogonal earthquake directions, '*Stiffness Irregularity Factor*', η_{ki}, which is defined as the ratio of the average storey drift at any storey to the average storey drift at the storey immediately above or below, is greater than 2.

$$\eta_{ki} = \frac{(\Delta_i / h_i)_{avg}}{(\Delta_{i+1} / h_{i+1})_{avg}} > 2 \tag{5.4}$$

$$\eta_{ki} = \frac{(\Delta_i / h_i)_{avg}}{(\Delta_{i-1} / h_{i-1})_{avg}} > 2 \tag{5.5}$$

5.2.2.3. Discontinuity of Vertical Structural Elements (B3) The cases where vertical structural elements (columns or structural walls) are removed at some stories and supported by beams or gusseted columns underneath, or the structural walls of upper stories are supported by columns or beams underneath.

5.2.3. Limitation of Storey Drifts

The reduced storey drift, Δi , of any column or structural wall shall be determined by the below equation as the difference of displacements between the two consecutive stories.

$$\Delta_i = d_i - d_{i-1} \tag{5.6}$$

In the above equation d_i and d_{i-1} shows the calculated displacements according to the reduced elastic earthquake loads at the column or shear wall ends of the i^{th} and $(i-1)^{th}$ stories of the building. For each earthquake direction, the relative story drift, δ_i , at the i th story of the building is calculated by the Equation 5.7.

$$\delta_i = R \times \Delta_i \tag{5.7}$$

The maximum relative value of storey drifts within a storey, $(\delta_i)_{max}$, for columns and structural walls of the i^{th} storey of a building for each earthquake direction shall satisfy the condition given by the Equation 5.8.

$$(\delta_i)_{max} / h_i \le 0.02 \tag{5.8}$$

5.3. Earthquake Characteristics and Load Analysis

Earthquake characteristics of the structural system which are evaluated and controlled on the basis of the Turkish Earthquake Code 2007 are given in Table 5.2.

Table 5.2. Model parameters

Building Importance Factor: I =1.4
Live Load Participation Factor: n = 0.60
Effective Ground Acceleration Coefficient: A_0 = 0.40
Local Site Class: Z3
Spectrum Characteristic Periods: T_A =0.15 s. T_B = 0.40 s.
Structural Behavior Factor: R = 4
Compressive Strength of Concrete: 10 MPa
Yield Stress for Rebar: 220 MPa
Modulus of Elasticity: 15811000 kN/m^2

Vertical loads determined according to TS 498 [20] are given below. They are used in determining the elastic seismic loads to perform the desired checks for the structural system model.

First and second story,

$g = 4.50\ kN/m^2$ (including slabs)

$q = 5.00\ kN/m^2$ (including infill walls)

Roof story,

$g = 1.50\ kN/m^2$

$q = 0.75\ kN/m^2$ (just snow load)

The self weights of the structural elements are calculated according to their sections, heights and 25 kN/m^3 specific weight of the reinforced concrete.

5.3.1. Determination of Total Equivalent Seismic Load

Total Equivalent Seismic Load (base shear), V_t, acting on the entire building in the earthquake direction considered shall be determined by the Equation 5.9.

$$V_t = \frac{W.A(T_1)}{R_a(T_1)} \geq 0.10 \times A_0 \times I \times W \tag{5.9}$$

Total building weight is calculated by using the Equation 5.10.

$$W = \sum_{i=1}^{N} w_i = g_i + n.q_i \tag{5.10}$$

Live load participation factor, n, is taken as 0.60 according to Table 5.3.

Table 5.3. Live load participation factor

Purpose of Occupancy of Building	n
Depot, warehouse, etc.	0.8
School, dormitory, sport facility, cinema, theatre, concert hall, car park, restaurant, shop, etc.	0.6
Residence, office, hotel, hospital, etc.	0.3

Spectral Acceleration Coefficient is calculated by using the Equation 5.11.

$$A(T) = A_0 \times I \times S(T) \tag{5.11}$$

Effective ground acceleration coefficient, A_0, is taken as 0.40 according to Table 5.4.

Table 5.4. Effective ground acceleration coefficient

Seismic Zone	A_0
1	0.4
2	0.3
3	0.2
4	0.1

The spectrum coefficient for T_1 period $S(T_1)$ can be determined by using Equation (5.12), depending on the local site conditions and the first natural vibration period of the building, T_1 (Figure 5.8).

$$
\begin{aligned}
S(T) &= 1 + 1.5\frac{T}{T_A} && (0 \le T \le T_A) \\
S(T) &= 2.5 && (T_A < T \le T_B) \\
S(T) &= 2.5\left(\frac{T_B}{T}\right)^{0.8} && (T_B < T)
\end{aligned}
\tag{5.12}
$$

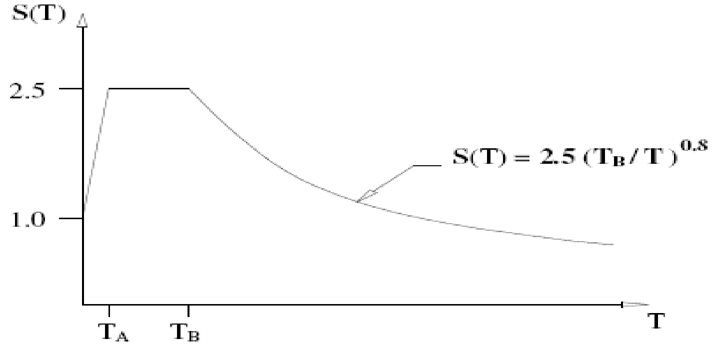

Figure 5.8. Elastic spectrum of the earthquake

Spectrum Characteristic Periods, T_A and T_B, appearing in Equation (5.12) are specified in Table 5.5, depending on Local Site Classes.

Table 5.5. Spectrum characteristic period

Local Site Class	T_A (s)	T_B (s)
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

Elastic seismic load to be determined in terms of spectral acceleration coefficient shall be divided to below defined Seismic Load Reduction Factor, defined below, to account for the specific nonlinear behavior of the structural system during earthquake.

Seismic Load Reduction Factor, $R_a(T)$, is determined by Equation (5.13) in terms of Structural Behavior Factor, R, defined for various structural systems, and the natural vibration period T.

$$R_a(T) = 1.5 + (R - 1.5)\frac{T}{T_A} \qquad (0 \le T \le T_A)$$
$$R_a(T) = R \qquad (T_A < T) \qquad (5.13)$$

The first natural vibration periods (T_1) of the structural system model is greater than T_A. So seismic load reduction factor, R_a, is equal to structural behavior factor, R. R = 4 is taken for buildings in which seismic loads are fully resisted by frames (systems of nominal ductility level) according to TEC 2007.

Excluding, ΔF_N, remaining part of the total equivalent seismic load shall be distributed to stories of the building (including Nth story) in accordance with Equation (5.14)

$$F_i = (V_t - F_N)\frac{w_i.H_i}{\sum_{j=1}^{N} w_j H_j} \qquad (5.14)$$

Height of the structural system model is less than 25 m. So ΔF_N=0 is taken for $H_N < 25m$ according to TEC 2007.

For the structural system of building the base shear forces, drift controls and irregularity checks are done in two directions of earthquake +X and +Y because of the symmetry.

T_{1X} = 0.699 sec $\qquad S(T_{1X})$= 2.212 $\qquad A(T_{1X})$=1.239

T_{1Y}= 0.839 sec $\qquad S(T_{1Y})$= 1.912 $\qquad A(T_{1Y})$=1.071

Total weight of the building;

$W_{G+0.3Q}$ = 5442.4 + 5449.8 + 376.3 = 11540.7 kN

$$V_{tX} = \frac{W \times A(T_1)}{R_a(T_1)} = 3574.73\ kN \qquad V_{tY} = \frac{W \times A(T_1)}{R_a(T_1)} = 3090.2\ kN$$

Total equivalent seismic load that will distributed to the stories of the building is given in Table 5.6 for X direction and in Table 5.7 for Y direction.

Table 5.6. Distribution of seismic loads to the stories (X direction)

Storey	$H_i(m)$	$W_i(kN)$	$H_i \times W_i$	$\frac{H_i \times W_i}{\sum H_j \times W_j}$	$F_i(kN)$
3	9.25	376.3	3480.8	0.07	250.23
2	6.65	5449.8	36241.2	0.631	2255.55
1	3.25	5442.4	17687.8	0.308	1101.1

Table 5.7. Distribution of seismic loads to the stories (Y direction)

Storey	$H_i(m)$	$W_i(kN)$	$H_i \times W_i$	$\frac{H_i \times W_i}{\sum H_j \times W_j}$	$F_i(kN)$
3	9.25	376.3	3480.8	0.07	216.31
2	6.65	5449.8	36241.2	0.631	1949.92
1	3.25	5442.4	17687.8	0.308	951.78

- The displacement results and the torsional irregularity (A1) factors are found for the direction X and Y as it can be seen in Table 5.8. Any torsional irregularity problems was faced in the structural system at each storey level.

Table 5.8. Torsional irregularity check for the structure (X and Y directions)

Storey	Direction	$(\Delta_i)_{\max}(cm)$	$(\Delta_i)_{\min}(cm)$	$(\Delta_i)_{avg}(cm)$	η_{bi}
3	X	7.04	6.66	6.85	1.03
3	Y	9.26	9.26	9.26	1
2	X	5.14	4.44	4.79	1.07
2	Y	6.59	6.59	6.59	1
1	X	2.01	1.85	1.93	1.04
1	Y	2.32	2.32	2.32	1

- As a type of irregularity in plan there is no floor discontinuity (A2) is met, because the cases defined in the 5.2.1.2 are satisfied at each story level.
- There is not any case where the projections beyond the re entrant corners in both of the earthquake directions exceed the 20 percent of the total plan dimensions of the building. There are no projections in plan irregularity. (A3).
- The total area for the columns for each story level and the ratio between the i^{th} and $(i+1)^{th}$ stories of the building is given in the Table 5.9. As it can be seen below there is no weak storey condition (Interstorey strength irregularity) (B1)

Table 5.9. Interstorey strength irregularity (weak storey) control

Storey	$\Sigma A_k (m^2)$	$\Sigma A_e (m^2)$	η_{ci}
3	6.7	1.005	-
2	5.5	0.825	1.22
1	2.4	0.36	2.29

- The displacements for each storey level, the ratios of the average storey drifts to the average storey drift at the storey immediately above or below and the stiffness irregularity factor, η_{ki}, which were given by Equations 5.4 and 5.5 are given in Table 5.10 . According to this evaluation there was a soft storey condition is determined for the second storey.(B2)

Table 5.10. Interstorey stiff. irregularity (B2) (soft storey) control

Storey	Direction	$\Delta_i (cm)$	$h_i (cm)$	$\frac{\Delta_i}{h_i}(10^{-3})$	η_{ki}
3	X	6.85	260	26.3	1.86 (3/2)
3	Y	9.26	260	35.6	1.84 (3/2)
2	X	4.79	340	14.1	0.54 (2/3) **2.37 (2/1)**
2	Y	6.59	340	19.3	0.54 (2/3) **2.70 (2/1)**
1	X	1.93	325	5.94	0.42 (1/2)
1	Y	2.32	325	7.14	0.54 (1/2)

- There is not any discontinuity for the vertical structural elements in the building, so it can be said that there is no this type of elevation irregularity for the building (B3).
- The displacements, reduced storey drift, Δ_i and the $\frac{(\delta_i)_{\max}}{h_i}$ can be seen in the Table 5.11. The building did not satisfy the story drift limitations at any stories. R = 4 is taken for buildings in which seismic loads are fully resisted by frames (systems of nominal ductility level).

Table 5.11. Story drift limitation check

Storey	Direction	$d_i(cm)$	$\Delta_i(cm)$	$\frac{(\delta_i)_{\max}}{h_i}$
3	X	7.04	1.90	0.0291
3	Y	9.26	2.67	0.0412
2	X	5.14	3.13	0.0372
2	Y	6.59	4.27	0.0501
1	X	2.01	2.01	0.0247
1	Y	2.32	2.32	0.0285

6. PERFORMANCE ANALYSIS WITH INCREMENTAL EQUIVALENT SEISMIC LOAD METHOD

The incremental equivalent seismic load method is explained in the section 3.7 according to section 7.6 of the TEC 2007 regulations. In this section the performance of the Haldun Taner Theatre Hall is evaluated with a nonlinear analysis method, incremental equivalent seismic load method. During these analyses SAP2000 V11.0.0 software is utilized and the plastic hinge parameters and locations are assigned to the structural system model considering the TEC 2007.

6.1. Properties and Dynamic Characteristics of the Building

The parameters for dynamic characteristic of the building are shown in Table 6.1.

Table 6.1. Dynamic characteristic of the building

Effective Ground Acceleration Coefficient: A_0 = 0.40
Local Site Class: Z3
Spectrum Characteristic Periods: T_A =0.15 s. T_B = 0.40 s.
Compressive Strength of Concrete (existing): 10 MPa
Yield Stress for Reinforcement Steel (existing): 220 MPa
Modulus of Elasticity: 15811000 kN/m^2
Building Information Level Factor: 0.75 (Limited Information Level)

This method can be applied to the structural system model because it consists of 3 stories the torsional irregularity coefficient is $\eta_{bi} < 1.4$ as it can be seen in Table 5.8 torsional irregularity check for the structure. The effective mass to total building mass corresponding to the first natural vibration mode in X direction is 0.833, in Y direction it is 0.799 both of them are above the value 0.70.

$$T_{1X} = 0.699\,\text{sec}. \qquad T_{1Y} = 0.839\,\text{sec}.$$

The *section stiffness for cracked sections* $(EI)_e$ is used for linear behavior of reinforced concrete members under the effect of bending before yielding. According to TEC2007, the section stiffness for cracked sections which is used to consider the bending stiffness of cracked reinforced concrete members change under the effect of various axial loads *(G+Q)* and this is also defined in the 3.7 of this study. The section stiffness is accepted as 0.4 for cracked beam sections. The section stiffness of columns can be seen in Appendix A in which for the intermediate values of the axial compression load N_D linear interpolation is assessed.

The stories are accepted as fully rigid diaphragm, although the third story has light steel roof structure not a reinforced concrete slab.

6.2. Definition of Plastic Hinges

The effective yield moments are given in the 5.1.1 and the yield curvatures which are described at these yield moments are named as effective yield curvatures. Although this description is used in this study, there are also more practical definitions about the yield curvature. For example Priestley pointed out that the yield curvature for the T-beam is defined as the division of 1.7 times of the strain value at the first yielding point of the steel to the depth of the beam [21].

The member damage strain capacities of the reinforced concrete beams can be calculated by the formulas which are described in the 3.11 and the results can be seen in Table 6.2.

$$\begin{aligned}
&Case\,1:\ (\varepsilon_{cu})_{MN} = 0.0035, && (\varepsilon_s)_{MN} = 0.010 \\
&Case\,2:\ (\varepsilon_{cg})_{GV} = 0.0035 + 0.01(\rho_s / \rho_{sm}) \le 0.0135, && (\varepsilon_s)_{GV} = 0.040 \\
&Case\,3:\ (\varepsilon_{cg})_{GC} = 0.004 + 0.014(\rho_s / \rho_{sm}) \le 0.018, && (\varepsilon_s)_{GC} = 0.060
\end{aligned} \tag{6.1}$$

In this case study; strain hardening is not considered for the definition of plastic hinges.

Table 6.2. Member damage strain capacities for beams

Section	ρ_s / ρ_{sm}	$(\varepsilon_{cu})_{MN}$ (10^{-3})	$(\varepsilon_{cg})_{GV}$ (10^{-3})	$(\varepsilon_{cg})_{GÇ}$ (10^{-3})	$(\varepsilon_s)_{MN}$ (10^{-3})	$(\varepsilon_s)_{GV}$ (10^{-3})	$(\varepsilon_s)_{GÇ}$ (10^{-3})
B40x40	0.202	3.5	5.52	6.83	10	40	60
B70x40	0.188	3.5	5.38	6.65	10	40	60
B90x40	0.198	3.5	5.48	6.77	10	40	60

The yield curvature values and the total curvature values which are calculated with the section analysis program XTRACT regarding the member damage strain capacities for the reinforced concrete members defined in the TEC 2007. These values calculated for the beam types, can be seen in the Table 6.3.

Table 6.3. Plastic curvature values for the beam damage levels

Beam Type	Plastic Curvatures for the Positive Yield Moment (1/m) (10^{-3})				Plastic Curvatures for the Negative Yield Moment (1/m) (10^{-3})			
	Yield	MN	GV	GC	Yield	MN	GV	GC
1a	4.67	30.73	35.3	199.1	4.38	29.6	131.4	193.82
2a	4.78	31.21	137.72	199.22	4.33	29.97	133.68	198.87
2b	5.02	32.78	135.78	212.78	4.43	30.58	138.97	200.77
3a	5.27	32.32	141.73	202.53	4.89	31.37	140.41	197.71
4a	5.02	32.62	143.78	210.38	4.54	31.26	138.86	203.66
5a	5.19	33.01	138.31	211.61	4.35	31.02	140.45	210.55
5b	5.49	30.61	116.81	141.61	4.54	31.22	138.76	207.86
6a	5.64	33.12	143.46	206.16	5.28	31.52	140.72	204.32
7a	5.08	27.87	139.12	207.52	4.63	30.72	135.57	202.77
8a	5.08	27.87	139.12	207.52	4.63	30.72	135.57	202.77
9a	5.49	32.99	142.11	211.31	5.05	30.74	140.45	210.35

The plastic curvature values are achieved by subtracting the yield curvature values from the total curvature values. Plastic curvature value times the plastic hinge length gives the plastic rotation value for a specific damage level. The plastic rotation values which are calculated for each beam damage level are assigned to the SAP2000 program as multiples of a reference value and these values are given in the Table 6.4. For this study this reference value is selected as the yield rotation and the other limit values are defined as referring this value. By the help of the above statement, the member damage values for each damage level can be obtained as a print out from the program.

Table 6.4. Rotation factors for the beam damage levels

Beam Type	Rotation Factors for the Positive Yield Moment (1/m)				Rotation Factors for the Negative Yield Moment (1/m)			
	Ref (10^{-3})	MN	GV	GC	Ref (10^{-3})	MN	GV	GC
1a	0.934	6.57	28.97	42.63	0.876	6.76	30	44.25
2a	0.956	6.53	28.81	41.68	0.866	6.92	30.87	45.93
2b	1.004	6.53	27.41	42.39	0.886	6.90	31.37	45.32
3a	1.054	6.13	26.89	38.43	0.978	6.41	28.71	40.43
4a	1.004	6.50	28.64	41.91	0.908	6.88	30.58	44.86
5a	1.038	6.36	26.65	40.77	0.870	7.13	32.29	48.40
5b	1.098	5.57	21.28	25.79	0.908	6.88	30.56	45.78
6a	1.128	5.87	25.44	36.55	1.056	5.97	26.65	38.70
7a	1.016	5.49	27.38	40.85	0.926	6.63	29.28	43.79
8a	1.016	5.49	27.38	40.85	0.926	6.63	29.28	43.79
9a	1.098	6.01	25.88	38.49	1.010	6.09	27.81	41.65

The yield surfaces for the rectangular columns which have symmetric reinforcement are defined with three yield curves for the angles 0, 45 and 90. As it is considered in TEC 2007 the maximum compression strain for the concrete is taken as 0.003, the maximum strain for the reinforcement was taken as 0.01. Axial load versus curvature values of these yield surfaces are calculated by the section analysis program XTRACT.

After the definition of plastic hinges both for beams and columns, in the SAP2000 program the beam hinges are assigned on the column faces and the column hinges assigned as bottom hinge, on the joint and upper hinge, on the beam face of the next storey.

6.3. Derivation of Capacity Curve

After, all the plastic properties of the members are defined; the structural system model is pushed to the displacement that the related seismic demand with an addition of the vertical loads on the system. The building has to be pushed to that displacement with a required lateral load distribution. According to TEC 2007, this lateral load distribution has to be proportional with the multiplication of the shape of the first mode for the given earthquake direction with the mass. While the displacement and internal forces are increasing, some members reach their ultimate load capacity and plastic hinges exist for those members. The members with the plastic hinges continue to rotate without a change in their load conditions. Between the existences of two plastic hinges the structural system behaves linear elastic and at the end the system reaches an unstable condition and finally its collapse level.. To convert the “top displacement – base shear” diagram to “modal displacement – modal acceleration” diagram Equations 6.2, 6.3, 6.4, 6.5, 6.6 and 6.7 were used.

$$M^*_{x1} = \left[\Phi_{x1}{}^T\right] \times [m] \times [\Phi_{x1}] \tag{6.2}$$

$$L^*_{x1} = \left[\Phi_{x1}{}^T\right] \times [m] \times [I] \tag{6.3}$$

$$\Gamma_{x1} = L^*_{x1} / M^*_{x1} \tag{6.4}$$

$$M_{x1} = L^{*2}{}_{x1} / M^*_{x1} \tag{6.5}$$

$$d_1 = u_{xN1} / (\Phi_{xN1}\, \Gamma_{x1}) \tag{6.6}$$

$$a_1 = V_{x1} / M_{x1} \tag{6.7}$$

Lateral Load distribution for the earthquake direction *X*,

$$\begin{Bmatrix} 0.0147 \\ 0.0374 \\ 0.0519 \end{Bmatrix} \begin{Bmatrix} 554.78 \\ 555.53 \\ 38.36 \end{Bmatrix} = \begin{Bmatrix} 8.155 \\ 20.777 \\ 1.991 \end{Bmatrix} \rightarrow \begin{Bmatrix} 1 \\ 2.547 \\ 0.244 \end{Bmatrix} \begin{pmatrix} 1^{st}\, storey \\ 2^{nd}\, storey \\ 3^{rd}\, storey \end{pmatrix}$$

Lateral Load distribution for the earthquake direction *Y*,

$$\begin{Bmatrix} 0.0130 \\ 0.0379 \\ 0.0530 \end{Bmatrix} \begin{Bmatrix} 554.78 \\ 555.53 \\ 38.36 \end{Bmatrix} = \begin{Bmatrix} 7.212 \\ 21.054 \\ 2.033 \end{Bmatrix} \rightarrow \begin{Bmatrix} 1 \\ 2.919 \\ 0.282 \end{Bmatrix} \begin{pmatrix} 1^{st}\, storey \\ 2^{nd}\, storey \\ 3^{rd}\, storey \end{pmatrix}$$

For the earthquake direction *X* at the first step of analysis,

$$M^*_{x1} = \begin{bmatrix} 0.283 & 0.721 & 1 \end{bmatrix} \begin{bmatrix} 554.78 & 0 & 0 \\ 0 & 555.53 & 0 \\ 0 & 0 & 38.36 \end{bmatrix} \begin{bmatrix} 0.283 \\ 0.721 \\ 1 \end{bmatrix} = 371.4\ kNs^2 / m$$

$$L^*_{x1} = \begin{bmatrix} 0.283 & 0.721 & 1 \end{bmatrix} \begin{bmatrix} 554.78 & 0 & 0 \\ 0 & 555.53 & 0 \\ 0 & 0 & 38.36 \end{bmatrix} \begin{bmatrix} 1 \\ 1 \\ 1 \end{bmatrix} = 595.8 \; kNs^2/m$$

$$\Gamma_{x1} = L^*_{x1} / M^*_{x1} = 595.8 / 371.4 = 1.604$$

$$M_{x1} = L^{*2}_{x1} / M^*_{x1} = 595.8^2 / 371.4 = 955.98 \text{ kNs}^2\text{/m}$$

For the earthquake direction Y at the first step of analysis,

$$M^*_{y1} = \begin{bmatrix} 0.245 & 0.715 & 1 \end{bmatrix} \begin{bmatrix} 554.78 & 0 & 0 \\ 0 & 555.53 & 0 \\ 0 & 0 & 38.36 \end{bmatrix} \begin{bmatrix} 0.245 \\ 0.715 \\ 1 \end{bmatrix} = 355.8 \; kNs^2/m$$

$$L^*_{y1} = \begin{bmatrix} 0.245 & 0.715 & 1 \end{bmatrix} \begin{bmatrix} 554.78 & 0 & 0 \\ 0 & 555.53 & 0 \\ 0 & 0 & 38.36 \end{bmatrix} \begin{bmatrix} 1 \\ 1 \\ 1 \end{bmatrix} = 571.7 \; kNs^2/m$$

$$\Gamma_{y1} = L^*_{y1} / M^*_{y1} = 571.7 / 355.8 = 1.607$$

$$M_{y1} = L^{*2}_{y1} / M^*_{y1} = 571.7^2 / 355.8 = 918.56 \text{ kNs}^2\text{/m}$$

In TEC 2007 for incremental equivalent seismic load method it is stated that, the load distribution shape can be taken as the first mode shape for the related earthquake direction. Although the mode shape changes after each plastic hinge formation, the displacement values can be accepted as the values which are calculated for the first mode. Because of that, the modal capacity curves are formed regarding above calculations. The values for these capacity curves for earthquake direction X is given in Table 6.5 and in Table 6.6 for the direction Y.

Table 6.5. The values for the modal capacity curve in direction X

$u_{xN1}{}^{(i)}(mm)$	$V_{xN1}{}^{(i)}(kN)$	$d_1{}^{(i)}(m)$	$a_1{}^{(i)}(g)$
0	0	0	0
12.5	656.61	0.007791	0.0700148
15.964	838.557	0.00995	0.089416
29.032	1460.96	0.018094	0.1557833
41.754	1847.485	0.026023	0.1969987
56.672	1994.833	0.035321	0.2127106
73.034	2093.862	0.045519	0.2232701
87.912	2169.152	0.054792	0.2312983
103.025	2251.255	0.064211	0.240053
116.242	2315.801	0.072448	0.2469356
133.176	2373.817	0.083003	0.2531219
148.127	2418.764	0.092321	0.2579146
161.536	2457.807	0.100678	0.2620778
179.536	2505.815	0.111897	0.267197
192.108	2536.418	0.119732	0.2704602
210.449	2575.931	0.131163	0.2746735
224.762	2608.884	0.140084	0.2781873
237.832	2624.68	0.14823	0.2798716
250	2647.333	0.155814	0.2822871

Table 6.6. The values for the modal capacity curve in direction Y

$u_{yN1}{}^{(i)}(mm)$	$V_{yN1}{}^{(i)}(kN)$	$d_1{}^{(i)}(m)$	$a_1{}^{(i)}(g)$
0	0	0	0
11.738	400.192	0.0073055	0.0444113
20.125	668.699	0.0125255	0.0742089
33.514	1031.691	0.0208586	0.114492
46.556	1296.466	0.0289757	0.1438754
65.119	1545.52	0.040529	0.1715142
78.174	1649.458	0.0486542	0.1830487
95.121	1730.359	0.0592018	0.1920267
109.26	1770.928	0.0680017	0.1965288
124.001	1819.276	0.0771762	0.2018942
138.26	1847.096	0.0860508	0.2049816
160.528	1884.975	0.09991	0.2091852
173.028	1907.607	0.1076898	0.2116968
189.176	1933.05	0.1177401	0.2145203
205.915	1962.59	0.1281582	0.2177985
218.415	1983.914	0.135938	0.2201649
234.557	1989.95	0.1459845	0.2208348
249.238	2003.6	0.1551217	0.2223496

6.4. Determination of the Target Displacement

The methodology of determining the nonlinear spectrum and top displacement is stated in the 3.7.2. In the TEC2007, the equal displacement rule is valid for the structural systems which have a first mode period calculated with the cracked section stiffness greater than the T_B value. The equal displacement rule is used to determine the top displacement of the structural system in both of two earthquake directions. In the Figure 6.1 (for direction X) and in Figure 6.2 (for direction Y) spectral acceleration-spectral displacement diagrams for the ground shaking having a 2% and 10% probabilities to be exceeded in 50 years are shown together with the modal capacity diagrams. In these graphs the displacements point out the elastic spectral displacement, S_{de1}, values for two ground motion demand levels. And according to the equal displacement rule these values are also the inelastic spectral displacements, S_{di1}, for these two target performance levels.

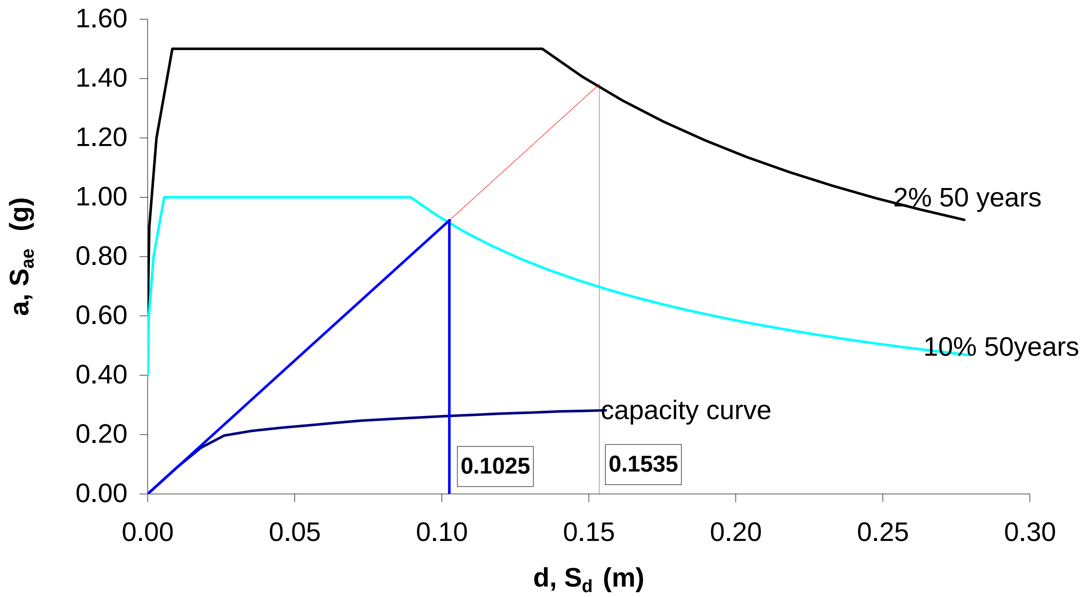

Figure 6.1. Demand and capacity diagrams for the X direction

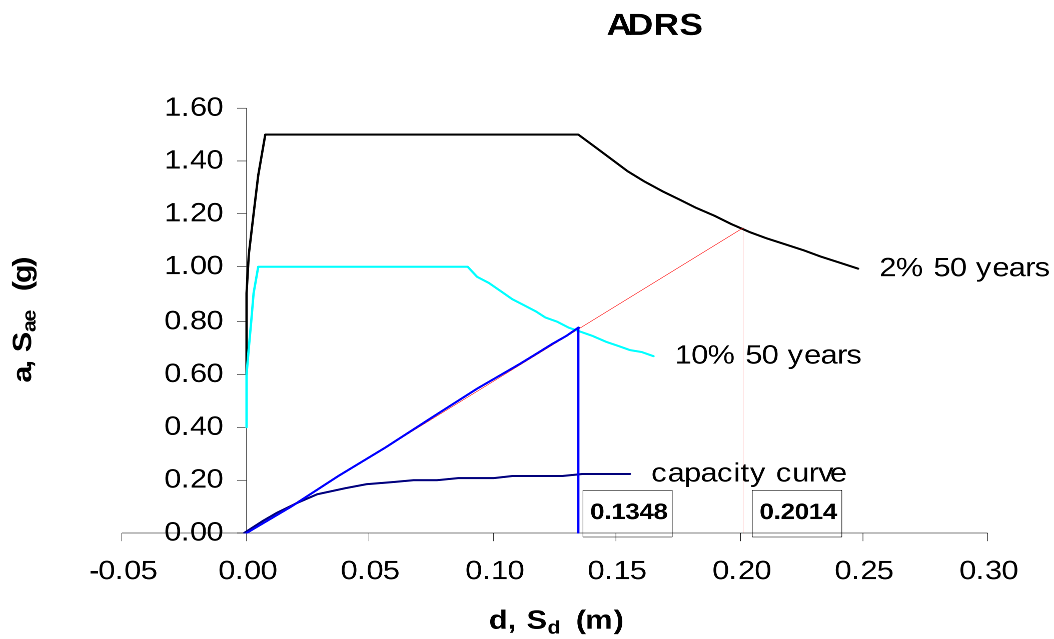

Figure 6.2. Demand and capacity diagrams for the Y direction

Spectral displacements and the target displacement under the design earthquake condition for earthquake direction X,

$$S_{di1} = S_{de1} = 0.0125\ m$$

$$u_{xN1} = d_1 \times (\ \Phi_{xN1} \times\ \Gamma_{x1}\ ) = 0.0125 \times 1 \times 1.604 = 0.164\ m$$

Spectral displacements and the target displacement under the biggest earthquake condition for earthquake direction X,

$$S_{di1} = S_{de1} = 0.1348\ m$$

$$u_{xN1} = d_1 \times (\ \Phi_{xN1} \times\ \Gamma_{x1}\ ) = 0.1348 \times 1 \times 1.604 = 0.246\ m$$

Spectral displacements and the target displacement under the design earthquake condition for earthquake direction Y,

$$S_{di1} = S_{de1} = 0.1535\ m$$

$$u_{yN1} = d_1\ \times (\ \Phi_{yN1}\ \times \Gamma_{y1}\ ) = 0.1535 \times 1 \times 1.607 = 0.247\ m$$

Spectral displacements and the target displacement under the biggest earthquake condition for earthquake direction Y,

$$S_{di1} = S_{de1} = 0.2014\ m$$

$$u_{yN1} = d_1\ (\ \Phi_{yN1}\ \Gamma_{y1}\ ) = 0.2014 \times 1 \times 1.607 = 0.324\ m$$

6.5. Specifying the Member Damage Regions

The structural system model is pushed to the calculated top displacement values under different target performance levels (2% of 50 years and 10% of 50 years) for two earthquake directions. The sections plastified during this operation make specific amounts of plastic rotations. These plastic rotations arise because of the deformations of the concrete and reinforcement in the section. The member damage strain capacities for different member damage levels are stated in the 3.12.

As it is defined in 6.2 of this chapter; by the help of the reference values and the limit values for different member damage levels, the deformations and the related member damage regions can be directly obtained from the SAP2000 program. Three examples for the representation of member damage regions on the structural system model can be seen in Figure 6.3, Figure 6.4 and Figure 6.5. In these figures green means minimum damage region, yellow means visible damage region, red means significant damage region and the purple means collapse region.

For the evaluation of columns, three closed diagrams are drawn which represents the MN,GV and GÇ member performance levels; the vertical axes means the axial load on the member at the target displacement and the horizontal axes means the total curvature of the member. These diagrams are drawn for each different type of section and the damage region of the member is decided according to its location on the diagram. Two examples about how to obtain the member damage regions from these diagrams can be seen in Figure 6.6 and in Figure 6.7. To calculate the total curvature; the plastic rotations are taken from the SAP2000 program then they are divided by the plastic hinge length to find plastic curvatures, the yield curvature values which can be seen in Table 6.7, are added to plastic curvature values.

Table 6.7. Plastic curvature values for columns in X and Y directions

Section	Yield Curvature X (1/m) (10^{-3})	Yield Curvature Y (1/m) (10^{-3})
C50x50	4.580	4.580
C30x40	7.785	5.692
C40x45	5.619	5.045
C46x60	5.671	3.616

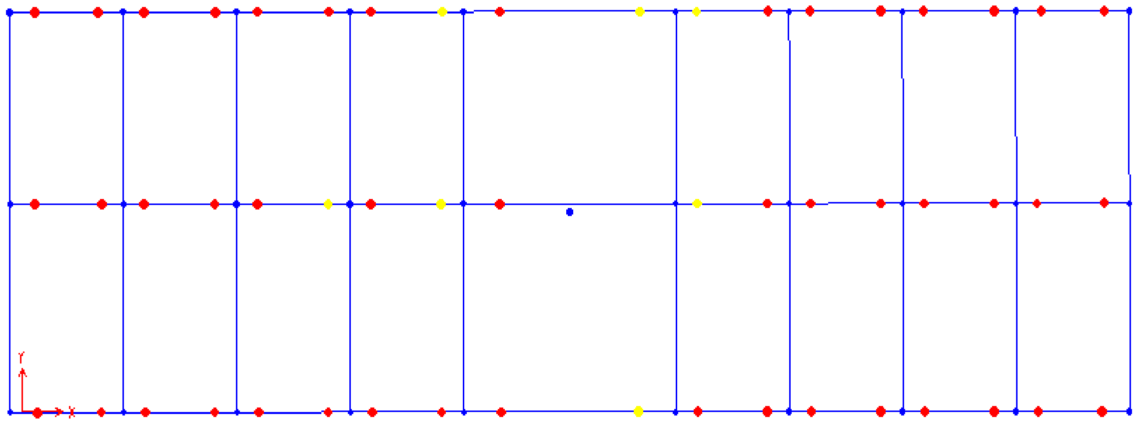

Figure 6.3. The beam damage regions for the biggest earthquake in X direction (Storey 1)

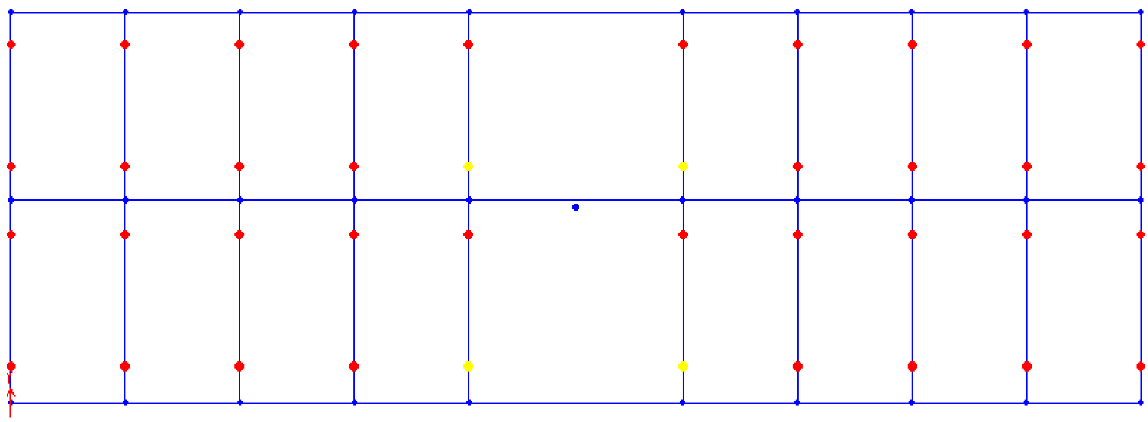

Figure 6.4. The beam damage regions for the biggest earthquake in Y direction (Storey 1)

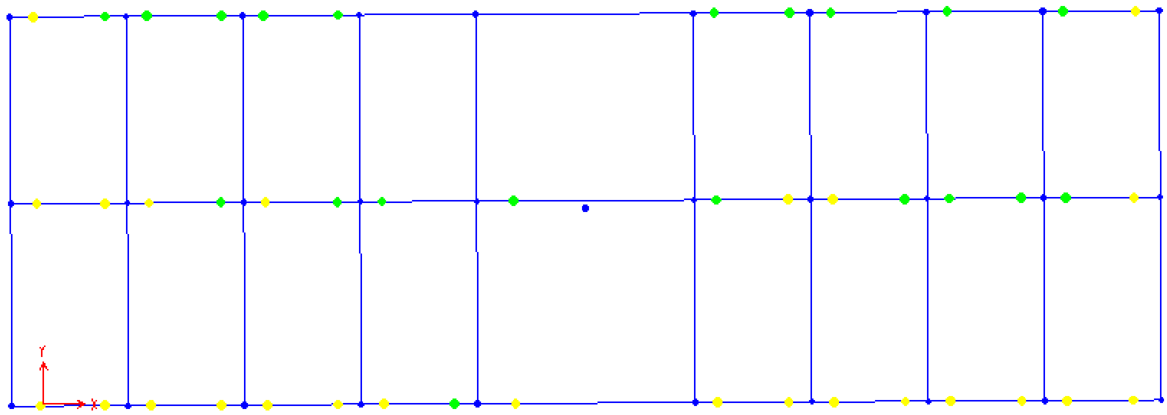

Figure 6.5. The beam damage regions for design earthquake in X direction (Storey2)

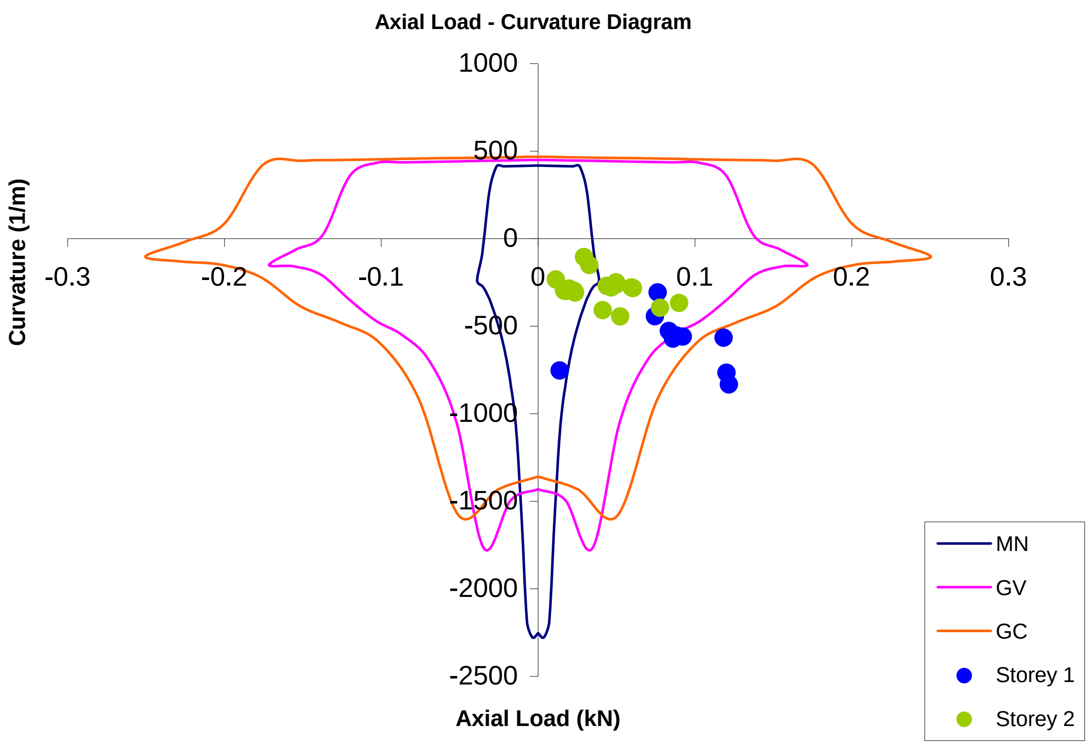

Figure 6.6. Member damage regions of C40x45 for the biggest earthquake (X)

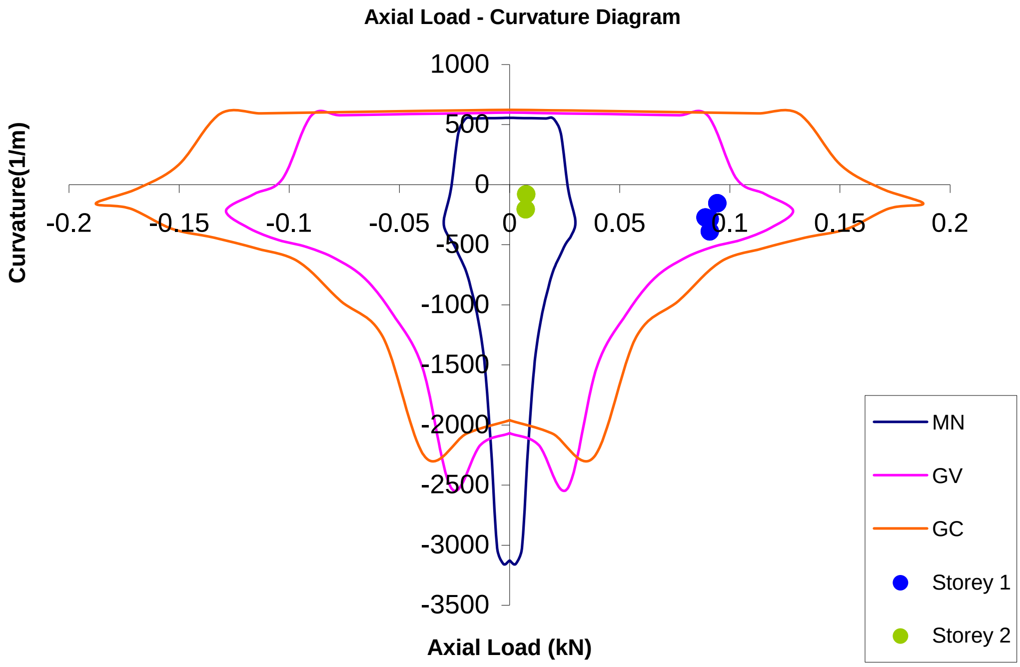

Figure 6.7. Member damage regions of C50x50 for design earthquake (Y)

6.6. Shear Strength Capacity and Column-Beam Joint Check

As it is stated in the 3.13, the shear strength capacities; of the reinforced concrete members except the beam-column joints will be determined in proportion to TS-500 according to the Equation (6.8) and Equation (6.9). The structural members which have a shear resistance smaller than the shear demand will be defined as brittle [22].

$$V_r = 0.8 \times V_{cr} + V_w \tag{6.8}$$

$$V_r = V_r = 0.8 \times 0.65 \times f_{ctm} \times b_w \times d + A_{sw} \times f_{ywm} \times d / s \tag{6.9}$$

Shear force in beam-column joints along the earthquake direction considered shall be calculated by Equation (6.10). For V_{kol} in the equation the shear demand is used at the last step of nonlinear analysis as it is mentioned in 3.13. And this shear force shall not exceed the values given in Equation (6.11) and Equation (6.12) to be able to say that the member is a ductile.

$$V_e = 1.25 f_{ym}(A_{s1} + A_{s2}) - V_{col} \tag{6.10}$$

In confined joints; $V_e = 0.60.b_j.h.f_{cm}$ (6.11)

In unconfined joints; $V_e = 0.45.b_j.h.f_{cm}$ (6.12)

6.7. Evaluating the Performance Levels of the Building

The structural system is a school building and it has to provide the *Immediate Occupancy (HK)* performance level for the design earthquake (10% 50 years) target performance condition. Also it has to provide the *Life Safety (CG)* performance level for the biggest ground shaking (%2 50 years) condition. And controlling the X and Y directions are sufficient to evaluate the performance level because of the symmetry.

6.7.1. Design Earthquake Performance Level (10% of 50 years) for X Direction

Table 6.8. Design earthquake performance level (10% of 50 years) for X direction

Section	**Story**	**MDR**	**%**	**VDR**	**%**	**SDR**	**%**	**CR**	**%**
B90x40	1	-	-	9/9	100	-	-	-	-
	2	3/9	33	6/9	67	-	-	-	-
B70x40	1	-	-	18/18	100	-	-	-	-
	2	4/18	22	11/18	61	-	-	-	-
C50x50	1	-	-	10/10	100	-	-	-	-
	2	2/10	20	1/10	10	-	-	-	-
C40x45	1	-	-	8/10	80	2/10	20	-	-
	2	-	-	10/10	100	-	-	-	-
C40x60	1	-	-	10/10	100	-	-	-	-
C30x40	2	-	-	10/10	100	-	-	-	-
	3	9/20	45	-	-	-	-	-	-
Beams	**1**	**-**	**-**	**27/27**	**100**	**-**	**-**	**-**	**-**
	2	**7/27**	**26**	**17/27**	**63**	**-**	**-**	**-**	**-**
Columns	**1**	**-**	**-**	**28/30**	**93**	**2/30**	**7**	**-**	**-**
	2	**2/30**	**7**	**21/30**	**70**	**-**	**-**	**-**	**-**
	3	**9/20**	**45**	**-**	**-**	**-**	**-**	**-**	**-**

According to the 3.4 more than 10% of the beams are in visible damage region and the other elements are not in minimum damage region so the building under the design earthquake target performance level “do not” satisfy the *Immediate Occupancy* building performance level.

But it satisfies the statements in 2.4.2 for the *Life Safety* performance level.

- Not more than 30% of beams are in significant damage region.
- The shear force carried by columns in significant damage region is less than 20 percent of the total shear force carried by columns in that storey (8% in first storey)
- The ratio of the shear force carried by the columns whose minimum damage limits are exceeded in both upper and lower end sections at any story to the shear force carried by all columns at the related story ratio is less than 30%. (12% at second storey)
- All other structural members are in minimum damage region or visible damage region.

As it can be seen in Table 6.9, the shear strength capacities of the members are calculated and there is no member which can be named as brittle, except one (B90x40) beam and one column (C40x45).

Table 6.9. Shear strength capacity check (10% 50 years) for X direction

	B70x40	B90X40	C50X50	C40X45	C40X60	C30X40
$b_w(mm)$	700	900	500	450	600	400
$d(mm)$	400	400	500	400	400	300
$A_{sw}(mm^2)$	50	50	50	50	50	50
$s(mm)$	250	250	250	250	250	250
$V(kN)$	156	191	152	113	139	78
$V_{\max}(kN)$	83	144	102	129	110	33
$V_{\min}(kN)$	-115	**-192**	-	-	-	-

The control of the beam column joints at some critical connections is accepted as adequate to say that the shear force at the last step of analysis is under the limit shear values at joints and these evaluations can be seen in Table 6.10. At the grid lines H2 and I2 at stories 1 and 2 the shear force is above the limit. At the grid lines H4 and I4 at story 2 the connection is brittle.

Table 6.10. Shear limit check at beam column joints (X)

Joint	Joint Type	Storey	$V_{kol}(kN)$	$V_e(kN)$	$V_{e,\max}(kN)$
H2&I2	Confined	1	129	1448	1080
		2	93	1484	1080
H4&I4	Unconfined	1	100	615	1080
		2	33	682	540
H1&I1	Unconfined	1	101	614	1125
G2&J2	Confined	1	67	562	810
		2	48	581	810
G4&J4	Unconfined	1	79	333	1080
		2	52	360	540

6.7.2. The Biggest Earthquake Performance Level (2% 50 years) for X Direction

Table 6.11. The biggest earthquake performance level (2% 50 years) for X direction

Section	**Storey**	**MDR**	**%**	**VDR**	**%**	**SDR**	**%**	**CR**	**%**
B90x40	1	-	-	-	-	9/9	100	-	-
	2	-	-	9/9	100	-	-	-	-
B70x40	1	-	-	-	-	18/18	100	-	-
	2	2/18	11	9/18	50	7/18	39	-	-
C50x50	1	-	-	10/10	100	-	-	-	-
	2	1/10	10	2/10	20	-	-	-	-
C40x45	1	-	-	3/10	30	4/10	40	3/10	30
	2	-	-	10/10	100	-	-	-	-
C40x60	1	-	-	10/10	100	-	-	-	-
C30x40	2	-	-	8/10	80	1/10	10	1/10	10
	3	8/20	40	2/20	10	-	-	-	-
Beams	**1**	**-**	**-**	**-**	**-**	**27/27**	**100**	**-**	**-**
	2	**2/27**	**7**	**18/27**	**67**	**7/27**	**26**	**-**	**-**
Columns	**1**	**-**	**-**	**23/30**	**77**	**4/30**	**13**	**3/30**	**10**
	2	**1/30**	**0.3**	**20/30**	**67**	**1/30**	**0.3**	**1/30**	**0.3**
	3	**8/20**	**40**	**2/20**	**10**	**-**	**-**	**-**	**-**

According to the 3.4.2, not more than 20% of the beams are in collapse region but the other elements are not in minimum damage, visible damage and significant damage regions because there are some columns in the collapse region at first and second storey; so the building under the design earthquake target performance level "do not" satisfy the *Collapse Prevention* level and the building is in the *Collapse* level, although the statement that it has to provide *Life Safety* performance level at this target performance.

As it can be seen in Table 6.12, the shear strength capacities of the members are calculated. 8/20 of C40x45 columns and one of the B90x40 beams at the longest span are brittle.

Table 6.12. Shear strength capacity check (2% 50 years) for X direction

	B70x40	B90X40	C50X50	C40X45	C40X60	C30X40
$b_w(mm)$	700	900	500	450	600	400
$d(mm)$	400	400	500	400	400	300
$A_{sw}(mm^2)$	50	50	50	50	50	50
$s(mm)$	250	250	250	250	250	250
$V(kN)$	156	191	152	113	139	78
$V_{\max}(kN)$	83	144	107	**-149**	120	35
$V_{\min}(kN)$	-115	**-203**	-	-	-	-

The control of the beam column joints at some critical connections is done which can be seen in Table 6.13. For the joints near the maximum span, H2 and I2 at stories 1 and 2 and the joints H4 and I4 at story 2 the shear forces are above the limit.

Table 6.13. Shear limit check at beam column joints (X)

Joint	Joint Type	Storey	$V_{kol}(kN)$	$V_e(kN)$	$V_{e,\max}(kN)$
H2&I2	Confined	1	150	1427	1080
		2	88	1489	1080
H4&I4	Unconfined	1	107	608	1080
		2	35	680	540
H1&I1	Unconfined	1	103	612	1125
G2&J2	Confined	1	99	530	810
		2	65	564	810
G4&J4	Unconfined	1	84	328	1080
		2	29	383	540

6.7.3. Design Earthquake Performance Level (10% 50 years) for Y Direction

Table 6.14. Design earthquake performance level (10% 50 years) for Y direction

Section	Storey	MDR	%	VDR	%	SDR	%	CR	%
B40x40	1	-	-	20/20	100	-	-	-	-
	2	-	-	20/20	100	-	-	-	-
C50x50	1	-	-	10/10	100	-	-	-	-
	2	-	-	-	-	-	-	-	-
C40x45	1	-	-	8/10	80	2/10	20	-	-
	2	10/10	100	-	-	-	-	-	-
C40x60	1	-	-	10/10	100	-	-	-	-
C30x40	2	10/10	100	-	-	-	-	-	-
	3	-	-	-	-	-	-	-	-
Beams	**1**	**-**	**-**	**20/20**	**100**	**-**	**-**	**-**	**-**
	2	**-**	**-**	**20/20**	**100**	**-**	**-**	**-**	**-**
Columns	**1**	**-**	**-**	**28/30**	**93**	**2/30**	**7**	**-**	**-**
	2	**20/30**	**67**	**-**	**-**	**-**	**-**	**-**	**-**
	3	**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**

All of the beams are in visible damage region and the other elements are not in minimum damage region so the building under the design earthquake target performance level "did not" satisfy the *Immediate Occupancy* building performance level. But it satisfies the conditions for the *Life Safety* performance level.

- There are not any beams in significant damage region.
- The shear force carried by columns in significant damage region is less than 20 percent of the total shear force carried by columns in that storey (7% in first storey)
- All other structural members are in minimum damage region or in visible damage region.

As it can be seen in Table 6.15, the shear strength capacities of all the members are smaller than the limit shear value so all of them can be regarded as ductile.

Table 6.15. Shear strength capacity check (10% 50 years) for Y direction

	B40X40	C50X50	C40X45	C40X60	C30X40
$b_w(mm)$	400	500	450	600	400
$d(mm)$	400	500	400	400	300
$A_{sw}(mm^2)$	50	50	50	50	50
$s(mm)$	250	250	250	250	250
$V(kN)$	104	152	113	139	78
$V_{\max}(kN)$	84	67	104	91	42
$V_{\min}(kN)$	-57	-	-	-	-

The shear limits are checked for three critical joints and it can be said that all the joints behave in a ductile manner as it is illustrated in Table 6.16.

Table 6.16. Shear limit check at beam column joints (Y)

Joint	Joint Type	Storey	$V_{kol}(kN)$	$V_e(kN)$	$V_{e,\max}(kN)$
H2&I2	Confined	1	79	550	1080
		2	70	559	1080
H4&I4	Unconfined	1	80	290	1080
		2	42	328	540
G1&J1	Unconfined	1	43	207	1125

6.7.4. The Biggest Earthquake Performance Level (2% 50 years) for Y Direction

Table 6.17. The biggest earthquake performance level (2% 50 years) for Y direction

Section	Storey	Minimum Damage	%	Visible Damage	%	Significant Damage	%	Collapse Region	%
B40x40	1	-	-	-	-	20/20	100	-	-
	2	-	-	-	-	20/20	100	-	-
C50x50	1	-	-	10/10	100	-	-	-	-
	2	4/10	40	-	-	-	-	-	-
C40x45	1	-	-	-	-	4/10	40	6/10	60
	2	10/10	100	-	-	-	-	-	-
C40x60	1	-	-	10/10	100	-	-	-	-
C30x40	2	10/10	100	-	-	-	-	-	-
	3	-	-	-	-	-	-	-	-
Beams	**1**	**-**	**-**	**20/20**	**100**	**-**	**-**	**-**	**-**
	2	**-**	**-**	**20/20**	**100**	**-**	**-**	**-**	**-**
Columns	**1**	**-**	**-**	**28/30**	**93**	**2/30**	**7**	**-**	**-**
	2	**20/30**	**67**	**-**	**-**	**-**	**-**	**-**	**-**
	3	**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**

Because there are some columns in the collapse region at first storey, the building under the design earthquake target performance level “do not” satisfy the *Collapse Prevention* level and the building is in the *Collapse* level, although the statement that it has to provide *Life Safety* performance level at this target performance.

The shear strength capacities of all the members except four columns (C40X45) are below the limit shear value, as it is shown in Table 6.18.

Table 6.18. Shear strength capacity check (2% 50 years) for Y direction

	B40X40	C50X50	C40X45	C40X60	C30X40
$b_w(mm)$	400	500	450	600	400
$d(mm)$	400	500	400	400	300
$A_{sw}(mm^2)$	50	50	50	50	50
$s(mm)$	250	250	250	250	250
$V(kN)$	104	152	113	139	78
$V_{max}(kN)$	84.028	63	156	107	42
$V_{min}(kN)$	-57.059	-	-	-	-

The shear limits are checked for three critical joints and it can be said that all the joints behave in a ductile manner as it can be seen in Table 5.19.

Table 6.19. Shear limit check at beam column joints (Y)

Joint	Joint Type	Storey	$V_{kol}(kN)$	$V_e(kN)$	$V_{e,\max}(kN)$
H2&I2	Confined	1	85	544	1080
		2	77	552	1080
H4&I4	Unconfined	1	78	292	1080
		2	48	322	540
G1&J1	Unconfined	1	49	201	1125

Regarding this section of this study the building can not satisfy the *Immediate Occupancy* performance level for design earthquake (10% 50 years) and also can not satisfy the *Life Safety* performance level for the biggest earthquake (2% 50 years).

According to TEC 2007; the retrofit strategies can eliminate the defaults that can cause earthquake damages with adding new members to increase the seismic capacity, mass reduction of the structural system, improving the seismic capacities of the existing structural members and providing continuity of strengthening of the system.

After deciding that the building is not capable to resist the earthquake ground motion, two retrofit strategies are chosen to strengthen the building. These technical strategies are addition of shear walls and steel braces. The architectural reasons have to be taken into account not to impair the use of the building. The components added to the building are not at the exactly same locations so this study just gives some opinions to compare the structural effectiveness of the strategies. There is not any section in the current code, TEC 2007, that exactly describes the application of retrofit strategies but the following sections will give an idea to estimate the effectiveness of the strategies to provide the desired building performance levels under different target performance demands with the incremental equivalent seismic load method.

7. RETROFIT THE BUILDING BY SHEAR WALLS

Retrofit the structural system means adding new structural members to system, strengthening the joint regions and decreasing the total mass of the structural system; to provide the increase of the resistance and deformation capacities and also to provide the continuity of the distribution of the internal forces for the structural system.

One of the most common ways to improve the seismic behavior of a reinforced concrete structure is addition of shear walls. The structural systems which have not got sufficient lateral rigidity and resistance will be retrofitted by cast in place shear walls. According to the TEC 2007 section 7.10.5.1, the shear walls added *in the frame plane* must be continuous from the foundation to the top elevation of shear wall. Shear walls will be fixed to the in plane frames by anchorage bars to make structural walls work together with the existing frames, also these anchorage bars have to got the capacity to resist the shear loads that will occur at the surface between the walls and the frame. In the case that the columns exist nearby the added shear walls, these columns can be used as end zones by enlarging columns or hiding them into the shear walls if there is a necessity. The foundation of the shear wall shall be sized so as to confidently transfer the internal forces to the foundation and the precautions must be taken to make these two foundation systems work together.

Shear walls can be added to a structure in a way that; not to cause irregularities, such as torsional irregularity, weak and soft storey irregularities. For this reason the addition of shear walls requires sophisticated engineering judgments.

7.1. Location of Shear Walls and Renewed Dynamic Properties

The location of the shear walls is decided after lots of trials and regarding the engineering sense. As a good trial, the shear walls located at the first and second stories of the building but this condition make the displacements for the first mode of the system like that, first and second stories depart to the positive direction while the third storey displaces negatively. The reason for such a mode shape is that, the mass and the stiffness of first and second stories are so much bigger than the top storey. And the problem is that, such a mode shape can not be analyzed with the incremental equivalent seismic load method by the SAP2000 according to the accepted material model and the accepted idealized plastic hinge model without strain hardening. So as a final, the shear walls which are modeled as working with perfect continuity are located form foundation to the top storey, as it can be seen in Figure 7.1. The locations for the shear walls as a plan view for the second storey can be seen in Figure 7.2. The design of the shear walls is done according to the current design code and the optimum design for them decided after several trials. The same sections and same reinforcement quantity and arrangement, as it is shown in Figure 7.3 and Figure 7.4, are used for the shear walls along all three stories.

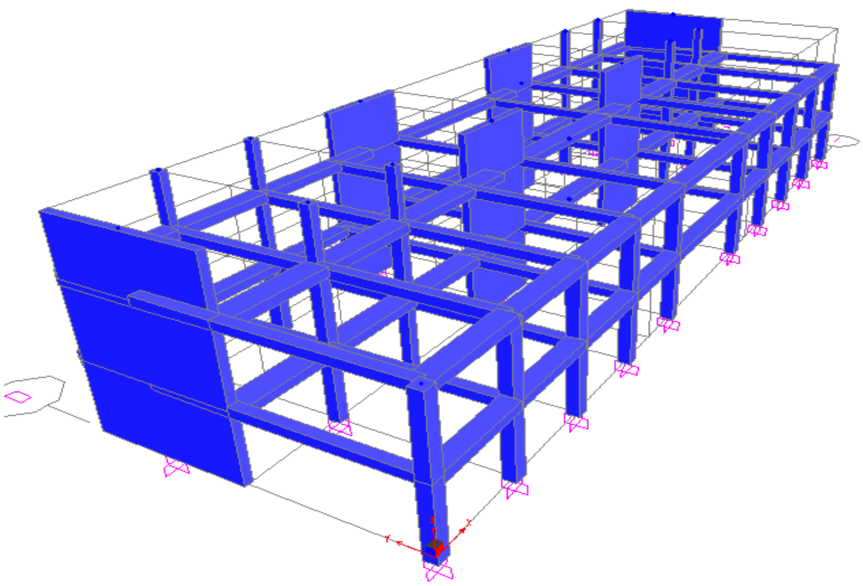

Figure 7.1. Retrofit the building by shear walls

The columns exist nearby the added shear walls; can be used as end zones for these shear walls. The columns nearby the shear walls are removed from the structural system model because of the fact that these columns make extra plastic rotations which will not exist in the real case conditions as a hidden end zone for the shear walls. In the structural system model the shear walls are modeled as frame elements which are located at the center of mass of the related shear walls and these frame elements are connected to the nearby beams with infinitely rigid structural members.

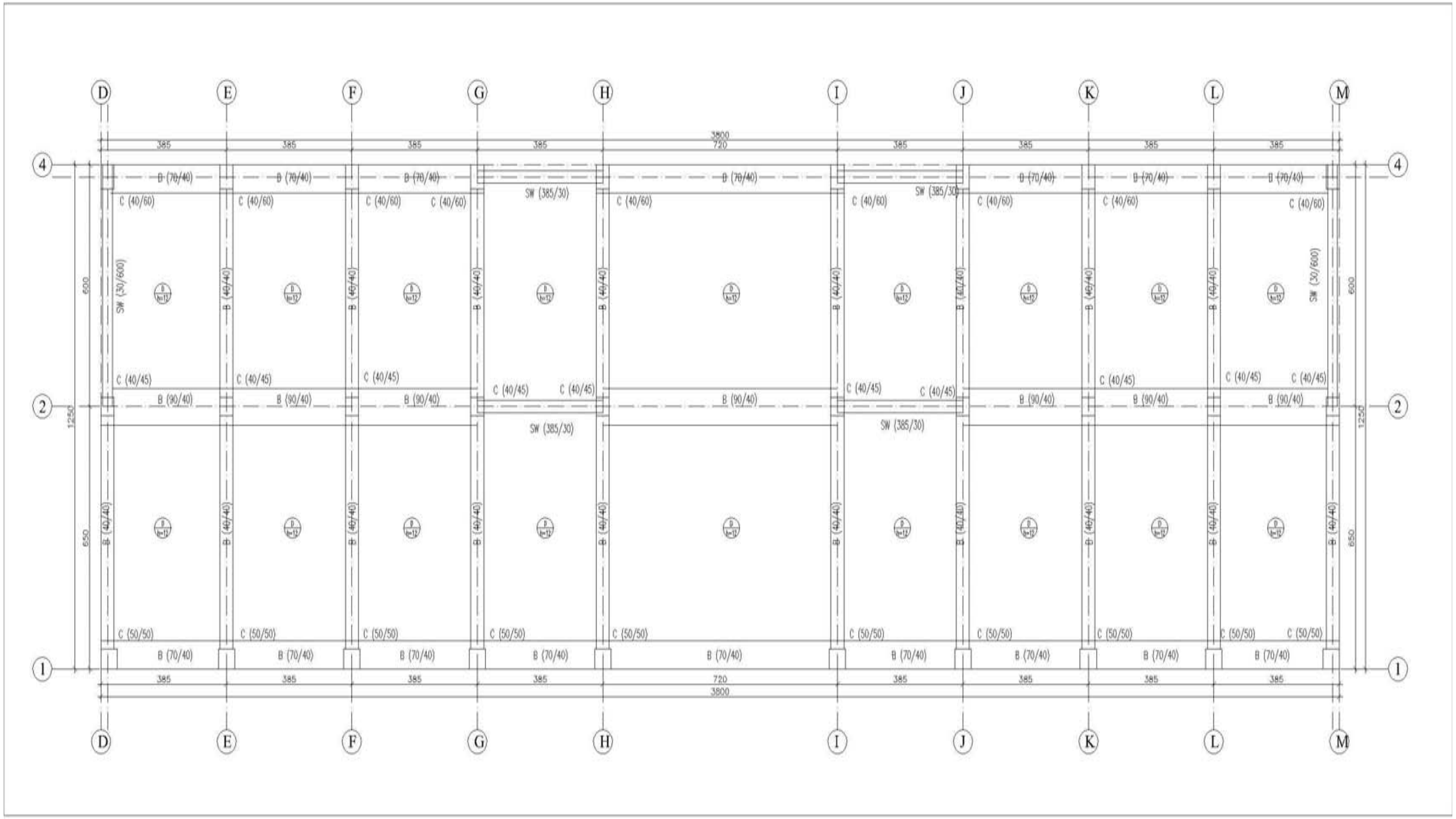

Figure 7.2. Location of shear walls for the first storey

For the section stiffness of the cracked sections, the assigned stiffness values to the columns and beams are used during the evaluation of retrofit analysis because of the fact that the vertical loads formed the section cracks and decreased the section stiffness before the building is retrofitted by shear walls. And these section cracks for the shear walls are accepted as; they are formed because of the self weight loads and a smaller part of the vertical dead and live loads. So the cracked section stiffness value for the shear walls was taken is 0.4 with a proper approximation.

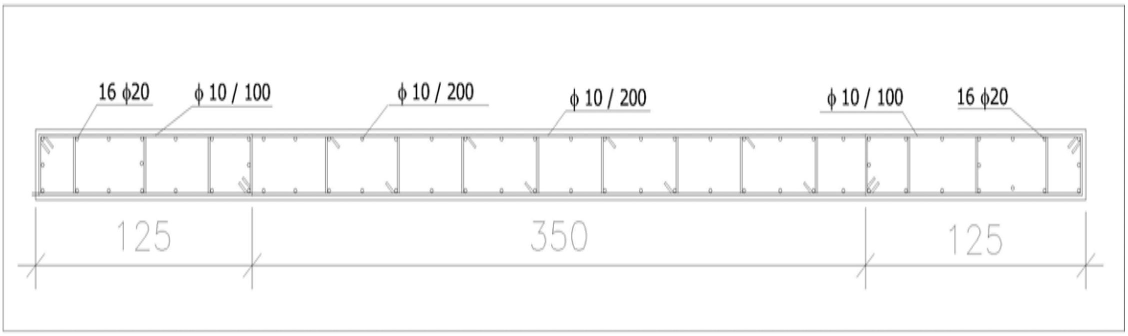

Figure 7.3. Reinforcement detail for P30x600

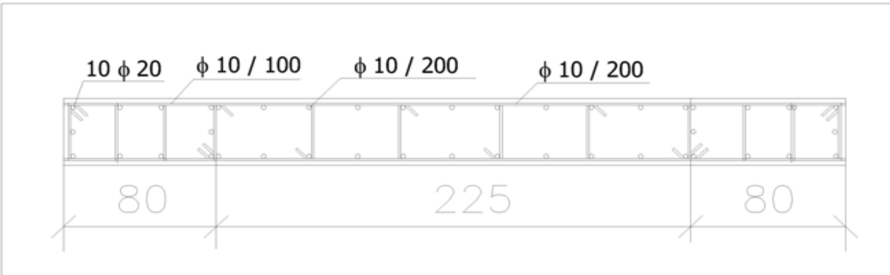

Figure 7.4. Reinforcement detail for P385x30

There is an increase of weights for all of the three stories because of the self weights of shear walls as it is shown in Table 7.1

Table 7.1. Total storey weights with addition of shear walls

Storey	$W_i(kN)$	$W_{i,with\ shear\ walls}(kN)$
3	376.3	605.8
2	5449.8	5965
1	5442.4	5971

The effective mass to total building mass corresponding to the first natural vibration mode in X direction is 0.781, in Y direction it is 0.796 and both of them are above the value 0.70.

$$T_{1X} = 0.227 \text{ sec} \qquad\qquad T_{1Y} = 0.185 \text{ sec}$$

7.2. Definition of Plastic Hinges

The same idealizations and definitions for the plastic hinges of beams and columns, which are described in 6.2, are valid for the new structural system model. For the shear walls which have symmetric reinforcement arrangements, the yield surfaces are defined with three yield curves for the angles 0, 45 and 90. As it is stated in TEC 2007; the maximum compression strain for the concrete is taken as 0.003, the maximum strain for the reinforcement is taken as 0.01. Axial loads and the corresponding curvature values of these yield surfaces are calculated by the section analysis program XTRACT.

7.3. Derivation of Capacity Curve

To start the pushover analysis, the model has to be pushed to the required displacement of the related seismic demand with an accepted lateral load distribution. This lateral load distribution is related with the displacements of the first mode shape and the storey masses for both of the earthquake directions X and Y.

Lateral Load distribution for the earthquake direction X,

$$\begin{Bmatrix} 0.0116 \\ 0.0350 \\ 0.0532 \end{Bmatrix} \begin{Bmatrix} 608.70 \\ 608.11 \\ 61.72 \end{Bmatrix} = \begin{Bmatrix} 7.061 \\ 21.283 \\ 3.283 \end{Bmatrix} \rightarrow \begin{Bmatrix} 1 \\ 3.014 \\ 0.465 \end{Bmatrix} \begin{pmatrix} 1^{st}\ storey \\ 2^{nd}\ storey \\ 3^{rd}\ storey \end{pmatrix}$$

Lateral Load distribution for the earthquake direction Y,

$$\begin{Bmatrix} 0.0123 \\ 0.0349 \\ 0.0520 \end{Bmatrix} \begin{Bmatrix} 608.70 \\ 608.11 \\ 61.72 \end{Bmatrix} = \begin{Bmatrix} 7.487 \\ 21.223 \\ 3.209 \end{Bmatrix} \rightarrow \begin{Bmatrix} 1 \\ 2.835 \\ 0.429 \end{Bmatrix} \begin{pmatrix} 1^{st}\, storey \\ 2^{nd}\, storey \\ 3^{rd}\, storey \end{pmatrix}$$

To convert the "top displacement – base shear" diagram to "modal displacement – modal acceleration" diagram the same methodology which is described in the 6.3 and the Equations 6.2, 6.3, 6.4, 6.5, 6.6 and 6.7 are used. And the results for the system with shear walls can be seen below.

For the earthquake direction X at the first step of analysis,

$$M^*_{x1} = \begin{bmatrix} 0.218 & 0.658 & 1 \end{bmatrix} \begin{bmatrix} 608.70 & 0 & 0 \\ 0 & 608.11 & 0 \\ 0 & 0 & 61.72 \end{bmatrix} \begin{bmatrix} 0.218 \\ 0.658 \\ 1 \end{bmatrix} = 353.8\ kNs^2/m$$

$$L^*_{x1} = \begin{bmatrix} 0.218 & 0.658 & 1 \end{bmatrix} \begin{bmatrix} 608.70 & 0 & 0 \\ 0 & 608.11 & 0 \\ 0 & 0 & 61.72 \end{bmatrix} \begin{bmatrix} 1 \\ 1 \\ 1 \end{bmatrix} = 594.5\ kNs^2/m$$

$$\Gamma_{x1} = L^*_{x1} / M^*_{x1} = 594.5/353.8 = 1.680$$

$$M_{x1} = L^{*2}_{x1} / M^*_{x1} = 594.5^2/353.8 = 998.81\ \text{kNs}^2/\text{m}$$

For the earthquake direction Y at the first step of analysis,

$$M^*_{y1} = \begin{bmatrix} 0.236 & 0.671 & 1 \end{bmatrix} \begin{bmatrix} 608.70 & 0 & 0 \\ 0 & 608.11 & 0 \\ 0 & 0 & 61.72 \end{bmatrix} \begin{bmatrix} 0.236 \\ 0.671 \\ 1 \end{bmatrix} = 369.7\ kNs^2/m$$

$$L^*_{y1} = \begin{bmatrix} 0.236 & 0.671 & 1 \end{bmatrix} \begin{bmatrix} 608.70 & 0 & 0 \\ 0 & 608.11 & 0 \\ 0 & 0 & 61.72 \end{bmatrix} \begin{bmatrix} 1 \\ 1 \\ 1 \end{bmatrix} = 613.81\ kNs^2/m$$

$$\Gamma_{y1} = L^*_{y1} / M^*_{y1} = 613.8/369.7 = 1.661$$

$$M_{y1} = L^{*2}_{y1} / M^*_{y1} = 613.8^2/369.7 = 1019.17\ \text{kNs}^2/\text{m}$$

The modal capacity curves are formed according to above calculations. The values for these capacity curves for earthquake direction X is given in Table 7.2 and in Table 7.3 for the direction Y.

Table 7.2. The values for the modal capacity curve in direction X

$u_{xN1}^{(i)}(mm)$	$V_{xN1}^{(i)}(kN)$	$d_1^{(i)}(m)$	$a_1^{(i)}(g)$
0.00	0.00	0.000	0.000
0.97	437.57	0.001	0.045
8.56	3758.86	0.005	0.384
12.83	5467.79	0.008	0.558
20.61	5788.08	0.012	0.591
31.26	6198.07	0.019	0.633
38.96	6449.51	0.023	0.658
46.61	6581.53	0.028	0.672
54.31	6654.09	0.032	0.679
62.56	6710.28	0.037	0.685
73.02	6758.22	0.043	0.690
83.52	6796.97	0.050	0.694
87.50	6838.18	0.052	0.698
92.05	6878.21	0.055	0.702
94.84	6911.47	0.056	0.705
102.50	6934.63	0.061	0.708
111.95	6953.35	0.067	0.710
120.07	6974.77	0.071	0.712
127.57	6992.38	0.076	0.714
136.84	7011.22	0.081	0.716
150.00	7034.21	0.089	0.718

Table 7.3. The values for the modal capacity curve in direction Y

$u_{yN1}^{(i)}(mm)$	$V_{yN1}^{(i)}(kN)$	$d_1^{(i)}(m)$	$a_1^{(i)}(g)$
0.00	0.00	0.000	0.000
2.41	1865.37	0.001	0.187
8.18	5926.34	0.005	0.593
10.62	7136.67	0.006	0.714
21.37	7511.62	0.013	0.751
30.19	7767.02	0.018	0.777
38.04	7983.89	0.023	0.799
45.98	8160.21	0.028	0.816
55.39	8303.37	0.033	0.830
64.74	8395.31	0.039	0.840
72.24	8455.28	0.044	0.846
85.92	8559.53	0.052	0.856
93.42	8616.88	0.056	0.862
100.92	8673.17	0.061	0.867
108.42	8729.57	0.065	0.873
122.18	8792.74	0.074	0.879
129.84	8843.18	0.078	0.884
135.09	8882.45	0.081	0.888
139.08	8918.06	0.084	0.892
147.95	8939.11	0.089	0.894
149.77	8957.88	0.090	0.896

7.4. Determination of the Target Displacement

The same methodology as in the 6.4 is used to determine the target displacement for the building with structural walls. As a difference the equal displacement rule is not valid for the new model because the periods of the first modes are smaller than the T_B value for both of the earthquake directions and the elastic spectral displacements, S_{de1}, are converted to inelastic spectral displacements, S_{di1}, with the value C_{R1} which is stated in the 3.10. In the Figure 7.5 (for direction X) an in Figure 7.6 (for direction Y) spectral acceleration-spectral displacement diagrams for the ground shaking having 2% and 10% probabilities to be exceeded in 50 years are shown together with the modal capacity diagrams.

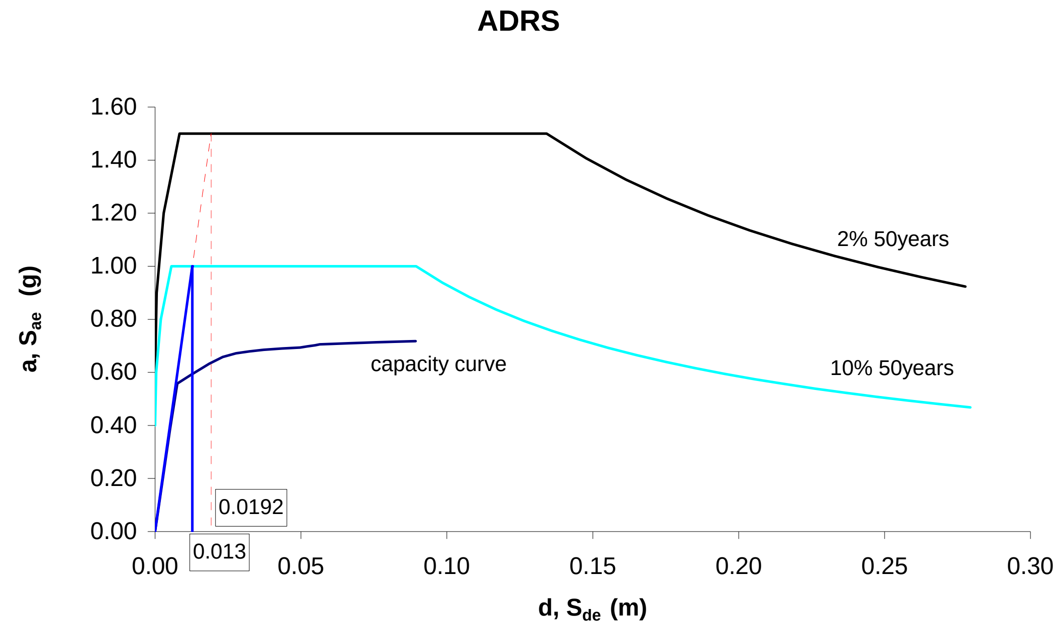

Figure 7.5. Demand and capacity diagrams for the X direction

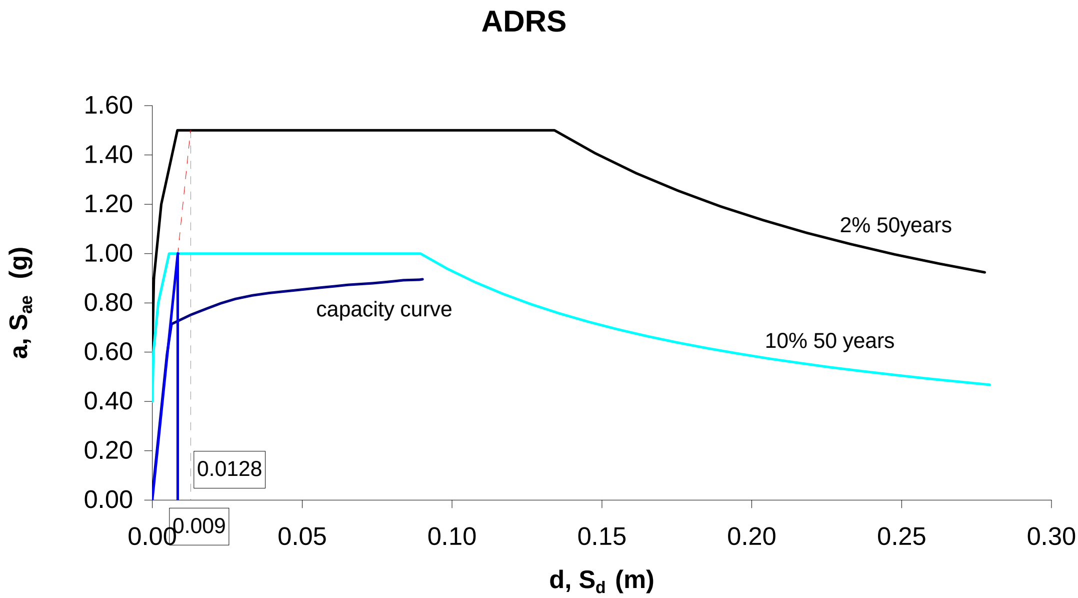

Figure 7.6. Demand and capacity diagrams for the Y direction

The spectral displacement ratios which are evaluated by consecutive iterations can be seen in Table 7.4.

Table 7.4. Iterations for the spectral displacement ratios (X 10% 50 years)

Iteration	1	2
C_{R1}	1.777	1.745
R_{y1}	1.90	1.83

Spectral displacements and the target displacement under the design earthquake condition for earthquake direction X,

$$S_{di1} = C_{R1} S_{de1} = 1.745 \times 0.0128 = 0.0223\ m$$

$$u_{xN1} = d_1\ (\ \Phi_{xN1}\ \Gamma_{x1}\ ) = 0.0223 \times 1 \times 1.680 = 0.0375\ m$$

The spectral displacement ratios which were evaluated by consecutive iterations can be seen in Table 7.5.

Table 7.5. Iterations for the spectral displacement ratios (X 2% 50 years)

Iteration	1	2	3	4	5
C_{R1}	2.048	1.991	1.947	1.904	1.890
R_{y1}	2.76	2.52	2.36	2.22	2.15

Spectral displacements and the target displacement under the biggest earthquake condition for earthquake direction X,

$$S_{di1} = C_{R1} S_{de1} = 1.890 \times 0.0192 = 0.0362\ m$$

$$u_{xN1} = d_1\ (\ \Phi_{xN1}\ \Gamma_{x1}\ ) = 0.0362 \times 1 \times 1.680 = 0.0608\ m$$

The spectral displacement ratios which were evaluated by consecutive iterations can be seen in Table 7.6.

Table 7.6. Iterations for the spectral displacement ratios (Y 10% 50 years)

Iteration	1	2	3
C_{R1}	2.185	1.688	1.647
R_{y1}	2.12	1.44	1.41

Spectral displacements and the target displacement under the design earthquake condition for earthquake direction Y,

$$S_{di1} = C_{R1} S_{de1} = 1.647 \times 0.00851 = 0.0140\ m$$

$$u_{xN1} = d_1\ (\ \Phi_{xN1}\ \Gamma_{x1}\ ) = 0.0140 \times 1 \times 1.661 = 0.0233\ m$$

The spectral displacement ratios which were evaluated by consecutive iterations can be seen in Table 7.7.

Table 7.7. Iterations for the spectral displacement ratios (Y 2% 50 years)

Iteration	1	2	3
C_{R1}	2.234	2.183	2.052
R_{y1}	2.22	2.12	1.88

Spectral displacements and the target displacement under the design earthquake condition for earthquake direction Y,

$$S_{di1} = C_{R1} S_{de1} = 2.052 \times 0.0128 = 0.0263\ m$$

$$u_{xN1} = d_1\ (\ \Phi_{xN1}\ \Gamma_{x1}\ ) = 0.0263 \times 1 \times 1.661 = 0.0436\ m$$

7.5. Specifying the Member Damage Regions

The structural system model is pushed to the calculated top displacement values in 7.4 under different target performance levels (2% 50 years and 10% 50 years) for two earthquake directions. The methodology to evaluate the member damage levels of the beams from the SAP2000 program directly by the help of the reference values and the limit values is described in 6.5. Three examples for the representation of member damage regions on the structural system model can be seen in Figure 7.7, Figure 7.8 and Figure 7.9. In these figures green means minimum damage region, yellow means visible damage region, red means significant damage region and the purple means collapse region.

For columns the same methodology is used as in 7.5 and the same closed diagrams for three member performance levels are used to evaluate the new member damage regions of the columns for the new, strengthened and more rigid structural system. The same methodology is used to determine the "axial load-total curvature" diagrams for the shear walls which are utilized for to determine the shear walls' member damage regions as it is shown in Figure 7.10 and Figure 7.11. The yield curvatures which are required to calculate total curvatures of the shear walls are $0.629\times10^{-3}\ 1/m$ for P30x600 functioned in X direction and $0.971\times10^{-3}\ 1/m$ for P385x30 functioned in Y direction. During the calculation of strain limits to obtain member damage levels for shear walls the ρ_s/ρ_{sm} ratio is taken as 1.

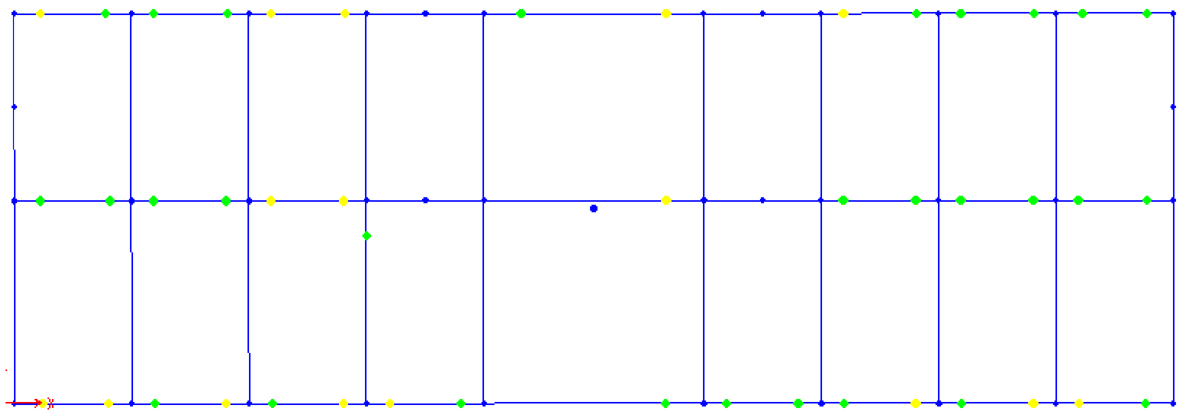

Figure 7.7. The beam damage regions for the biggest earthquake in X direction (Storey1)

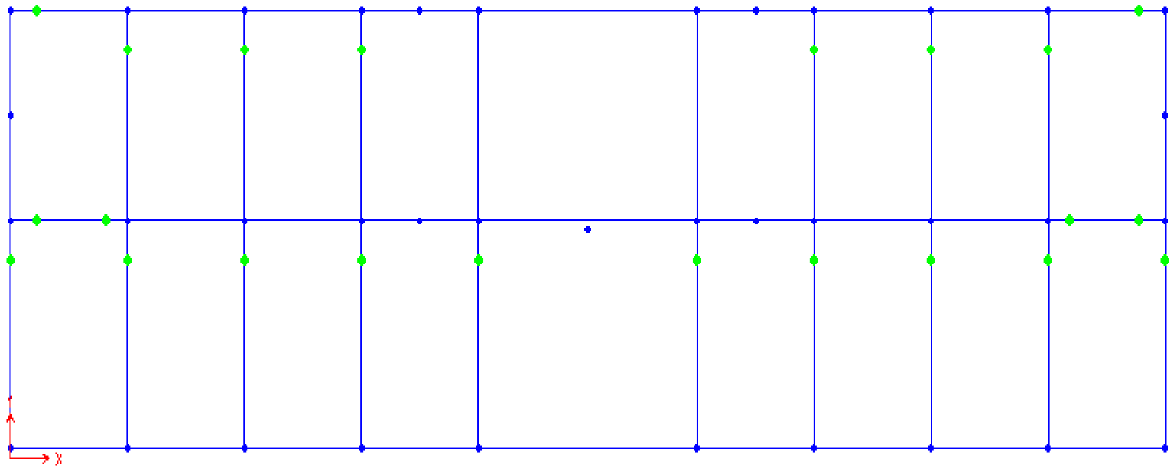

Figure 7.8. The beam damage regions for the biggest earthquake in Y direction (Storey1)

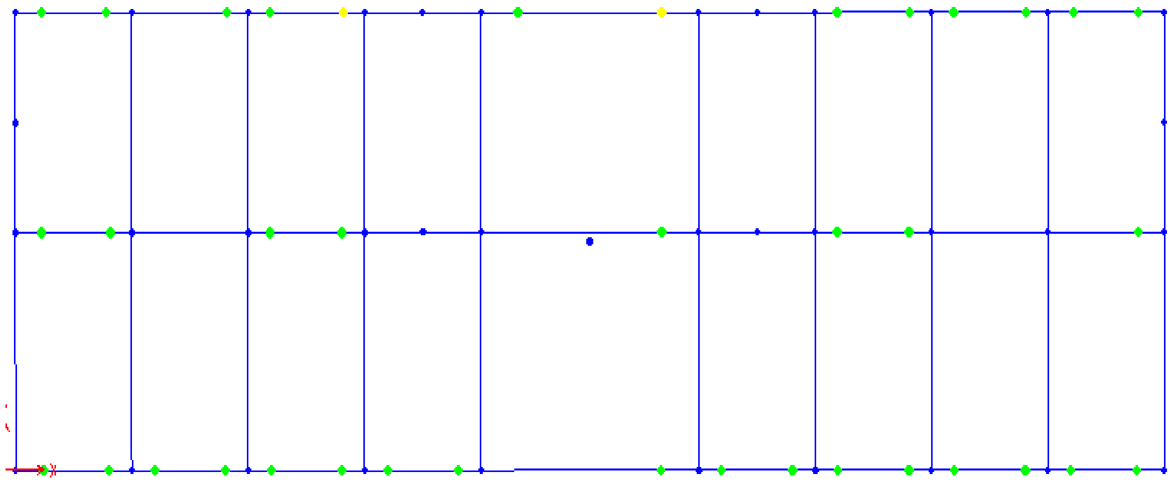

Figure 7.9. The beam damage regions for design earthquake in X direction (Storey1)

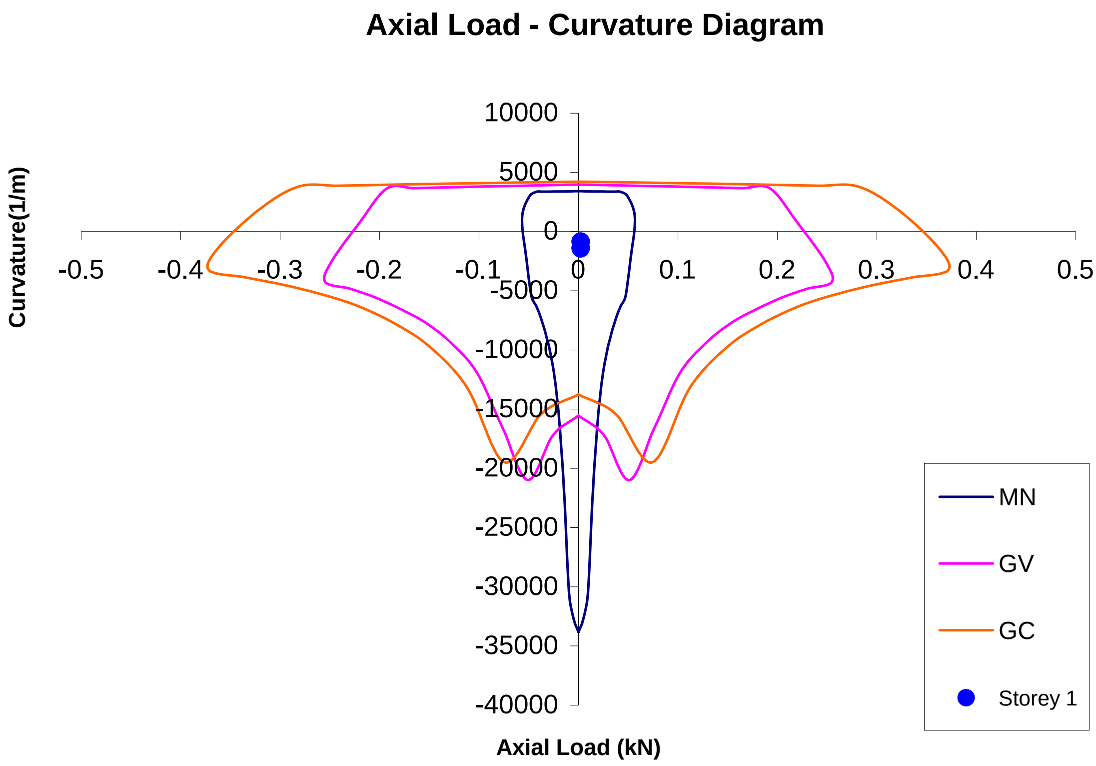

Figure 7.10. Member damage regions of P385x30 for the biggest earthquake (X)

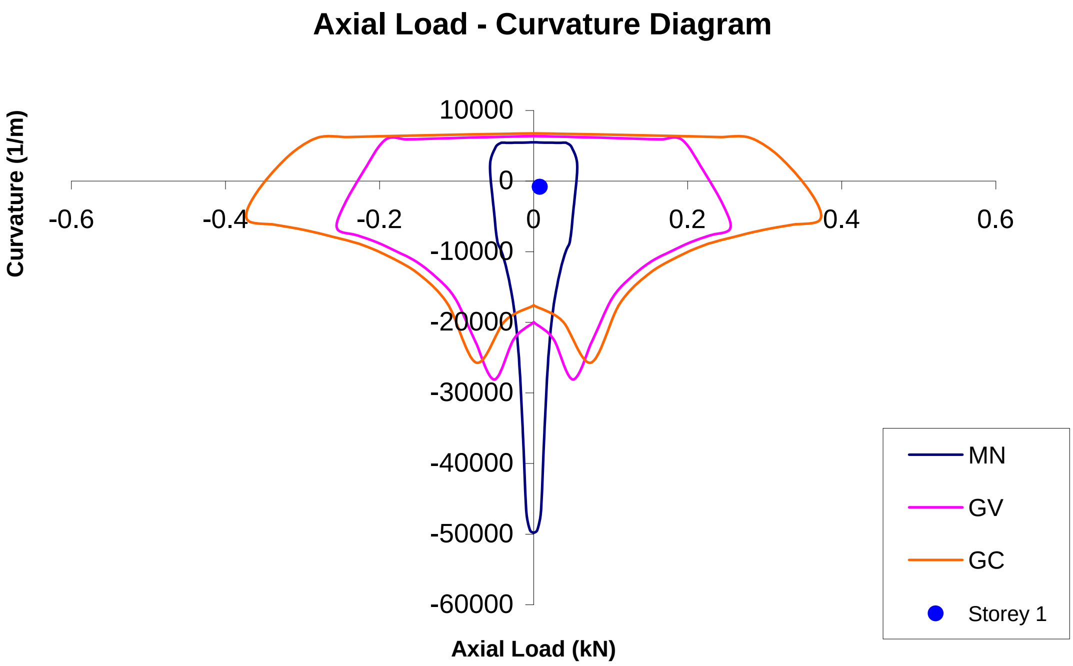

Figure 7.11. Member damage regions of P30x600 for design earthquake (Y)

7.6. Shear Strength Capacity and Column-Beam Joint Check

The shear strength capacities of the reinforced concrete members except the beam-column joints which are determined in proportion to TS-500 with the equations specified in the 6.6; have to be greater than the shear demand at the last step of the pushover analysis to call the member as ductile.

For the beam column joints; the critical joints which are investigated in the 6.7 become beam-shear wall joints; so in this section the beam column joints of the structural system are not examined.

7.7. Evaluating the Performance Levels of the Building

The structural system is a school building and it has to provide the *Immediate Occupancy Level (HK)* for the design earthquake (10% 50 years) target performance condition. Also it has to provide the *Life Safety (CG)* performance level for the biggest ground shaking (%2 50 years) condition. The performance levels for the retrofitted buildings are not described in the TEC 2007 but to examine the members whether they reach the required member performances will be a good estimation for the success of the retrofit study.

7.7.1. Design Earthquake Performance Level (10% 50 years) for X Direction

Table 7.8. Design earthquake performance level (10% 50 years) for X direction

Section	**Storey**	**MDR**	**%**	**VDR**	**%**	**SDR**	**%**	**CR**	**%**
B90x40	1	7/7	100	-	-	-	-	-	-
	2	7/7	100	-	-	-	-	-	-
B70x40	1	14/16	12	2/16	12	-	-	-	-
	2	14/16	12	2/16	12	-	-	-	-
C50x50	1	-	-	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-
C40x45	1	4/4	100	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-
C40x60	1	-	-	-	-	-	-	-	-
C30x40	2	-	-	-	-	-	-	-	-
	3	-	-	-	-	-	-	-	-
P385x30	1	4/4	100	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-
	3	-	-	-	-	-	-	-	-
Beams	**1**	**21/23**	**91**	**2/23**	**9**	**-**	**-**	**-**	**-**
	2	**21/23**	**91**	**2/23**	**9**	**-**	**-**	**-**	**-**
Columns	**1**	**4/18**	**22**	**-**	**-**	**-**	**-**	**-**	**-**
	2	**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**
	3	**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**
Shear Walls	**1**	**4/4**	**100**	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-
	3	-	-	-	-	-	-	-	-

At the first story less than 10% of the beams are in the visible damage region. The columns and shear walls as the other structural members are in the minimum damage region. So the building under the design earthquake target performance level satisfies the *Immediate Occupancy* building performance level.

As it can be seen in Table 7.9, the shear strength capacities of the members are calculated and all the members are expected to behave in a ductile manner.

Table 7.9. Shear strength capacity check (10% 50 years) for X direction

	B70x40	B90X40	C50X50	C40X45	C40X60	C30X40	P385x30	P30x600
$b_w(mm)$	700	900	500	450	600	400	300	6000
$d(mm)$	400	400	500	400	400	300	3850	300
$A_{sw}(mm^2)$	50	50	50	50	50	50	79	79
$s(mm)$	250	250	250	250	250	250	100	100
$V(kN)$	156	191	152	113	139	78	3636	1884
$V_{\max}(kN)$	99	173	80	69	65	19	1632	182
$V_{\min}(kN)$	-76	-109	-	-	-	-	-	-

Another aspect to examine the performance of the shear walls is inspecting their contributions to the shear load capacity of the system. Total base shear of the system is 8607 kN and 7256 kN of it is responded by shear walls which means %84 of the total lateral load demand.

7.7.2. The Biggest Earthquake Performance Level (2% 50 years) for X Direction

Table 7.10. The biggest earthquake performance level (2% 50 years) for X direction

Section	Storey	MDR	%	VDR	%	SDR	%	CR	%
B90x40	1	5/7	71	2/7	29	-	-	-	-
	2	5/7	71	2/7	29	-	-	-	-
B70x40	1	5/16	12	11/16	88	-	-	-	-
	2	3/16	12	13/16	88	-	-	-	-
C50x50	1	10/10	100	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-
C40x45	1	4/4	100	-	-	-	-	-	-
	2	4/4	100	-	-	-	-	-	-
C40x60	1	4/4	100	-	-	-	-	-	-
C30x40	2	4/4	100	-	-	-	-	-	-
	3	7/8	88	-	-	-	-	-	-
P385x30	1	4/4	100	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-
P30x600	1	2/2	100	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-
Beams	**1**	**10/23**	**43**	**13/23**	**57**	**-**	**-**	**-**	**-**
	2	**8/23**	**35**	**15/23**	**65**	**-**	**-**	**-**	**-**
Columns	**1**	**18/18**	**100**	**-**	**-**	**-**	**-**	**-**	**-**
	2	**8/18**	**44**	**-**	**-**	**-**	**-**	**-**	**-**
	3	**7/8**	**88**	**-**	**-**	**-**	**-**	**-**	**-**
Shear Walls	**1**	**6/6**	**100**	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-

According to the Table 7.10, there are no beams in the significant damage region. All other structural elements in the name of columns and shear walls are in the minimum damage region. Then it can be said that the system satisfies the *Life Safety* performance level for the target level of the biggest ground motion.

Table 7.11. Shear strength capacity check (2% 50 years) for X direction

	B70x40	B90X40	C50X50	C40X45	C40X60	C30X40	P385x30	P30x600
$b_w(mm)$	700	900	500	450	600	400	300	6000
$d(mm)$	400	400	500	400	400	300	3850	300
$A_{sw}(mm^2)$	50	50	50	50	50	50	79	79
$s(mm)$	250	250	250	250	250	250	100	100
$V(kN)$	156	191	152	113	139	78	3636	1884
$V_{\max}(kN)$	111	181	85	70	70	19	1969	460
$V_{\min}(kN)$	-84	-152	-	-	-	-	-	-

As it can be seen in Table 7.11, the shear strength capacities of the members are calculated and all the members are expected to behave in a ductile manner.

The shear load carried by the columns is 1912 kN and the shear load carried by the structural walls is 9716 kN, this value refers the 84% of the shear load demand.

7.7.3. Design Earthquake Performance Level (10% 50 years) for Y Direction

Table 7.12. Design earthquake performance level (10% 50 years) for Y direction

Section	Storey	MDR	%	VDR	%	SDR	%	CR	%
B40x40	1	8/18	44	-	-	-	-	-	-
	2	4/18	22	-	-	-	-	-	-
P30x600	1	2/2	100	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-
	3	-	-	-	-	-	-	-	-
Beams	**1**	**8/18**	**44**	**-**	**-**	**-**	**-**	**-**	**-**
	2	**4/18**	**22**	**-**	**-**	**-**	**-**	**-**	**-**
Shear Walls	**1**	**2/2**	**100**	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-
	3	-	-	-	-	-	-	-	-

In the building there is not any column in the visible damage region. The columns have no plastification at the target displacement. And shear walls at the first storey are in the minimum damage region. So the building under the design earthquake target performance level satisfies the *Immediate Occupancy* building performance level.

As it can be seen in Table 7.13, the shear strength capacities of all the members are smaller than the limit shear value so all of them can be mentioned as ductile.

Table 7.13. Shear strength capacity check (10% 50 years) for Y direction

	B40X40	C50X50	C40X45	C40X60	C30X40	P385x30	P30x600
$b_w(mm)$	400	500	450	600	400	300	6000
$d(mm)$	400	500	400	400	300	3850	300
$A_{sw}(mm^2)$	50	50	50	50	50	79	79
$s(mm)$	250	250	250	250	250	100	100
$V(kN)$	104	152	113	139	78	1280	5666
$V_{max}(kN)$	62	27	41	56	11	101	3149
$V_{min}(kN)$	-57	-	-	-	-	-	-

The shear walls are carrying 6704 kN of the total 7707 kN lateral load which means 87% of the total lateral load.

7.7.4. The Biggest Earthquake Performance Level (2% 50 years) for Y Direction

Table 7.14. The biggest earthquake performance level (2% 50 years) for Y direction

Section	Storey	MDR	%	VDR	%	SDR	%	CR	%
B40x40	1	16/18	89	2/18	11	-	-	-	-
	2	16/18	89	-	-	-	-	-	-
P30x600	1	2/2	100	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-
	3	-	-	-	-	-	-	-	-
Beams	**1**	**16/18**	**89**	**2/18**	**-**	**-**	**-**	**-**	**-**
	2	**16/18**	**89**	**-**	**-**	**-**	**-**	**-**	**-**
Shear Walls	**1**	**2/2**	**100**	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-
	3	-	-	-	-	-	-	-	-

According to the Table 7.14 there are no beams in the significant damage region. All other structural elements in the name of columns have no plastification at the target displacement and the shear walls are in the minimum damage region. Then it can be said that the system satisfies the *Life Safety* performance level for the target level of the biggest ground motion.

As it can be seen in Table 7.15, the shear strength capacities of all the members are smaller than the limit shear value so all of them can be mentioned as ductile.

Table 7.15. Shear strength capacity check (2% 50 years) for Y direction

	B40X40	C50X50	C40X45	C40X60	C30X40	P385x30	P30x600
$b_w(mm)$	400	500	450	600	400	300	6000
$d(mm)$	400	500	400	400	300	3850	300
$A_{sw}(mm^2)$	50	50	50	50	50	79	79
$s(mm)$	250	250	250	250	250	100	100
$V(kN)$	104	152	113	139	78	1280	5666
$V_{\max}(kN)$	72	32	46	70	18	101	5421
$V_{\min}(kN)$	-55	-	-	-	-	-	-

The shear walls are carrying 11274 kN of the total 12993 kN lateral load which means 87% of the total lateral load.

As a conclusion the building provided the *Immediate Occupancy (HK)* performance level for the design earthquake (earthquake with 10% probability to occur in 50 years) target performance condition. Also it provided the *Life Safety (CG)* performance level for the biggest ground motion (earthquake with %2 probability to occur in 50 years).

8. RETROFIT THE BUILDING BY STEEL BRACES

8.1. Introduction

As another retrofit strategy the building was strengthened with steel braces. Addition of steel braces is also a common way to improve the seismic performance of the buildings. It makes the buildings gain strength and stiffness. The basic concept is to increase the lateral strength and stiffness of the building through providing steel bracing scheme that is capable of carrying the forces induced by the incoming ground motion. Emphasis is placed on studying the effects of the brace configuration, the brace cross sectional area, the brace slenderness ratio and the bracing system arrangement on the behavior of the retrofitted frames. The steel braces must be added to the appropriate locations not to cause the defined irregularities. The steel braces are added at the locations similar to the shear walls, thereby the comparison of the effectiveness of the system is easy to interpret. But the locations have to be the most appropriate ones regarding the structural and architectural reasons.

During the section selection, the box sections are chosen and the reason to prefer the box section is that, the section's radius of gyration is same in the both directions which help to reduce the slenderness ratio of the section. The section details for the related stories can be seen in Table 8.1. The main factor for selecting brace section is that, less and bigger sections have more strength in terms of yielding and buckling and they increase the system capacity curve more but they make the columns carry extra compression and tension loads which leads axial load deformation problems. To overcome that problem more and smaller sections with less strength are preferred because they increase the stiffness and capacity curve of the system at the same time they create less axial load deformation problems on reinforced concrete columns. So it can be said that during the section selection of the braces; *stiffness* is a more privileged factor than the *strength* of the braces. Again, the third story is retrofitted to obtain a first mode shape which can be analyzed by the incremental equivalent seismic load method with the assigned plastic hinge and material idealizations.

For the third storey smaller sections are used from which adequate performances are obtained after several trials. For the system selection; concentric bracing system is preferred to the eccentric bracing system; because of the fact that the axial loads on the eccentric braces create shear loads, above the shear capacities of the beams that the eccentric braces are attached.

Table 8.1. The section details of the steel braces

Section	B (mm)	t (mm)	A (cm^2)	M (kg/m)	I (cm^4)	W (cm^3)	i (cm)	Storey Direction
100x100x6.3	100	6.3	23.2	18.2	336	67.1	3.80	3 (X)
150x150x10	150	10	54.9	43.1	1773	236	5.68	1 & 2(X)
150x150x10	150	10	54.9	43.1	1773	236	5.68	3(Y)
200x200x12.5	200	12.5	92.1	72.3	5336	534	7.61	1 & 2(Y)

Similar locations as the shear walls are added may be used for a better comparison of the effectiveness of the system but the same locations can not be used because of the following reason. The sections essential to satisfy the desired performance levels of the building under the target performance levels are obtained in the condition of same locations. Those sections carry so much axial loads that make the columns rotate more than the desired level and make the columns carry axial loads more than their capacities which lead some deformation problems because of tension or compression. So smaller and more sections at similar locations are used as it can be seen in Figure 8.1. Although an exact comparison can not be done, this condition gives some opinions to compare the structural effectiveness of the strategies.

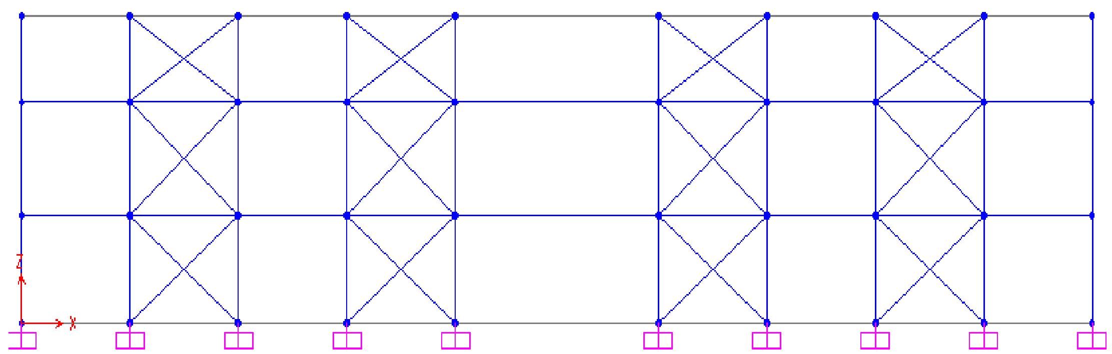

Figure 8.1. The location of steel braces in XZ direction

For the cracked section stiffness for beams and columns same values are used as in 6 and 7. There is an increase in the total story weights but not as much as in the shear wall added ones as it is shown in Table 8.2.

Table 8.2. Total storey weights with addition of steel braces

Storey	W_i (kN)	$W_{i,\text{ with shear walls}}$ (kN)	$W_{i,\text{ with braces}}$ (kN)
3	376.3	605.8	394.2
2	5449.8	5965	5486.9
1	5442.4	5971	5480.3

The addition of steel braces increases the axial loads on the columns and except the shear capacity (ductility control) control for beam and columns; the axial load capacity control also shall be done. The axial load capacities of the columns are controlled both for the tension and compression cases under the effect of uniaxial bending moments and if there is a necessity, the required precautions are also taken. As material properties of the braces ST52 steel is used for all types of braces.

The effective mass to total building mass corresponding to the first natural vibration mode in X direction was 0.913, in Y direction it was 0.919 both of them were above the value 0.70.

$$T_{1X} = 0.213 \text{ sec} \qquad\qquad T_{1Y} = 0.208 \text{ sec}$$

8.2. Definition of Plastic Hinges

The same idealizations and definitions for the plastic hinges of beams and columns, described in 6.2, are valid for the new structural system model with the addition of steel braces.

Also some idealizations are done for the hinges on the steel braces in terms buckling and yielding cases. These are important criteria for the selection of steel brace sections during the trials of the system modeling. For the yield case of the steel braces the yield strength of the brace is calculated by Equation 8.1. According to this yield strength, F_y, axial deformations, Δ_T, are obtained at that load by Equation 8.2 and in the FEMA356 (Section 5-Table 5.7) there are limit values for these axial deformations at different damage levels as it can be seen in Table 8.3. The axial load hinges (in tension) for yielding case are defined regarding the yield strength, axial deformation and axial deformation limits according to FEMA356 [23].

Table 8.3. Axial deformation limits for braces in tension (except eccentric braces)

Damage Levels	IO	LS	CP
Limit	$0.25\,\Delta_T$	$7\,\Delta_T$	$9\,\Delta_T$

$$F_y = \sigma_y \times A \tag{8.1}$$

$$\Delta_T = \frac{F_y \times l}{E \times A} \tag{8.2}$$

σ_y : Yield stress of the steel (it was used 3.6 t/cm^2 for ST52)

l : Total length of the brace

E : Modulus of elasticity (it was 2100 t/cm^2)

A : Sectional area of the brace

For the buckling case the axial load hinges (in compression) are also idealized like that; the hinge is formed regarding a load which means the buckling capacity, F_b, and after that load the brace does not carry any axial load in compression. The buckling load capacities of the braces are calculated by the Equation 8.3. The slenderness ratios of the added braces are also controlled to avoid local buckling according to TEC 2007 and the limit slenderness ratio for the rectangular sections with high ductility level is shown in Equation 8.4.

$$F_b = \frac{\pi^2 \times E \times I}{l^2} \tag{8.3}$$

$$b/t \text{ or } h/t \leq 0.7\sqrt{\frac{E}{\sigma_y}} \tag{8.4}$$

I : Moment of inertia in m^4

b : Width of the section

h : Height of the section

t : Thickness of the section

According to the above formulas the hinges under the axial loads both in tension and compression are defined in terms of yielding and buckling. The values which are required to define these hinges are given in Table 8.4.

Table 8.4. Yield and buckling capacities and slenderness ratio control for the sections

Section	Storey	$F_y(kN)$	$\Delta_T(cm)$	$F_b(kN)$	b/t	$0.7\sqrt{\frac{E}{\sigma_y}}$
100x100x6.3 (X)	1	819.36	8.47	269.2	15.87	16.9
	2	819.36	8.64	260	15.87	16.9
	3	819.36	7.81	316.5	15.87	16.9
150x150x10 (X)	1	1938.8	8.47	1546.6	15	16.9
	2	1938.8	8.64	1519.7	15	16.9
	3	1938.8	7.81	1669.9	15	16.9
150x150x10 (Y)	1	1938.8	11.47	774	15	16.9
	2	1938.8	11.59	758	15	16.9
	3	1938.8	11.02	843	15	16.9
200x200x12.5 (Y)	1	3552.4	11.47	2217	16	16.9
	2	3552.4	11.59	2281	16	16.9
	3	3552.4	11.02	2330	16	16.9

8.3. Derivation of Capacity Curve

After, all the axial load hinges for the steel braces are defined and assigned to the system; the structural system model is pushed to the target displacements for two target performance levels as the design ground motion and biggest ground motion in direction X and Y. The building has to be pushed to that displacement with an appropriate lateral load distribution which is shown below.

Lateral Load distribution for the earthquake direction X,

$$\begin{Bmatrix} 0.0195 \\ 0.0367 \\ 0.0418 \end{Bmatrix} \begin{Bmatrix} 558.64 \\ 559.32 \\ 40.26 \end{Bmatrix} = \begin{Bmatrix} 10.893 \\ 21.030 \\ 1.683 \end{Bmatrix} \rightarrow \begin{Bmatrix} 1 \\ 1.931 \\ 0.154 \end{Bmatrix} \begin{pmatrix} 1^{st}\ storey \\ 2^{nd}\ storey \\ 3^{rd}\ storey \end{pmatrix}$$

Lateral Load distribution for the earthquake direction Y,

$$\begin{Bmatrix} 0.0199 \\ 0.0365 \\ 0.0419 \end{Bmatrix} \begin{Bmatrix} 558.64 \\ 559.32 \\ 40.26 \end{Bmatrix} = \begin{Bmatrix} 11.117 \\ 20.415 \\ 1.687 \end{Bmatrix} \rightarrow \begin{Bmatrix} 1 \\ 1.836 \\ 0.152 \end{Bmatrix} \begin{pmatrix} 1^{st}\ storey \\ 2^{nd}\ storey \\ 3^{rd}\ storey \end{pmatrix}$$

The calculations below for the directions X and Y under two target performance levels are about converting the “top displacement – base shear” diagram to “modal displacement – modal acceleration” diagram by the same methodology which is described in the 6.3 for the system with addition of steel braces.

For the earthquake direction X at the first step of analysis,

$$M^*_{x1} = \begin{bmatrix} 0.466 & 0.878 & 1 \end{bmatrix} \begin{bmatrix} 558.64 & 0 & 0 \\ 0 & 559.32 & 0 \\ 0 & 0 & 40.26 \end{bmatrix} \begin{bmatrix} 0.466 \\ 0.878 \\ 1 \end{bmatrix} = 592.9\ kNs^2 / m$$

$$L^{*}_{x1} = \begin{bmatrix} 0.466 & 0.878 & 1 \end{bmatrix} \begin{bmatrix} 558.64 & 0 & 0 \\ 0 & 559.32 & 0 \\ 0 & 0 & 40.26 \end{bmatrix} \begin{bmatrix} 1 \\ 1 \\ 1 \end{bmatrix} = 791.85 \; kNs^2/m$$

$$\Gamma_{x1} = L^{*}_{x1} / M^{*}_{x1} = 791.85 / 592.9 = 1.335$$

$$M_{x1} = L^{*2}_{x1} / M^{*}_{x1} = 791.85^2 / 592.9 = 1057.54 \text{ kNs}^2\text{/m}$$

For the earthquake direction Y at the first step of analysis,

$$M^{*}_{y1} = \begin{bmatrix} 0.475 & 0.871 & 1 \end{bmatrix} \begin{bmatrix} 558.64 & 0 & 0 \\ 0 & 559.32 & 0 \\ 0 & 0 & 40.26 \end{bmatrix} \begin{bmatrix} 0.475 \\ 0.871 \\ 1 \end{bmatrix} = 590.6 \; kNs^2/m$$

$$L^{*}_{y1} = \begin{bmatrix} 0.475 & 0.871 & 1 \end{bmatrix} \begin{bmatrix} 558.64 & 0 & 0 \\ 0 & 559.32 & 0 \\ 0 & 0 & 40.26 \end{bmatrix} \begin{bmatrix} 1 \\ 1 \\ 1 \end{bmatrix} = 792.72 \; kNs^2/m$$

$$\Gamma_{y1} = L^{*}_{y1} / M^{*}_{y1} = 792.72 / 590.6 = 1.342$$

$$M_{y1} = L^{*2}_{y1} / M^{*}_{y1} = 792.72^2 / 590.6 = 1063.96 \text{ kNs}^2\text{/m}$$

The modal capacity curves are formed according to above calculations. The values for these capacity curves for earthquake direction X is given in Table 8.5 and for direction Y in Table 8.6.

Table 8.5. The values for the modal capacity curve in direction X

$u_{xN1}^{(i)}(mm)$	$V_{xN1}^{(i)}(kN)$	$d_1^{(i)}(m)$	$a_1^{(i)}(g)$
0.00	0.00	0.000	0.000
5.00	2046.92	0.004	0.197
7.87	3220.08	0.006	0.310
13.36	5054.79	0.010	0.487
18.72	6372.95	0.014	0.614
24.16	7278.44	0.018	0.702
29.80	8066.13	0.022	0.778
35.03	8589.14	0.026	0.828
41.24	9066.64	0.031	0.874
46.26	9486.65	0.035	0.914
51.37	9657.00	0.038	0.931
57.73	9812.31	0.043	0.946
63.35	9947.49	0.047	0.959
68.95	10058.19	0.052	0.970
74.12	10146.18	0.055	0.978
79.66	10217.61	0.060	0.985
84.75	10268.72	0.063	0.990
92.23	10314.50	0.069	0.994
97.55	10356.01	0.073	0.998
100.00	10395.26	0.075	1.002

Table 8.6. The values for the modal capacity curve in direction Y

$u_{yN1}^{(i)}(mm)$	$V_{yN1}^{(i)}(kN)$	$d_1^{(i)}(m)$	$a_1^{(i)}(g)$
0	0	0	0
7.50	2666.58	0.006	0.255
8.23	2914.47	0.006	0.279
16.02	5025.93	0.012	0.482
24.23	6473.69	0.018	0.620
32.29	7355.20	0.024	0.705
40.15	7870.10	0.030	0.754
47.80	8296.72	0.036	0.795
56.45	8661.54	0.042	0.830
65.02	8819.25	0.048	0.845
72.64	8957.45	0.054	0.858
80.18	9053.15	0.060	0.867
88.97	9156.80	0.066	0.877
96.48	9239.78	0.072	0.885
105.00	9320.69	0.078	0.893
112.73	9401.17	0.084	0.901
121.15	9466.98	0.090	0.907
128.65	9507.97	0.096	0.911
136.85	9529.57	0.102	0.913
144.61	9541.33	0.108	0.914
150.00	9553.48	0.112	0.915

8.4. Determination of the Target Displacement

The same methodology as in the 6.4 is used to determine the target displacement for the structural system retrofitted by steel braces. The equal displacement rule is not valid for the new model because the periods of the first modes are smaller than the T_B value for both of the earthquake directions. The elastic spectral displacements, S_{de1}, are converted to inelastic spectral displacements, S_{di1}, with the value C_{R1} which is stated in the 3.10. In the Figure 8.2 (for direction X) an in Figure 8.3 (for direction Y) spectral acceleration-spectral displacement diagrams for the earthquake demands design ground motion and the biggest ground motion and the modal capacity diagrams can be seen together on the same diagrams.

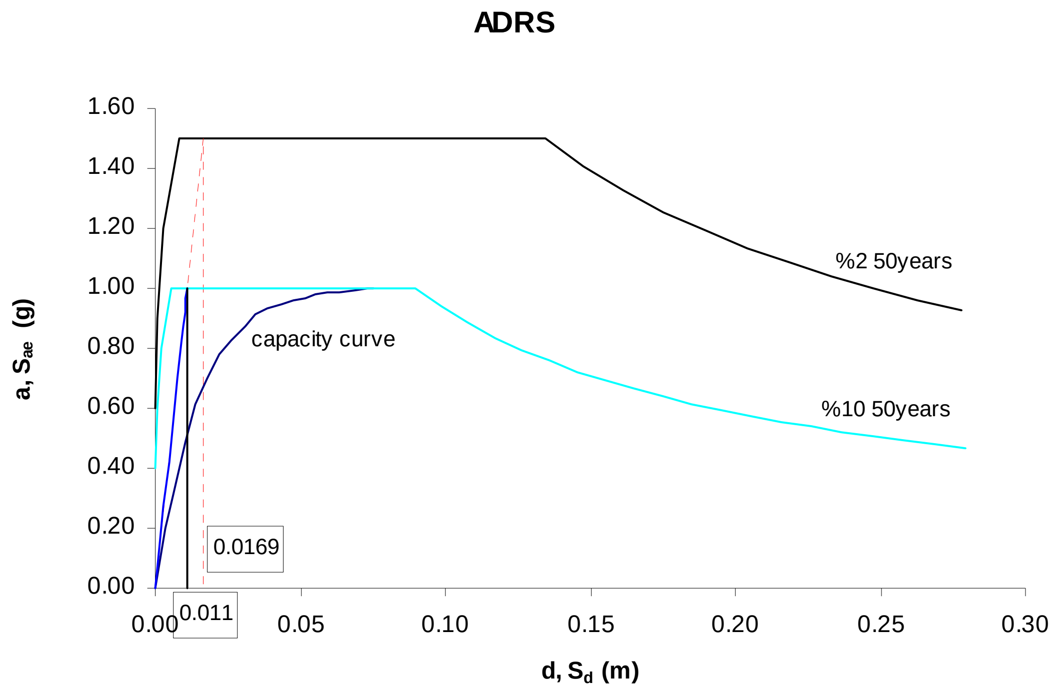

Figure 8.2. Demand and capacity diagrams for the X direction

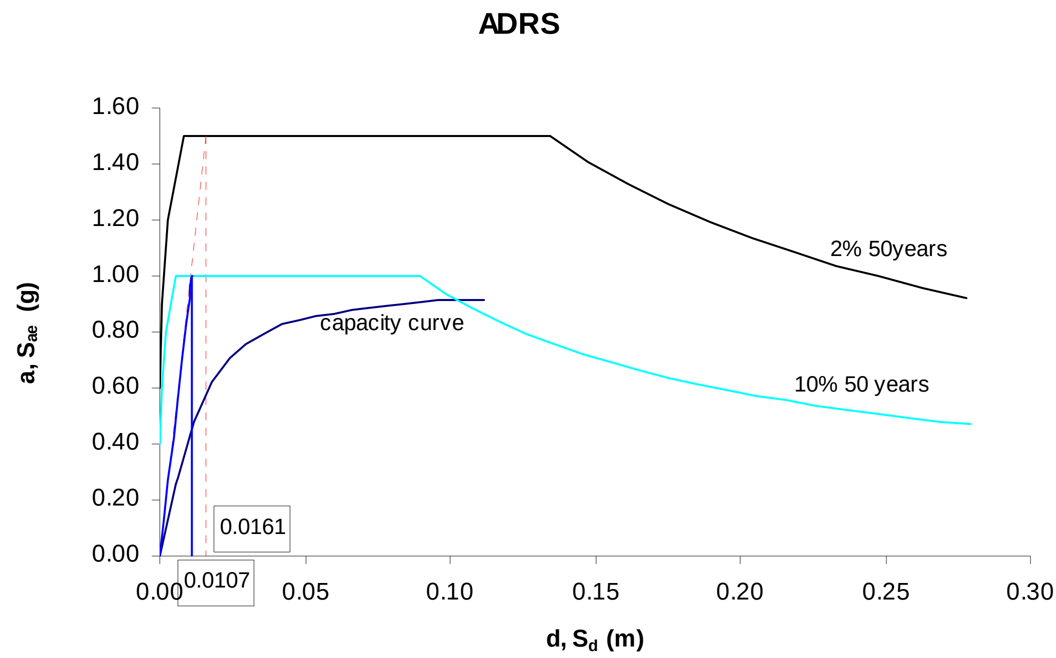

Figure 8.3. Demand and capacity diagrams for the Y direction

The spectral displacement ratios which are evaluated by consecutive iterations can be seen in Table 8.7

Table 8.7. Iterations for the spectral displacement ratios (X 10% 50 years)

Iteration	1	2	3	4	5	6
C_{R1}	2.255	1.846	1.544	1.360	1.308	1.289
R_{y1}	3.22	1.87	1.43	1.25	1.20	1.18

Spectral displacements and the target displacement under the design earthquake condition for earthquake direction X,

$$S_{di1} = C_{R1} S_{de1} = 1.289 \times 0.0112 = 0.0144 \; m.$$

$$u_{xN1} = d_1 \; (\Phi_{xN1} \; \Gamma_{x1} \;) = 0.0144 \times 1 \times 1.335 = 0.0193 \; m.$$

The spectral displacement ratios which were evaluated by consecutive iterations can be seen in Table 8.8.

Table 8.8. Iterations for the spectral displacement ratios (X 2% 50 years)

Iteration	1	2	3	4	5
C_{R1}	2.333	2.035	1.834	1.723	1.697
R_{y1}	3.73	2.32	1.85	1.66	1.62

Spectral displacements and the target displacement under the biggest earthquake condition for earthquake direction X,

$$S_{di1} = C_{R1} S_{de1} = 1.697 \times 0.0169 = 0.0287 \ m.$$

$$u_{xN1} = d_1 \ (\Phi_{xN1} \ \Gamma_{x1} \ ) = 0.0287 \times 1 \times 1.335 = 0.0383 \ m.$$

The spectral displacement ratios which were evaluated by consecutive iterations can be seen in Table 8.9.

Table 8.9. Iterations for the spectral displacement ratios (Y 10% 50 years)

Iteration	1	2	3	4	5	6	7
C_{R1}	2.413	2.022	1.694	1.525	1.399	1.322	1.279
R_{y1}	3.98	2.18	1.58	1.39	1.27	1.21	1.17

Spectral displacements and the target displacement under the design earthquake condition for earthquake direction Y,

$$S_{di1} = C_{R1} S_{de1} = 1.279 \times 0.0107 = 0.0137 \ m.$$

$$u_{xN1} = d_1 \ (\Phi_{xN1} \ \Gamma_{x1} \ ) = 0.0137 \times 1 \times 1.342 = 0.0184 \ m.$$

The spectral displacement ratios which were evaluated by consecutive iterations can be seen in Table 8.10.

Table 8.10. Iterations for the spectral displacement ratios (Y 2% 50 years)

Iteration	1	2	3	4	5	6
C_{R1}	2.542	2.180	1.977	1.853	1.795	1.768
R_{y1}	4.47	2.67	2.07	1.82	1.73	1.69

Spectral displacements and the target displacement under the biggest earthquake condition for earthquake direction Y,

$S_{di1} = C_{R1} S_{de1} = 1.768 \times 0.161 = 0.0285\ m.$

$u_{xN1} = d_1\ (\ \Phi_{xN1}\ \Gamma_{x1}\ ) = 0.0285 \times 1 \times 1.342 = 0.0382\ m.$

8.5. Specifying the Member Damage Regions

The examples Figure 8.4, Figure 8.5 and Figure 8.6, for the representation of member damage regions on the structural system model directly from the SAP2000 program, can be seen below. The methodology to evaluate that representation by the help of plastic hinge idealization and reference value-limit value description is stated in 6.5. In these figures green means minimum damage region, yellow means visible damage region, red means significant damage region and the purple means collapse region.

For columns there are two control cases for determining the exact member damage regions. One of them is the “axial load-total curvature” diagram in which there are three closed member performance level diagrams and the points, representing the axial load and the total curvature of the column at the target displacement. The methodology of this control case is defined in 6.5 and the examples can be seen in Figure 8.7 and Figure 8.8. In these figures for the tension side of the axial load axes the columns are outside the member damage regions but as an approval only the horizontal values (total curvature values) are considered and the member damage levels are defined according to these values. The second control begins at this point, the columns gain extra axial loads with the addition of steel braces and because of that the axial load capacity control is also done. The axial load capacities of the columns are controlled both for the tension and compression cases under the effect of uniaxial bending moments by drawing the “moment-axial load” diagram and determining the locations of the columns on this diagram whether in or out of that capacity curve. Four examples are shown in Figure 8.9, Figure 8.10 and Figure 8.11 in which the tension limits are exceeded by the columns like as in the figures mentioned during the axial load-curvature control. In Figure 8.12 it can be seen that the highest compression values are in the boundaries.

There is also a control for the member damage levels of steel braces. The buckling force, yielding force and the axial deformations at tension are checked at the target displacements of the pushover analysis in two directions X and Y.

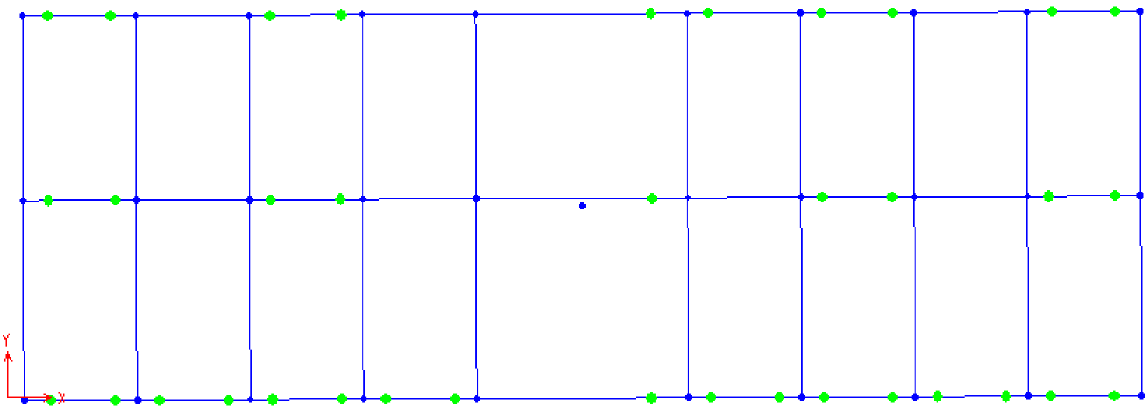

Figure 8.4. The beam damage regions for the biggest earthquake in X direction (Storey1)

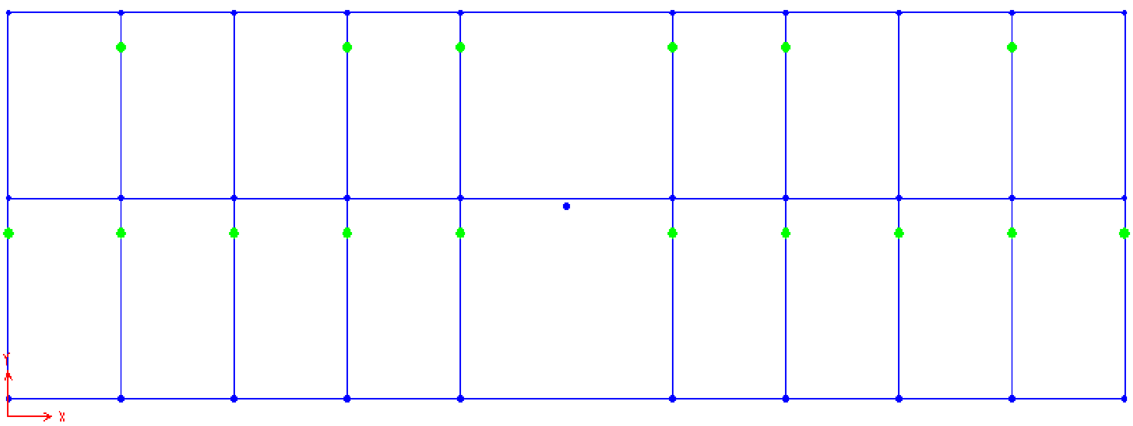

Figure 8.5. The beam damage regions for the biggest earthquake in Y direction (Storey1)

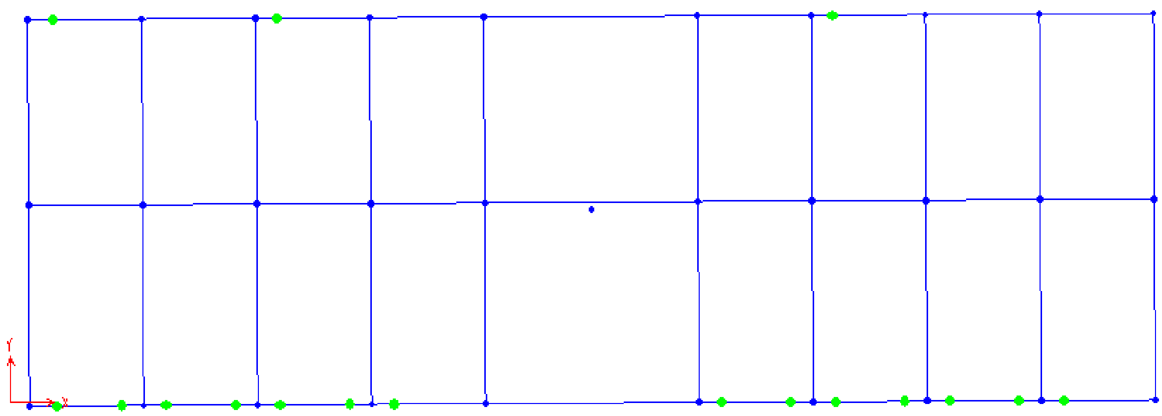

Figure 8.6. The beam damage regions for design earthquake in X direction (Storey1)

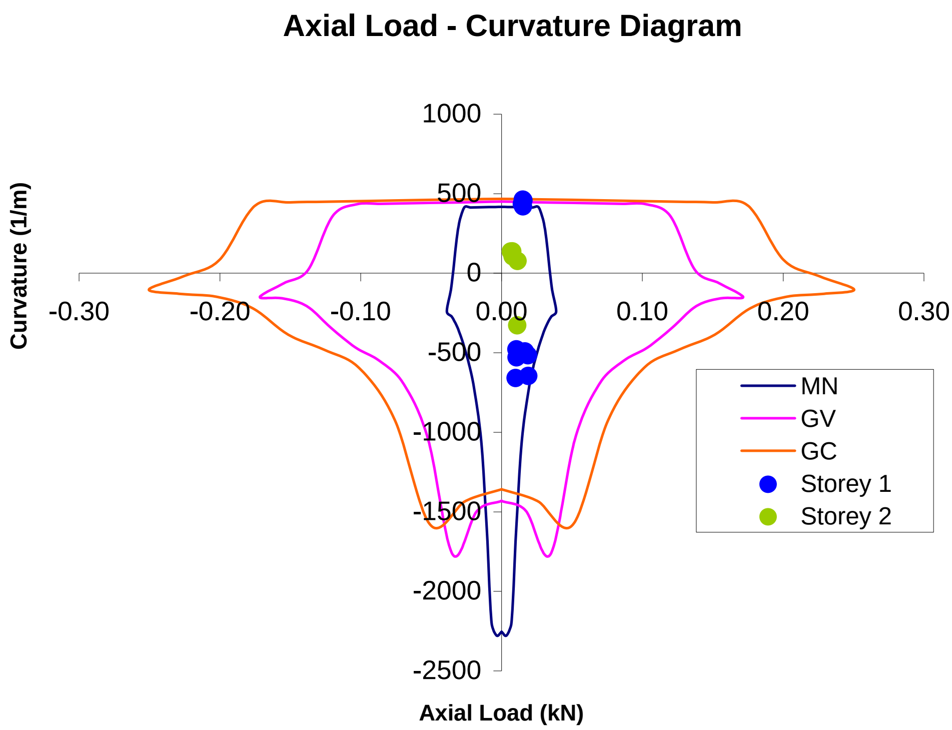

Figure 8.7. Member damage regions of C40x45 for design earthquake (X)

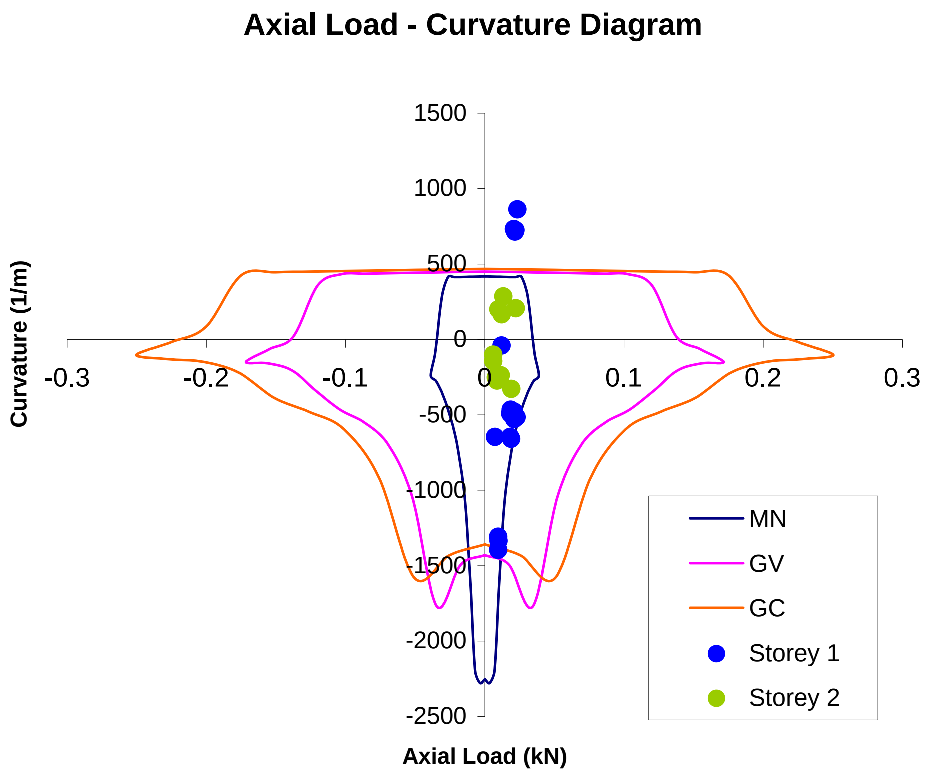

Figure 8.8. Member damage regions of C40x45 for the biggest earthquake (X)

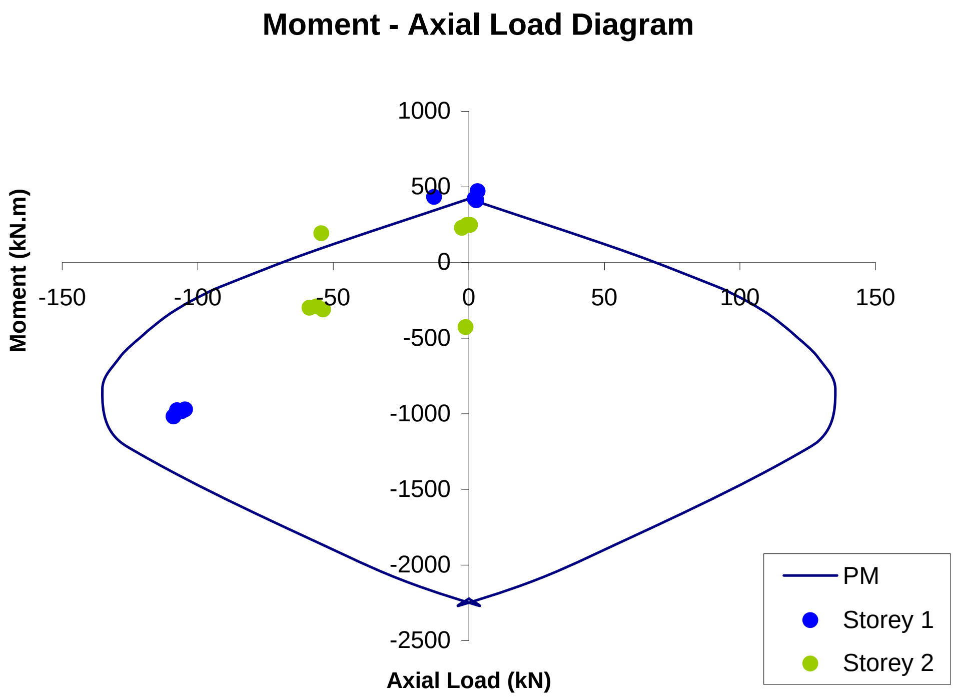

Figure 8.9. Axial load check of C40x45 for design earthquake (X)

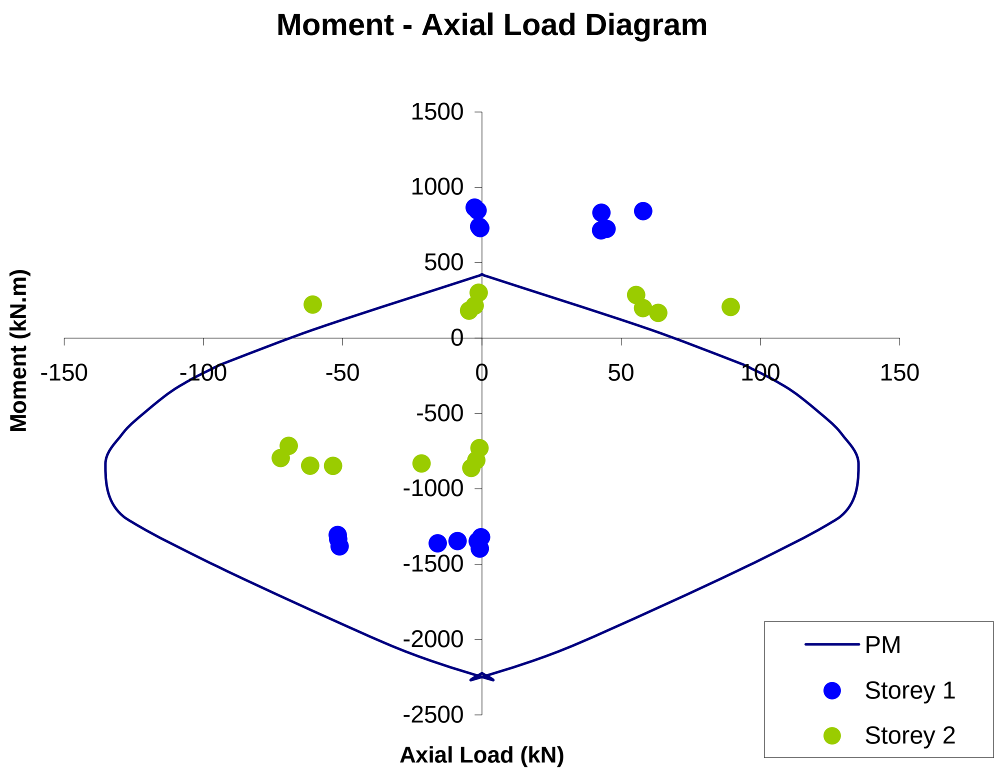

Figure 8.10. Axial load check of C40x45 for the biggest earthquake (X)

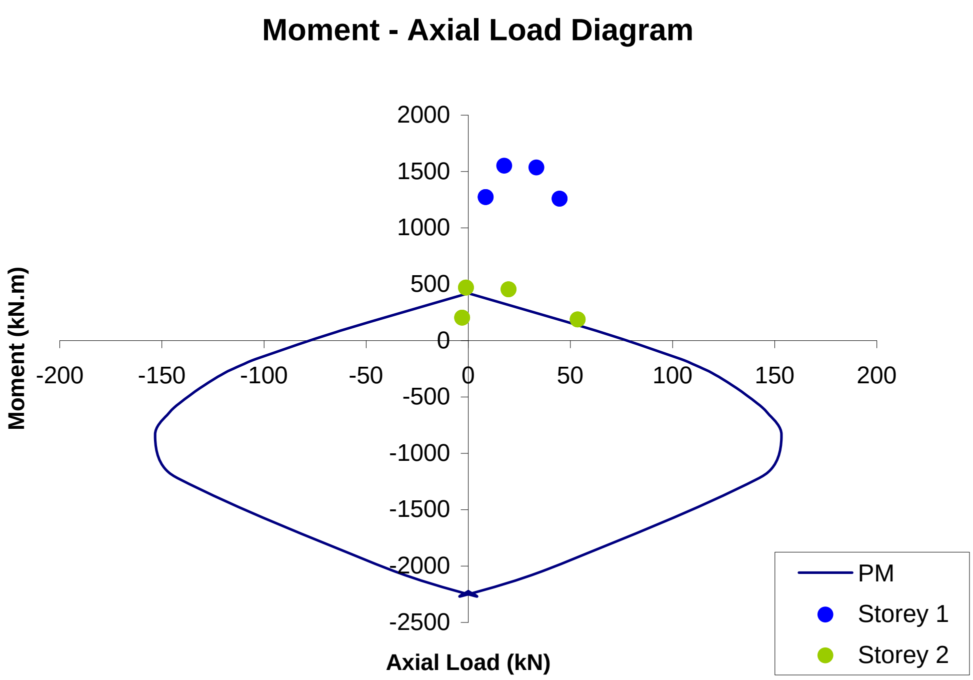

Figure 8.11. Axial load check of C40x45 for the biggest earthquake (Y)

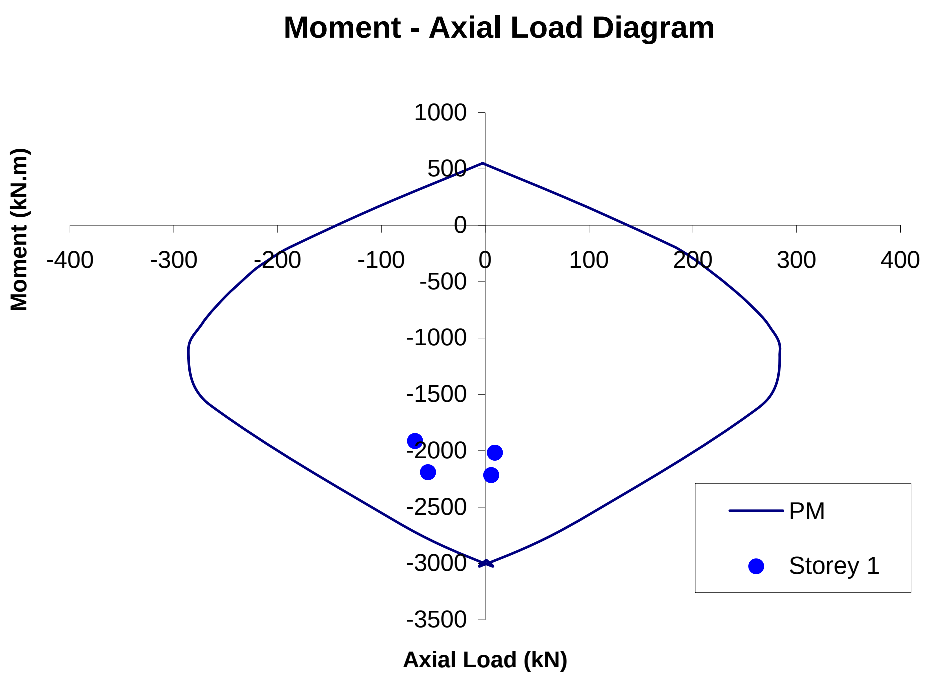

Figure 8.12. Axial load check of C40x60 for the biggest earthquake (Y)

8.6. Evaluating the Performance Levels of the Building

The evaluation of the performance level of the structural system retrofitted by concentric steel braces is done for the design earthquake and the biggest earthquake in two directions X and Y. Although the performance levels are not described in TEC 2007, providing the *Immediate Occupancy (HK)* level for the design earthquake and *Life Safety (CG)* performance level for the biggest ground shaking give a good estimation for the effectiveness of the retrofit study.

8.6.1. Design Earthquake Performance Level (10% 50 years) for X Direction

Table 8.11. Design earthquake performance level (10% 50 years) for X direction

Section	Storey	MDR	%	VDR	%	SDR	%	CR	%
B90x40	1	-	-	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-
B70x40	1	11/18	61	-	-	-	-	-	-
	2	3/18	17	-	-	-	-	-	-
C50x50	1	-	-	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-
C40x45	1	10/10	100	-	-	-	-	-	-
	2	4/10	40	-	-	-	-	-	-
C40x60	1	5/10	50	-	-	-	-	-	-
C30x40	2	4/10	100	-	-	-	-	-	-
	3	-	-	-	-	-	-	-	-
Beams	**1**	**18/27**	**67**	-	-	**-**	**-**	**-**	**-**
	2	**3/27**	**11**	-	-	**-**	**-**	**-**	**-**
Columns	**1**	**15/30**	**50**	**-**	**-**	**-**	**-**	**-**	**-**
	2	**8/30**	**27**	**-**	**-**	**-**	**-**	**-**	**-**
	3	**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**

The columns and beams are in the minimum damage region at the first and second stories. So the building under the design earthquake target performance level satisfies the *Immediate Occupancy* building performance level.

Also the braces B150x150x10 and B100x100x6.3 carrying axial loads are under their yielding and buckling capacities so there is no axial load deformation for these members.

For the axial load control of the columns *four* C40x45 columns and *four* C40x60 columns exceed their tension capacities at the bottom end plastic hinges in the first storey but in case of the member damage regions their curvature values are in the minimum damage region limit after that they are strengthened against the axial loads in the name of tension. The strengthening for axial load capacity is applied to *eight* C40x45 columns and *eight* C40x60 columns because of the earthquake directions +X and –X.

As it can be seen in Table 8.12, the shear strength capacities of the members are calculated and all the members are expected to behave in a ductile manner.

Table 8.12. Shear strength capacity check (10% 50 years) for X direction

	B70x40	B90X40	C50X50	C40X45	C40X60	C30X40
$b_w(mm)$	700	900	500	450	600	400
$d(mm)$	400	400	500	400	400	300
$A_{sw}(mm^2)$	50	50	50	50	50	50
$s(mm)$	250	250	250	250	250	250
$V(kN)$	197	243	188	139	174	96
$V_{\max}(kN)$	95	158	78	51	37	13
$V_{\min}(kN)$	-83	-144	-	-	-	-

8.6.2. The Biggest Earthquake Performance Level (2% 50 years) for X Direction

Table 8.13. The biggest earthquake performance level (2% 50 years) for X direction

Section	Storey	MDR	%	VDR	%	SDR	%	CR	%
B90x40	1	5/9	55	-	-	-	-	-	-
	2	5/9	55	-	-	-	-	-	-
B70x40	1	15/18	83	-	-	-	-	-	-
	2	12/18	67	-	-	-	-	-	-
C50x50	1	8/10	80	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-
C40x45	1	10/10	100	-	-	-	-	-	-
	2	5/10	50	-	-	-	-	-	-
C40x60	1	8/10	80	-	-	-	-	-	-
C30x40	2	4/10	100	-	-	-	-	-	-
	3	-	-	-	-	-	-	-	-
Beams	**1**	**20/27**	**74**	-	-	**-**	**-**	**-**	**-**
	2	**17/27**	**63**	-	-	**-**	**-**	**-**	**-**
Columns	**1**	**26/30**	**87**	**-**	**-**	**-**	**-**	**-**	**-**
	2	**9/30**	**30**	**-**	**-**	**-**	**-**	**-**	**-**
	3	**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**

As it can be seen in Table 8.13, there are not any beams in the significant damage region and all the columns are in the minimum damage region. So it can be said that the *Life Safety* performance level is satisfied for the biggest earthquake in the direction X.

Also the braces B150x150x10 and B100x100x6.3 are carrying axial loads under their yielding and buckling capacities so there are no axial load deformations for these members.

For the axial load control of the columns *four* C40x45 columns, *four* C40x60 columns and *one* C30x40 column exceed their tension capacities at the bottom end plastic hinges in the first storey but they are accepted as in minimum damage region after that they are strengthened for tension. The strengthening for tension capacity is applied to *eight* C40x45 columns, *eight* C40x60 columns and *two* C30x40 columns because of the earthquake direction +X and –X.

There is not any structural member, exceeding its shear capacity at the target displacement of the pushover analysis as it can be seen in the related Table 8.14.

Table 8.14. Shear strength capacity check (2% 50 years) for X direction

	B70x40	B90X40	C50X50	C40X45	C40X60	C30X40
$b_w(mm)$	700	900	500	450	600	400
$d(mm)$	400	400	500	400	400	300
$A_{sw}(mm^2)$	50	50	50	50	50	50
$s(mm)$	250	250	250	250	250	250
$V(kN)$	197	243	188	139	174	96
$V_{\max}(kN)$	107	172	87	54	52	18
$V_{\min}(kN)$	-83	-144	-	-	-	-

8.6.3. Design Earthquake Performance Level (10% 50 years) for Y Direction

Table 8.15. Design Earthquake Performance Level (10% 50 years) for Y Direction

Section	Storey	MDR	%	VDR	%	SDR	%	CR	%
B40x40	1	4/20	20	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-
C50x50	1	-	-	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-
C40x45	1	6/10	60	-	-	-	-	-	-
	2	2/10	20	-	-	-	-	-	-
C40x60	1	-	-	-	-	-	-	-	-
C30x40	2	-	-	-	-	-	-	-	-
	3	-	-	-	-	-	-	-	-
Beams	**1**	**4/20**	**20**	-	-	**-**	**-**	**-**	**-**
	2	-	-	-	-	**-**	**-**	**-**	**-**
Columns	**1**	**6/30**	**20**	**-**	**-**	**-**	**-**	**-**	**-**
	2	**2/30**	**7**	**-**	**-**	**-**	**-**	**-**	**-**
	3	**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**

Except the columns C40x45 which are in the minimum damage region the other columns do not show any plastification at the target performance level and also the beams are all in the minimum damage region. So the building under the design earthquake target performance level satisfies the *Immediate Occupancy* building performance level.

The axial load capacities of the steel braces B200x200x12.5 and B150x150x10 are not exceeded and no buckling effect and axial load deformation because of yielding is observed.

Two bottom end plastic hinges of C40x45 columns exceed their tension capacities at the first floor and the axial load strengthening method is applied to *four* C40x45 columns because of the symmetry of +Y and –Y earthquake directions.

As it can be seen in Table 8.16, the shear strength capacities of the members are calculated and all the members are accepted as ductile.

Table 8.16. Shear strength capacity check (10% 50 years) for X direction

	B40X40	C50X50	C40X45	C40X60	C30X40
$b_w(mm)$	400	500	450	600	400
$d(mm)$	400	500	400	400	300
$A_{sw}(mm^2)$	50	50	50	50	50
$s(mm)$	250	250	250	250	250
$V(kN)$	127	188	143	191	104
$V_{\max}(kN)$	67	32	45	69	13
$V_{\min}(kN)$	-57	-	-	-	-

8.6.4. The Biggest Earthquake Performance Level (2% 50 years) for Y Direction

Table 8.17. The biggest earthquake performance level (2% 50 years) for Y direction

Section	Storey	MDR	%	VDR	%	SDR	%	CR	%
B40x40	1	16/20	80	-	-	-	-	-	-
	2	8/20	40	-	-	-	-	-	-
C50x50	1	10/10	100	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-
C40x45	1	8/10	80	-	-	-	-	-	-
	2	6/10	60	-	-	-	-	-	-
C40x60	1	8/10	80	-	-	-	-	-	-
C30x40	2	4/10	40	-	-	-	-	-	-
	3	-	-	-	-	-	-	-	-
Beams	**1**	**16/20**	**80**	-	-	**-**	**-**	**-**	**-**
	2	**8/20**	**40**	-	-	**-**	**-**	**-**	**-**
Columns	**1**	**26/30**	**87**	**-**	**-**	**-**	**-**	**-**	**-**
	2	**10/30**	**33**	**-**	**-**	**-**	**-**	**-**	**-**
	3	**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**

As it can be seen in Table 7.17, there is not any beam in the visible and significant damage region and all the columns are in the minimum damage region. So it can be said that the *Life Safety* performance level is satisfied for the biggest earthquake in Y direction.

Also the braces B150x150x10 and B200x200x12.5 are carrying axial loads under their yielding and buckling capacities so no axial load deformations is observed for these members.

In the first story *six* C40x45 columns exceed their tension capacities and because of the symmetry in Y direction *six* C40x45 columns and *six* C40x60 columns are accepted as deformed by axial loads. In the second storey *four* C40x45 columns exceed their tension capacities and because of the symmetry in Y direction *four* C40x45 columns and *four* C30x40 columns are strengthened against tension loads.

There is no structural member exceeding its shear capacity at the target displacement of the pushover analysis as it can be seen in the related Table 8.18.

Table 8.18. Shear strength capacity check (2% 50 years) for X direction

	B40X40	C50X50	C40X45	C40X60	C30X40
$b_w(mm)$	400	500	450	600	400
$d(mm)$	400	500	400	400	300
$A_{sw}(mm^2)$	50	50	50	50	50
$s(mm)$	250	250	250	250	250
$V(kN)$	127	188	143	191	104
$V_{\max}(kN)$	73	66	67	111	20
$V_{\min}(kN)$	-57	-	-	-	-

As a conclusion the building provided the *Immediate Occupancy (HK)* performance level for the design earthquake (10% 50 years) target performance level. Also it provides the *Life Safety (CG)* performance level for the biggest ground motion (%2 50 years). And this assessment gives a prevision about the effectiveness of retrofitting the structural system with steel bracing.

8.7. Retrofit Strategy against Axial Load Deformations

As it is mentioned above there are columns which will be deformed as brittle because of the tension loads from the transferred axial loads of steel braces to the columns. In the first storey *ten* C40x45 and *ten* C40x60 columns; in the second storey *ten* C40x45 and *ten* C30x40 columns exceed the tension capacities during the pushover analysis. Although their axial load capacities are not in the range of required member damage regions, they are assumed as strengthened against axial load deformations and only the curvature values are taken into account. The precautions for increasing the tensile strength also increase the ductility and rotation capacities of the columns but this effect is neglected because of the modeling simplicity and to be on the safe side.

Columns have not got enough tension capacity so some strategies are thought and some of them are applied to the structural model. For the RC jacketing strategy, it is just an assumption to accept that the columns which have been there for eighty years with a vey low strength and low modulus of elasticity, because of the strength reduction and creep; will work together with the RC jackets. Although it is accepted as above, in the structural system model the limited reinforcement areas of the RC jackets are not adequate to overcome the tension stress on the column.

Fiber reinforced polymers that are used to strengthen the beam column joints generally will not be sufficient to resist the tension loads occurr on the columns.

Jacketing with the steel plates or locating the braces eccentric to columns will not be agreed because existing columns remained inside of the 90x40 and 70x40 wide beams. It is not possible to make continuous system without breaking the beams for steel plate jacketing. Breaking the beams or anchored the steel plates with bolts to the other storey throughout the beam which makes the beam shear cracked; are not good alternatives to apply. To strengthen the column with steel plates up to the joint can not be used also because the joint region will carry that tensile load and it will form axial load deformation at the joint.

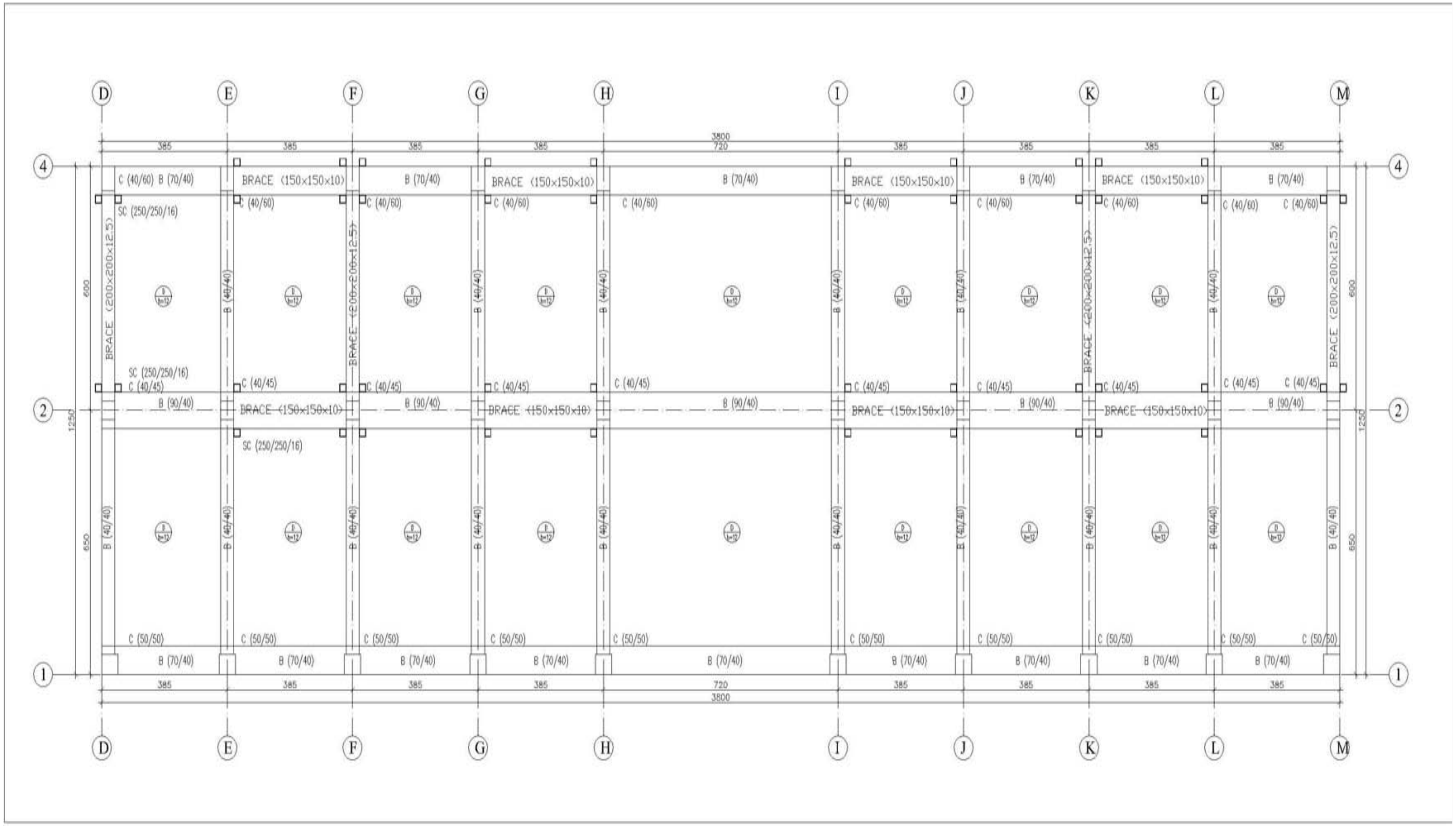

Figure 8.13. Location of the steel columns for retrofit against axial load

After all these trials the appropriate strategy is found, a steel frame will be formed around the columns which are deformed because of tensile stresses. Figure 8.13 shows the location of the steel columns added to the system. In Figure 8.14 sectional view for X direction and Figure 8.15 illustrates the details of weld and bolted connections. In Figure 8.16 sectional view for Y direction can be seen and Figure 8.17 illustrates the details of weld and bolted connections. Figure 8.18 shows the plan view of a connection detail,

Steel frame mentioned above compose of members as steel columns, steel beams, steel plates and steel connector beam. These members can be seen in Figure 8.14 and in Figure 8.16 and to talk about the load transfer principle of this structural system it can be said that the steel beam is connected to reinforced concrete beam with bolts (anchorages with epoxy) that make the steel frame system work properly with existing RC structural system. The displacement demand of the ground motion will move the slab and the RC beam in a direction, for example -X direction, and these anchored bolts can make the steel beam move in the same direction. Then the steel plate at the connection region will resist that demand of the steel beam and transferees load to the connector beam that connects the steel columns and also creates a compression load on the steel brace. According to this mechanism the tensile loads which are created on the RC columns are carried by steel columns. Then through the steel columns the load is transferred to the foundation in which a new foundation system is added in terms of steel columns. Calculations done to evaluate the appropriate sections and required connection details are below. In the system 48 steel columns are used; the slab is drilled to make these columns continue to the next storey; there are two columns next to one RC column and four steel columns are used to connect one X brace to the structural system and four steel columns are added to the system without connecting to braces but just to form a symmetry around the RC column.

The shear capacities of the beam column joints of the reinforced concrete system must be adequate because the considerable amount of shear loads is transferred along the added steel frames to the foundation of the system.

8.7.1. Connection Members and Connection Details for X Direction

Calculations are examined below with six parts which are named with letters according to Figure 8.14 but the figure is just an imaginative representation of the connection detail, calculations are done according to the maximum load cases at different connections [24].

(a) In that part, calculations to find the required steel column section are defined in Equation 8.5. Steel column (SC 250x250x16) has to resist the tensile forces without the vertical loads which are accepted to be carried by RC columns. In some connections two columns resist the tensile force. The steel column section is selected as 250x250x16 box section because it has to be consistent with the steel beam that connected two columns.

$$N_{\max} = A_{\text{section}} \times \sigma_{all} \tag{8.5}$$

$N = 864\ kN$, (at the last step of pushover analysis)

$N = 1179\ kN$, (without vertical loads)

$N = 589.5\ kN$, (for one steel column)

$A_{\text{section}} = 147\ cm^2$, (**SC 250x250x16**)

$\sigma_{all} = 24\ kN / cm^2$, (ST52 HZ)

HZ means the loading type consists of vertical loads with lateral loads.

$N_{\max} = 147 \times 24 = 3528\ kN > 589.5\ kN$

It is accepted that the compression load is taken by RC columns but an investigation also made for the case, compression load without vertical loads is carried by steel and RC columns together regarding their stiffness which is defined by the modulus of elasticity, the sectional area and the section length as it can be seen in Equation 8.6. One RC column and two steel columns are taken into account to calculate the worst case situation.

$$k = \frac{E \times A_{\text{section}}}{l_{\text{section}}} \tag{8.6}$$

$E_{steel} = 2 \times 10^5\ MPa$

$E_{RC} = 0.158 \times 10^5\ MPa$

$A_{steel} = 147\ cm^2$

$A_{RC} = 40 \times 45 = 1800 cm^2$

$k_{steel} = \dfrac{2 \times 10^5 \times 147 \times 10^2}{325 \times 10} = 904.6 \times 10^3\ \text{N/mm}$

$k_{RC} = \dfrac{0.158 \times 10^5 \times 1800 \times 10^2}{325 \times 10} = 875.1 \times 10^3\ \text{N/mm}$

$k_{RC} = 0.97\ k_{steel}$

Two steel columns with stiffness k and one reinforced concrete column with stiffness *0.97k* resist $1396\ kN$ compression load and this load was accepted $1081\ kN$ without gravity loads and this control can be done by Equation 8.7 and Equation 8.8.

$$N_{\max} = \frac{A_{\text{section}} \times \sigma_{all}}{w} \tag{8.7}$$

$$\lambda = \frac{s}{i} \rightarrow w \tag{8.8}$$

$\lambda = \dfrac{s}{i} = \dfrac{325}{9.50} \rightarrow w = 1.14$

$N_{\max} = \dfrac{147 \times 24}{1.14} = 3528\ kN > 305.4\ kN$

(b) The section of the steel beam between two columns has to resist the axial load coming from the steel brace. After that requirement, this connector beam is added to the columns with *full penetration groove weld* at the mill and the sections of the connector have to be consistent with the steel column. The critical point of the section investigation of the connector is that, it had to resist the moment created by the connection length and the brace axial load. This moment is calculated by Equation 8.9 and the moment capacity of the chosen section 250x250x16 is calculated by Equation 8.10.The axial load on the brace is $937\ kN$.

$$M = \frac{P \times l}{4} \tag{8.9}$$

$$M_{capacity} = W \times \sigma_{all} \tag{8.10}$$

$$M = \frac{907 \times 90}{4} = 20407.5\ kN.cm$$

$$M_{capacity} = 1061 \times 24 = 25464\ kN.cm > 20407.5\ kN.cm$$

(c) The brace is connected to the connector beam with a steel plate connection. Steel plate is welded to the 250x250x16 connector beam and the capacity check of that welded connection is done regarding the Equation 8.11 and Equation 8.12

$$P = 907\ kN$$

$$a = 10\ mm \qquad \begin{pmatrix} \mathrm{a} \ \geq \ 3\ mm \\ \mathrm{a} \ \leq \ 0.7 \times \mathrm{t_{min}} \\ \mathrm{a} \ \leq \ 0.7 \times 16 = 11.2\ mm \end{pmatrix}$$

$$F_{weld} = \sum a \times l \tag{8.11}$$

$$F_{weld} = 4 \times (25 - 1) \times 1 = 96\ cm^2$$

$$\tau_{w,all} = 19\ kN / cm^2, \quad \text{(ST 52, HZ loading type)}$$

$$\tau_{w,all} = \frac{P}{F_{weld}} \tag{8.12}$$

$$\tau_{w,all} = \frac{907}{96} = 9.45\ kN / cm^2 < 19\ kN / cm^2$$

(d) 150x150x10 steel brace is connected to the steel frame with a steel plate which is located through the middle of the box section and welded to that brace. The capacity control of the welded connection is investigated with Equation 8.11 and Equation 8.12.

As a result of the plastic analysis (at the last step of the pushover analysis) it is seen that the steel braces remain in the elastic region. Because of that the connections are designed according to the axial load on the braces and the load capacities of the braces are not taken into account.

Anyway, calculations for the connection details are done according to ASD (Allowable Stress Design) which also gives an extra safety to the design considerations.

$$P = 907\ kN$$

$$a = 7\ mm \qquad \begin{pmatrix} \mathrm{a}\ \geq\ 3\ mm \\ \mathrm{a}\ \leq 0.7 \times \mathrm{t}_{\mathrm{min}} \\ \mathrm{a}\ \leq 0.7 \times 10 = 7\ mm \end{pmatrix}$$

$$F_{weld} = 4 \times (40 - 2 \times 0.7) \times 0.7 = 108.08\ cm^2$$

$$\tau_{w,all} = 19\ kN / cm^2 \quad , \qquad \text{(ST 52, HZ loading type)}$$

$$\tau_{w,all} = \frac{907}{108} = 8.39\ kN / cm^2 < 19\ kN / cm^2$$

(e) The plate mentioned above is connected to a steel plate which connects the steel brace to the connector beam that is stated in 8.7.1 (c). This connection is a bolted connection and the related calculations are done according to the Equation 8.13 and Equation 8.14.

$$P = 907\ kN$$

$$P_{s,all} = 191\ kN \qquad \text{, (for one bolt M30 ST52 10.9 type)}$$

$$t_{\min} = \min\ (t_1, t_2) = (30, 30) = 30\ mm$$

$$\sigma_{all} = 48\ kN / cm^2$$

$$P_{b,all} = \sigma_{all} \times t_{\min} \times d \tag{8.13}$$

$P_{b,all} = 48 \times 3 \times 3.1 = 446.4\ kN$

$P_{all} = \min\ (P_{s,all}, P_{b,all}) = \min\ (191\ ,\ 446.4) = 191\ kN$

$$n = \frac{P}{P_{all}} \tag{8.14}$$

$n = \frac{907}{191} = 6$ M30 bolts

Bolt spacing check horizontally,

$$e_1 = 75\ mm \begin{pmatrix} > 2d = 2\times 31 = 62\ mm \\ < 3d = 3\times 31 = 93\ mm \\ < 6t_{\min} = 6\times 30 = 180\ mm \end{pmatrix},\quad e = 100\ mm \begin{pmatrix} > 3d = 3\times 31 = 93\ mm \\ < 8d = 8\times 31 = 248\ mm \\ < 15t_{\min} = 15\times 30 = 450\ mm \end{pmatrix}$$

Bolt spacing check vertically,

$$e_1 = 75\ mm \begin{pmatrix} > 2d = 2\times 31 = 62\ mm \\ < 3d = 3\times 31 = 93\ mm \\ < 6t_{\min} = 6\times 30 = 180\ mm \end{pmatrix},\quad e = 100\ mm \begin{pmatrix} > 3d = 3\times 31 = 93\ mm \\ < 8d = 8\times 31 = 248\ mm \\ < 15t_{\min} = 15\times 30 = 450\ mm \end{pmatrix}$$

To investigate the axial load bearing capacity of the steel plate 800x250x30, Equation 8.15 is used.

$\sigma_{all} = 24\ kN / cm^2$

$$P_{all} = (h - 2d) \times t \times \sigma_{all} \tag{8.15}$$

$P_{all} = (25 - 2\times 3.1) \times 3 \times 24 = 1353.6\ kN > 907\ kN$

(f) To form a continuous steel frame the last part is the steel beam which also connects the steel system to the reinforced concrete structure. The steel beam is composed of a T beam in which two plates with a height of 300 mm. and with a thickness of 30 mm are connected with full penetration groove weld. This steel beam is also connected to above RC beam with anchorage bolts and the holes are strengthened with epoxy. This connection makes the steel beam move together with the RC slab and it is assumed that these two different systems are working together properly. The steel beam is connected to the steel frame with the same plate that connects the braces to the connector beam and the beam-plate connection is a bolted connection. The capacity check of the bolted connection is done by Equation 8.13 and Equation 8.14. As an important subject the holes of the bolts are 1.5 times of the bolt diameter in vertical direction which gives a release to the beam in vertical direction and by the help of that release the steel beam had no vertical axial load on it, which can cause a shear failure in the connected RC beam.

$P = 907 \times \cos 40.2 = 716\ kN$

$P_{s,all} = 191\ kN$, (for one bolt M30 ST52 10.9 type)

$t_{\min} = \min\ (t_1, t_2) = (30,30) = 30\ mm$

$\sigma_{all} = 48\ kN / cm^2$

$P_{b,all} = 48 \times 3 \times 3.2 = 446.4\ kN$

$P_{all} = \min\ (P_{s,all}, P_{b,all}) = \min\ (191\ ,\ 446.4) = 191\ kN$

$n = \dfrac{716}{191} = 6$ M30 bolts

Bolt spacing check horizontally,

$$e_1 = 75\ mm \begin{pmatrix} > 2d = 2 \times 31 = 62\ mm \\ < 3d = 3 \times 31 = 93\ mm \\ < 6t_{\min} = 6 \times 30 = 180\ mm \end{pmatrix},\quad e = 100\ mm \begin{pmatrix} > 3d = 3 \times 31 = 93\ mm \\ < 8d = 8 \times 31 = 248\ mm \\ < 15t_{\min} = 15 \times 30 = 450\ mm \end{pmatrix}$$

Bolt spacing check vertically,

$$e_1 = 75\ mm \begin{pmatrix} > 2d = 2\times 31 = 62\ mm \\ < 3d = 3\times 31 = 93\ mm \\ < 6t_{\min} = 6\times 30 = 180\ mm \end{pmatrix},\quad e = 150\ mm \begin{pmatrix} > 3d = 3\times 31 = 93\ mm \\ < 8d = 8\times 31 = 248\ mm \\ < 15t_{\min} = 15\times 30 = 450\ mm \end{pmatrix}$$

To investigate the axial load bearing capacity of the steel plate connected to steel beam, Equation 8.15 is used.

$$\sigma_{all} = 24\ kN/cm^2$$

$$P_{all} = (30 - 2\times 3.1\times 1.5)\times 3\times 24 = 1490.4\ kN > 716\ kN$$

8.7.2. Connection Members and Connection Details for Y Direction

Connection details for the Y direction are designed under six parts represented in Figure 8.16. The figure is just an imaginative representation for the connection detail, below calculations are done according to the maximum load case. In all the details same methodology and same strategy for the X direction are used to apply the connection details so just the calculations are given in that section.

(a) In that part, calculations to find the required steel column section are defined in Equation 8.5. (SC 250x250x16).

$$N_{\max} = A_{section} \times \sigma_{all}$$

$N = 1578\ kN$, (at the last step of pushover analysis)

$N = 1893\ kN$, (without vertical loads)

$N = 946.5\ kN$, (for one steel column)

$A_{section} = 147\ cm^2$, (**SC 250x250x16**)

$\sigma_{all} = 24\ kN/cm^2$, (ST52, HZ)

HZ means the loading type consists of vertical loads with lateral loads.

$N_{\max} = 147 \times 24 = 3528\ kN > 946.5\ kN$

As it can be seen in Equation 8.6, one RC column and two steel columns are taken into account to calculate the worst case situation for the compression capacity check. For the +Y direction of seismic motion tension loads are carried by C40x45 columns and C40x60 columns are under compression loads.

$$E_{steel} = 2 \times 10^5\ MPa$$

$$E_{RC} = 0.158 \times 10^5\ MPa$$

$$A_{steel} = 147\ cm^2$$

$$A_{RC} = 40 \times 60 = 2400 cm^2$$

$$k_{steel} = \frac{2 \times 10^5 \times 147 \times 10^2}{325 \times 10} = 904.6 \times 10^3\ \text{N/mm}$$

$$k_{RC} = \frac{0.158 \times 10^5 \times 2400 \times 10^2}{325 \times 10} = 1166.8 \times 10^3\ \text{N/mm}$$

$$k_{RC} = 1.29\ k_{steel}$$

Two steel columns with stiffness k and one reinforced concrete column with stiffness *1.29k* resisted $2216\ kN$ compression load and this load was accepted $2065\ kN$ without gravity loads and this control can be done by Equation 8.7 and Equation 8.8.

$$\lambda = \frac{s}{i} = \frac{325}{9.50} \rightarrow w = 1.14$$

$$N_{\max} = \frac{147 \times 24}{1.14} = 3528\ kN > 627.66\ kN$$

(b) The moment on the connector beam SC 250x250x16 is calculated by Equation 8.9 and the moment capacity of the chosen section is calculated by Equation 8.10.The axial load on the brace is $1644\ kN$.

$$M = \frac{1644 \times 40}{4} = 16440\ kN.cm$$

$$M_{capacity} = 1061 \times 24 = 25464\ kN.cm > 16440\ kN.cm$$

(c) The brace is connected to the connector beam with one steel plate. Steel plate is welded to the 250x250x16 connector beam and the capacity check of that welded connection is done regarding the Equation 8.11 and Equation 8.12

$$P = 1644\ kN$$

$$a = 11\ mm \qquad \begin{pmatrix} \mathrm{a} \ \geq \ 3\ mm \\ \mathrm{a} \ \leq \ 0.7 \times \mathrm{t}_{\mathrm{min}} \\ \mathrm{a} \ \leq \ 0.7 \times 16 = 11.2\ mm \end{pmatrix}$$

$$F_{weld} = 4 \times (25 - 1.1) \times 1.1 = 105.6\ cm^2$$

$$\tau_{w,all} = \frac{1644}{105.6} = 15.57\ kN / cm^2 < 19\ kN / cm^2$$

(d) 200x200x12.5 steel brace is connected to the steel frame with a steel plate which is located through the middle of the box section and welded to that brace. The capacity control of the welded connection is investigated with Equation 8.11 and Equation 8.12.

$$P = 1644\ kN$$

$$a = 8\ mm \qquad \begin{pmatrix} \mathrm{a} \ \geq \ 3\ mm \\ \mathrm{a} \ \leq \ 0.7 \times \mathrm{t}_{\mathrm{min}} \\ \mathrm{a} \ \leq \ 0.7 \times 12.5 = 8.75\ mm \end{pmatrix}$$

$$F_{weld} = 4 \times (50 - 2 \times 0.8) \times 0.8 = 154.88\ cm^2$$

$$\tau_{w,all} = 19\ kN / cm^2 \qquad \text{(ST 52, HZ loading type)}$$

$$\tau_{w,all} = \frac{1644}{154.88} = 10.61\ kN / cm^2 < 19\ kN / cm^2$$

(e) The plate mentioned above is connected to a steel plate which connects the steel brace to the connector beam that is stated in 8.7.2 (c). This connection is a bolted connection and the related calculations are done according to the Equation 8.13 and Equation 8.14.

$$P = 1644\ kN$$

$$P_{s,all} = 191\ kN \quad \text{, (for one bolt M30 ST52 10.9 type)}$$

$$t_{\min} = \min\ (t_1, t_2) = (35, 35) = 35\ mm$$

$$\sigma_{all} = 48\ kN / cm^2$$

$$P_{b,all} = 48 \times 3.5 \times 3.1 = 520.8\ kN$$

$$P_{all} = \min\ (P_{s,all}, P_{b,all}) = \min\ (191\ ,\ 520.8) = 191\ kN$$

$$n = \frac{1644}{191} = 10 \ \text{ M30 bolts}$$

Bolt spacing check horizontally,

$$e_1 = 75\ mm \begin{pmatrix} > 2d = 2 \times 31 = 62\ mm \\ < 3d = 3 \times 31 = 93\ mm \\ < 6t_{\min} = 6 \times 35 = 210\ mm \end{pmatrix},\ e = 100\ mm \begin{pmatrix} > 3d = 3 \times 31 = 93\ mm \\ < 8d = 8 \times 31 = 248\ mm \\ < 15t_{\min} = 15 \times 35 = 525\ mm \end{pmatrix}$$

Bolt spacing check vertically,

$$e_1 = 75\ mm \begin{pmatrix} > 2d = 2 \times 31 = 62\ mm \\ < 3d = 3 \times 31 = 93\ mm \\ < 6t_{\min} = 6 \times 35 = 210\ mm \end{pmatrix},\ e = 200\ mm \begin{pmatrix} > 3d = 3 \times 31 = 93\ mm \\ < 8d = 8 \times 31 = 248\ mm \\ < 15t_{\min} = 15 \times 35 = 525\ mm \end{pmatrix}$$

To investigate the axial load bearing capacity of the steel plate 1100x350x35, Equation 8.15 is used.

$\sigma_{all} = 24\ kN/cm^2$

$P_{all} = (35 - 2\times 3.1)\times 3.5\times 24 = 2419.2\ kN > 1644\ kN$

(f) The steel beam is connected to the steel frame with the same plate that connects the brace to the connector beam and the beam-plate connection is a bolted connection. The capacity check of the bolted connection is done by Equation 8.13 and Equation 8.14.

$P = 1644\times \cos 28.1 = 1451\ kN$

$P_{s,all} = 191\ kN$, (for one bolt M30 ST52 10.9 type)

$t_{\min} = \min\ (t_1, t_2) = (35, 35) = 35\ mm$

$\sigma_{all} = 48\ kN/cm^2$

$P_{b,all} = 48\times 3\times 3.1 = 446.4\ kN$

$P_{all} = \min\ (P_{s,all}, P_{b,all}) = \min\ (191\ ,\ 446.4) = 191\ kN$

$n = \dfrac{1451}{191} = 8$ M30 bolts

Bolt spacing check horizontally,

$$e_1 = 75\ mm \begin{pmatrix} > 2d = 2\times 31 = 62\ mm \\ < 3d = 3\times 31 = 93\ mm \\ < 6t_{\min} = 6\times 30 = 180\ mm \end{pmatrix}, e = 100\ mm \begin{pmatrix} > 3d = 3\times 31 = 93\ mm \\ < 8d = 8\times 31 = 248\ mm \\ < 15t_{\min} = 15\times 30 = 450\ mm \end{pmatrix}$$

Bolt spacing check vertically,

$$e_1 = 75\ mm \begin{pmatrix} > 2d = 2\times 31 = 62\ mm \\ < 3d = 3\times 31 = 93\ mm \\ < 6t_{\min} = 6\times 30 = 180\ mm \end{pmatrix}, e = 150\ mm \begin{pmatrix} > 3d = 3\times 31 = 93\ mm \\ < 8d = 8\times 31 = 248\ mm \\ < 15t_{\min} = 15\times 30 = 450\ mm \end{pmatrix}$$

To investigate the axial load bearing capacity of the steel plate connected to the steel beam, Equation 8.15 is used.

$$\sigma_{all} = 24\ kN / cm^2$$

$$P_{all} = (30 - 2 \times 3.1 \times 1.5) \times 3.5 \times 24 = 1738.8\ kN > 1451\ kN$$

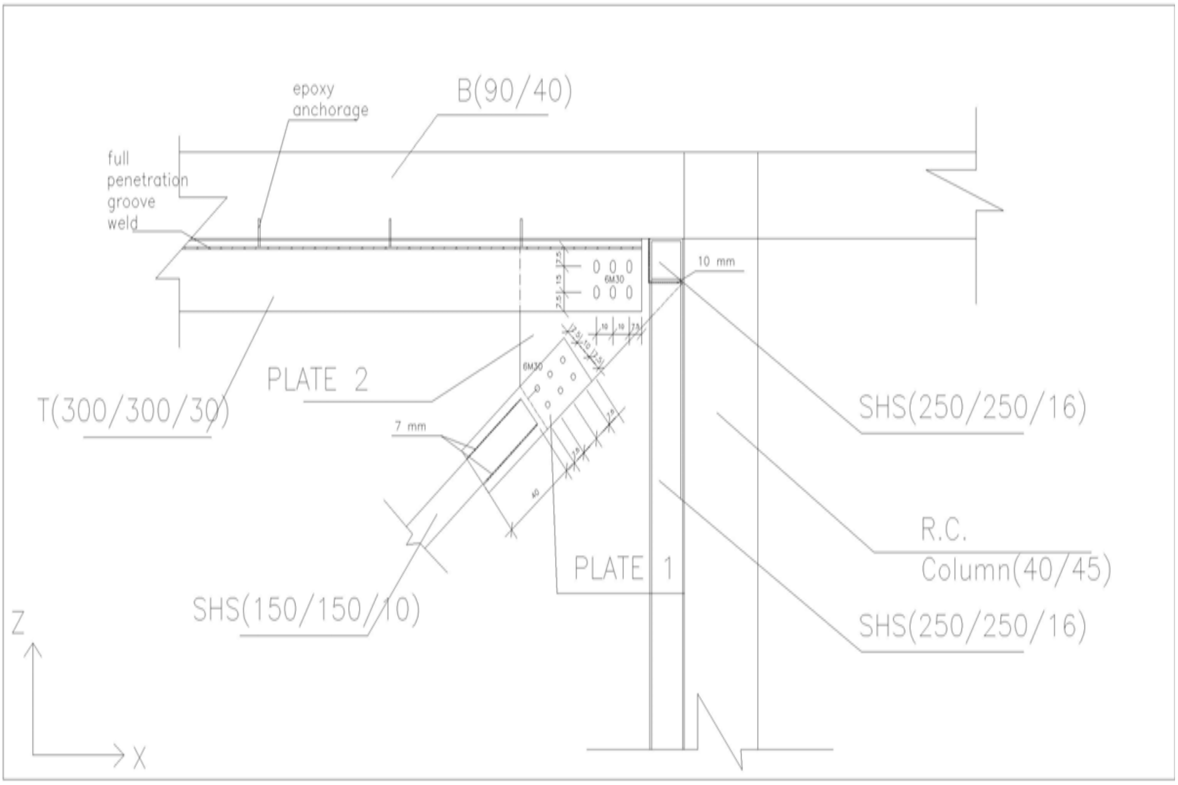

Figure 8.14. Side view for the connection detail (X-Z)

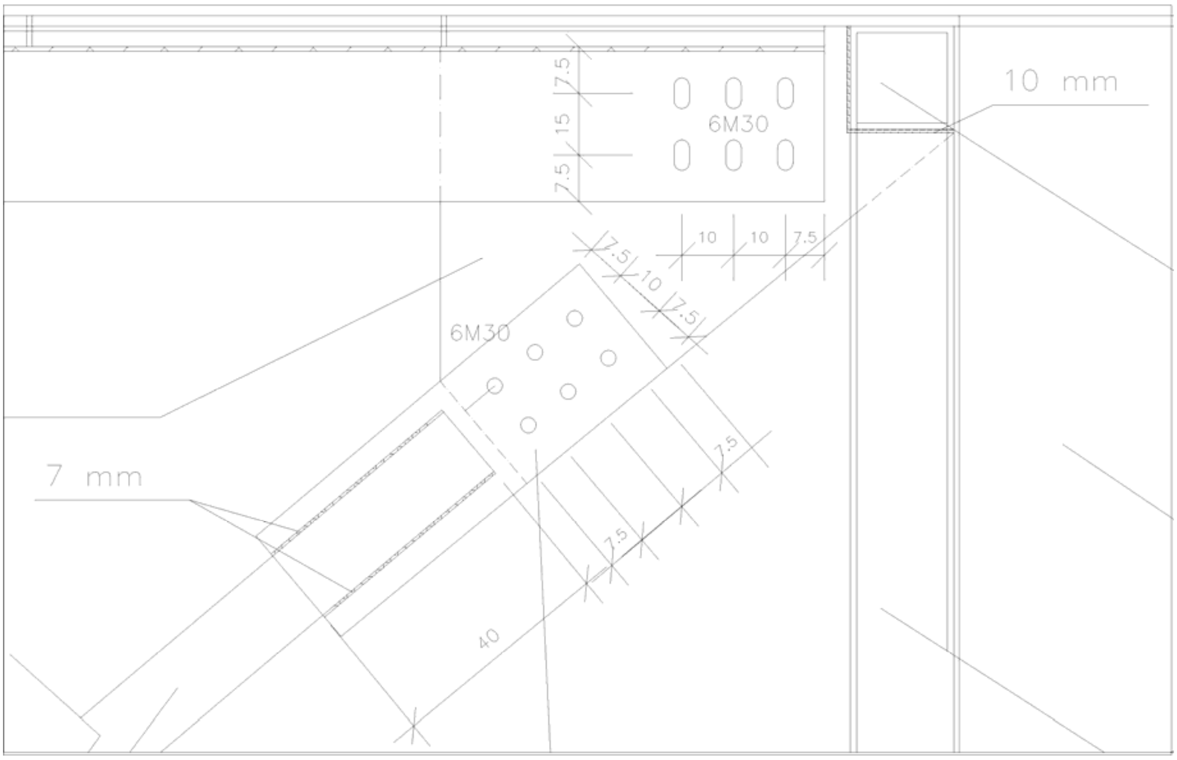

Figure 8.15. Enlarged view of the connection detail (X-Z)

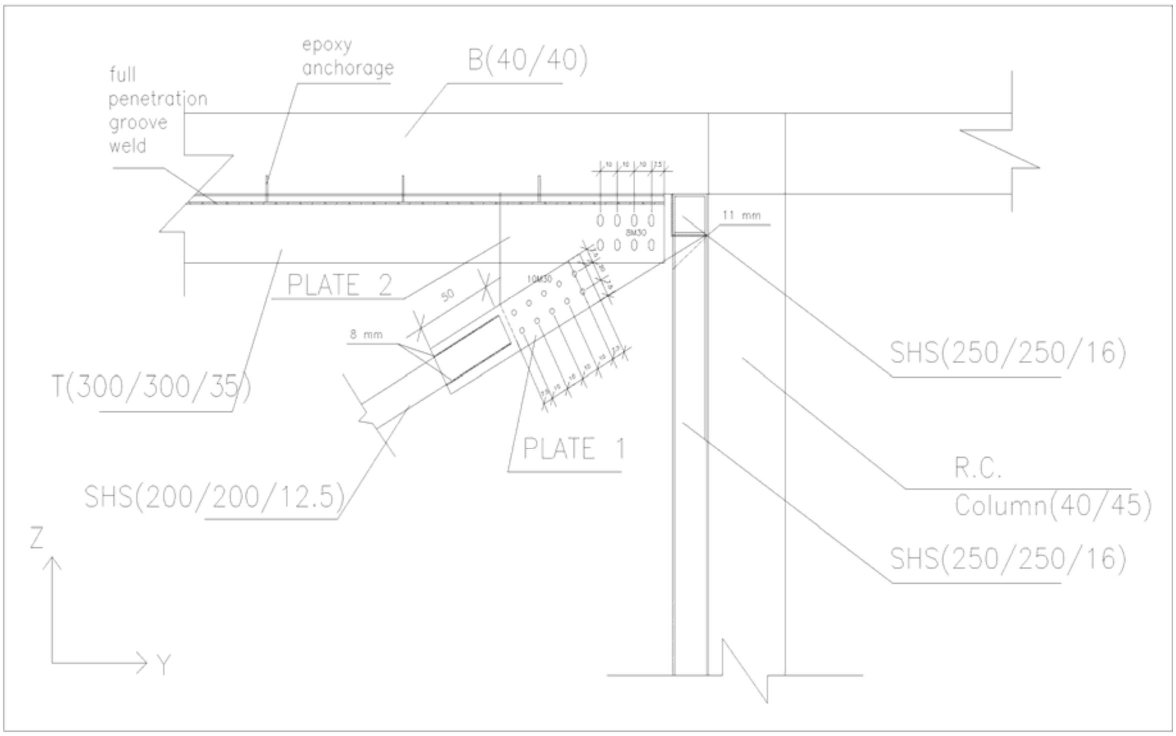

Figure 8.16. Side view for the connection detail (Y-Z)

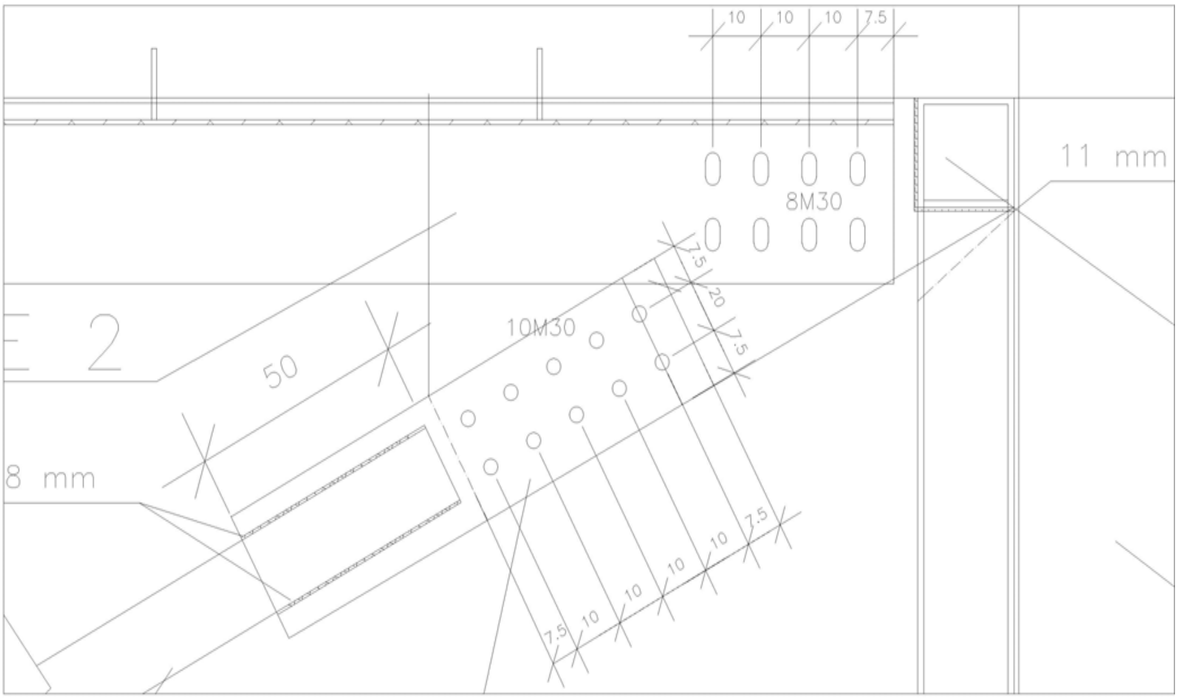

Figure 8.17. Enlarged view of the Connection Detail (Y-Z)

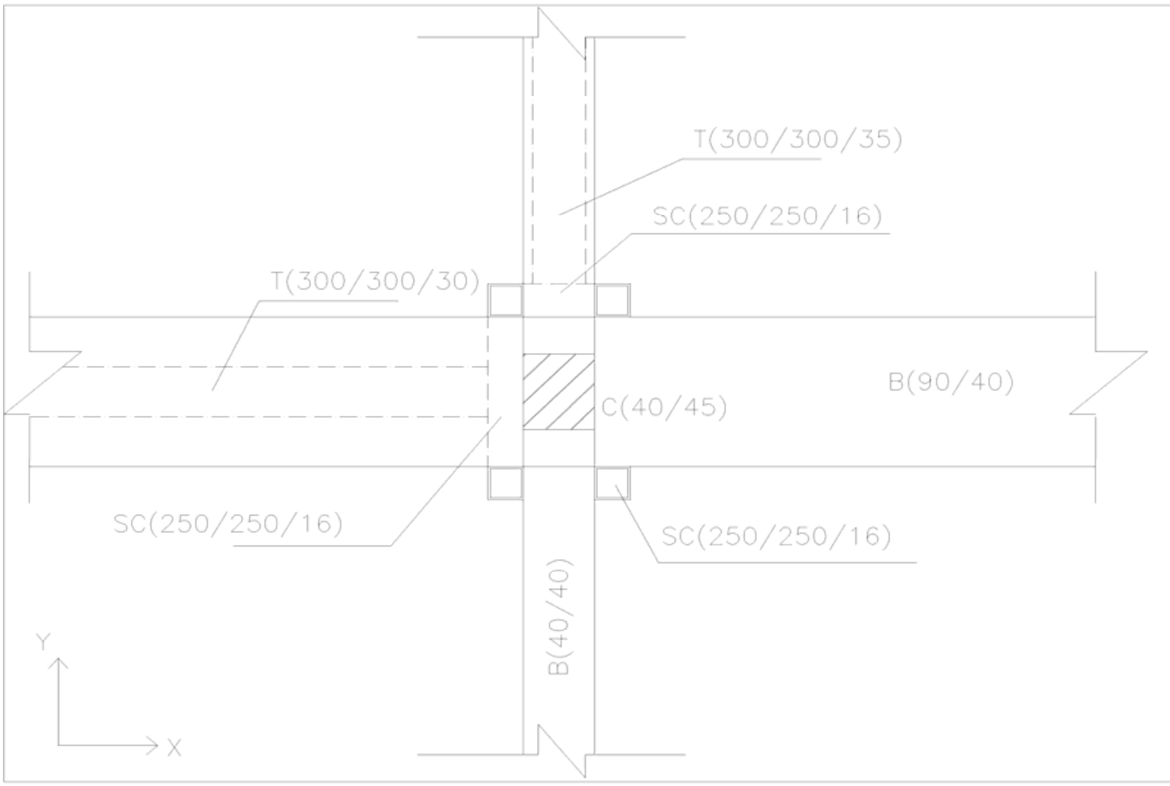

Figure 8.18. Plan view of the connection detail (X-Y)

8.8. Cost and Construction Time Analysis

As it can be seen in the previous sections, retrofit strategies shear walls and steel braces both make the building satisfy the desired performance levels and improve the strength and stiffness of the system. For the case of strategy selection it can be said that the factor of seismic performance is approximate for the two strategies although the effectiveness of them is not same.

Compared with shear walls, steel braces provide lower levels of stiffness and strength to the structure but they add far less mass to the structure than shear walls. Another advantage of steel braces is the construction with less disruption of the building which results in a less of light and smaller effect on of traffic pattern within the structure.

The parameters to choose the convenient retrofit strategy are the cost of the project, construction time and architectural reasons, because the seismic performance factor is approximately same. The strategy selection will be done according to the importance of these factors for the building and because the fact that it is a historical building, retrofit system must be designed to preserve historical structural or non structural features in the most inconspicuous manner.

In Table 8.19, cost and construction time analysis can be seen for the addition of shear walls and steel brace. The cost of the materials added to the system is calculated by unit prices and the estimated construction times are given in the figures. For the braced frames strategy the steel frames added to avoid tensile loads are also taken into account.

Addition of the braced frames is less time consuming compared to shear walls. The main disadvantage of the braced frames strategy is the much higher cost of it compared with the shear walls as it can be seen in Table 8.19.

Table 8.199. Cost and construction time analysis for retrofit systems

Strategy	Concrete (m^3)	Unit Price (YTL/m^3)	Steel Rein. (ton)	Unit Price (YTL/ton)	Steel (ton)	Unit Price (YTL/ton)	Total Price (YTL)	Constr. Time (months)
Shear Wall	76.03	110	8.62	660	-	-	14052	10
Braced Frames	-	-	-	-	93.5	1120	104787	7

9. CONCLUSIONS AND RECOMMENDATIONS

9.1. Conclusions

In this study the building in the case study ‘Haldun Taner Theatre Hall’ is evaluated for the seismic performance levels of *Immediate Occupancy* in design earthquake (10% probability to occur in 50 years) and *Life Safety* in the biggest earthquake (2% probability to occur in 50 years). The effectiveness of the retrofit strategies addition of shear walls and addition of braced frames are determined according to the improvement of seismic capacities. The following results can be obtained from that investigation:

- The existing building does not satisfy the desired performance levels, the building is in the collapse region for the biggest earthquake (2% 50 years) and it has to be strengthened.
- Addition of shear walls provide enough lateral rigidity and stiffness to satisfy the *Immediate Occupancy* performance level for design earthquake and *Life Safety* performance level for the biggest earthquake.
- Addition of braced frames also provides enough lateral rigidity and stiffness to satisfy the *Immediate Occupancy* performance level for design earthquake and *Life Safety* performance level for the biggest earthquake.
- As a comparison for the effectiveness of these two retrofit strategies it can be said that shear walls improve the stiffness and strength of the building better regarding the initial periods and capacity curve of the retrofitted system. This opinion is reached because the steel braces are added to the system as two times of shear walls to give the same improvement to the building. This can not be an exact comparison because the locations and numbers of the added retrofit members are not exactly same but the results just give an opinion about their effectiveness.
- Addition of shear walls and steel frames both make the building satisfy the desired performance levels and to decide the most convenient retrofit strategy other factors have to be taken into account. The factors, other than the seismic performance, can be named as project cost and schedule, aesthetics and architectural constraints.

- Compared with shear walls, steel braces provide lower levels of stiffness and strength to the structure but they add far less mass to the structure than shear walls. Another advantage of steel braces is the construction with less disruption of the building which results in a less of light and smaller effect on of traffic pattern within the structure. Addition of the braced frames is less time consuming compared to shear walls. The main disadvantage of the braced frames strategy is the much higher cost of it compared with the shear walls. The strategy selection will be done according to the importance of these factors for the building and because the fact that it is a historical building, retrofit system must be designed to preserve historical structural or non structural features in the most inconspicuous manner.
- For the braced frame system selection; concentric bracing system is preferred to the eccentric bracing system; because of the fact that the axial loads on the eccentric braces create shear loads, above the shear capacities of the beams.
- During the section selection of the braces *stiffness* is a more privileged factor than the *strength* of the braces. Bigger sections with more strength carry so much axial loads that make the columns rotate more than the desired level and make the columns carry axial loads more than their capacities which lead some deformation problems because of tension or compression.
- For the concentric bracing system the main disadvantage is that the tension loads over the tensile capacity of the columns occur on them which lead to non ductile axial load deformations of columns. The only convenient precaution to carry the extra tension loads, adding a steel frame system, is a very expensive and laborious way to retrofit the building. Also some assumptions are made as the added steel frame system will work properly with the existing reinforced concrete system.

9.2. Recommendations

As a result of this study some recommendations are obtained as a suggestion and they are listed below:

- As a good trial the shear walls located at the first and second stories of the building but this condition make the displacements for the first mode of the system like that, first and second stories depart to the positive direction while the third story displaces negatively. The reason for such a mode shape is that the mass and the stiffness of first and second stories are so much bigger than the top storey. And the problem is that such a mode shape can not be analyzed with the incremental equivalent seismic load method by the SAP2000. For this occasion the mode superposition method can be tried to solve this problem with different software.

- The Mander material model is used for the definition of plastic hinges and these hinges are assumed as perfectly elastoplastic. This idealization procedure may not be essential for the retrofit study; different material models and plastic hinges with the addition of strain hardening can be used.
- The strength of the materials are reduced with 0.75 because of the limited information level of the building. The modulus of elasticity of the concrete can also be reduced because of the creep effect, while evaluating the performance level of the building.
- For this study it has to be kept in mind that, there is not any section in the current code TEC 2007 that exactly describes the retrofit strategies but this study gives an idea to estimate the effectiveness of the retrofit strategies by providing the desired building performance levels under different target performance demands with the incremental equivalent seismic load method.

APPENDIX A: DETERMINATION OF THE CRACKED SECTION STIFFNESS OF COLUMNS

Table A.1. Linear interpolation of cracked section stiffness (Storey 1)

Frame	N (kN)	Section	Area	Ratio	Storey1 $(EI)_0$
48	-214.91	C50X50	0.25	0.09	0.40
49	-380.69	C50X50	0.25	0.15	0.47
50	-375.76	C50X50	0.25	0.15	0.47
51	-360.47	C50X50	0.25	0.14	0.46
52	-522.48	C50X50	0.25	0.21	0.55
53	-522.48	C50X50	0.25	0.21	0.55
54	-360.47	C50X50	0.25	0.14	0.46
55	-375.76	C50X50	0.25	0.15	0.47
56	-380.69	C50X50	0.25	0.15	0.47
57	-214.91	C50X50	0.25	0.09	0.40
58	-360.23	C40X45	0.18	0.20	0.53
59	-672.42	C40X45	0.18	0.37	0.76
60	-659.03	C40X45	0.18	0.37	0.75
61	-630.35	C40X45	0.18	0.35	0.73
62	-928.20	C40X45	0.18	0.52	0.80
63	-928.20	C40X45	0.18	0.52	0.80
64	-630.35	C40X45	0.18	0.35	0.73
65	-659.03	C40X45	0.18	0.37	0.75
66	-672.42	C40X45	0.18	0.37	0.76
67	-360.23	C40X45	0.18	0.20	0.53
68	-194.03	C40X60	0.24	0.08	0.40
69	-359.93	C40X60	0.24	0.15	0.47
70	-350.05	C40X60	0.24	0.15	0.46
71	-327.54	C40X60	0.24	0.14	0.45
72	-492.69	C40X60	0.24	0.21	0.54
73	-492.69	C40X60	0.24	0.21	0.54
74	-327.54	C40X60	0.24	0.14	0.45
75	-350.05	C40X60	0.24	0.15	0.46
76	-359.93	C40X60	0.24	0.15	0.47
77	-194.03	C40X60	0.24	0.08	0.40

Table A.2. Linear interpolation of cracked section stiffness (Storey 2)

Frame	N (kN)	Section	Area	Ratio	Storey1 $(EI)_0$
172	-111.25	C50X50	0.25	0.03	0.40
174	-200.61	C50X50	0.25	0.06	0.40
176	-197.42	C50X50	0.25	0.06	0.40
178	-186.91	C50X50	0.25	0.06	0.40
180	-276.42	C50X50	0.25	0.09	0.40
182	-276.42	C50X50	0.25	0.09	0.40
184	-186.91	C50X50	0.25	0.06	0.40
186	-197.42	C50X50	0.25	0.06	0.40
188	-200.61	C50X50	0.25	0.06	0.40
190	-111.25	C50X50	0.25	0.03	0.40
192	-193.96	C40X45	0.18	0.08	0.40
194	-361.47	C40X45	0.18	0.16	0.48
196	-354.12	C40X45	0.18	0.16	0.48
198	-336.90	C40X45	0.18	0.15	0.47
200	-498.88	C40X45	0.18	0.23	0.58
202	-498.88	C40X45	0.18	0.23	0.58
204	-336.90	C40X45	0.18	0.15	0.47
206	-354.12	C40X45	0.18	0.16	0.48
208	-361.47	C40X45	0.18	0.16	0.48
210	-193.96	C40X45	0.18	0.08	0.40
212	-98.48	C30X40	0.12	0.05	0.40
214	-187.73	C30X40	0.12	0.12	0.42
216	-181.83	C30X40	0.12	0.11	0.42
218	-168.35	C30X40	0.12	0.10	0.40
220	-257.95	C30X40	0.12	0.17	0.49
222	-257.95	C30X40	0.12	0.17	0.49
224	-168.35	C30X40	0.12	0.10	0.40
226	-181.83	C30X40	0.12	0.11	0.42
228	-187.73	C30X40	0.12	0.12	0.42
230	-98.48	C30X40	0.12	0.05	0.40

Table A.3. Linear interpolation of cracked section stiffness (Storey 3)

Frame	N (kN)	Section	Area	Ratio	Storey1 $(EI)_0$
125	-16.46	C30X40	0.12	0.01	0.40
126	-25.12	C30X40	0.12	0.01	0.40
127	-25.12	C30X40	0.12	0.01	0.40
128	-25.12	C30X40	0.12	0.01	0.40
129	-32.66	C30X40	0.12	0.02	0.40
130	-32.66	C30X40	0.12	0.02	0.40
131	-25.12	C30X40	0.12	0.01	0.40
132	-25.12	C30X40	0.12	0.01	0.40
133	-25.12	C30X40	0.12	0.01	0.40
134	-16.46	C30X40	0.12	0.01	0.40
135	-25.84	C30X40	0.12	0.02	0.40
136	-43.90	C30X40	0.12	0.03	0.40
137	-43.90	C30X40	0.12	0.03	0.40
138	-43.90	C30X40	0.12	0.03	0.40
139	-59.60	C30X40	0.12	0.04	0.40
140	-59.60	C30X40	0.12	0.04	0.40
141	-43.90	C30X40	0.12	0.03	0.40
142	-43.90	C30X40	0.12	0.03	0.40
143	-43.90	C30X40	0.12	0.03	0.40
144	-25.84	C30X40	0.12	0.02	0.40

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