DETAILED ASSESSMENT STUDY AND CONCEPT RETROFIT DESIGN OF EXISTING TYPICAL RC BUILDINGS IN TURKEY BY ECCENTRICALLY STEEL BRACED FRAMES

by
Rahman SHAHSHENAS
B.S., in C.E., Boğaziçi University, 2006

Submitted to the Institute for Graduate Studies in
Science and Engineering in partial fulfillment of
the requirements for the degree of
Master of Science

Graduate Program in Civil Engineering
Boğaziçi University
2009

ACKNOWLEDGEMENTS

Many people have contributed to shape the content and structure of this thesis. It is therefore my great pleasure to thank those who contributed and supported me in the process to write this thesis.

Especially, I would like to express my gratitude to my thesis supervisor, Prof. Gülay Altay for her guidance, support and encouragement throughout the preparation of this thesis. Without her sympathy, patience and guidance, the accomplishment would be impossible.

I would also like to thank Prof. Cavidan Yorgun and Assist Prof. Kutay Orakçal for their kind and supportive attitude to me and showing high interests and valuable advices to my thesis.

I am grateful to my parents for the endless support they have given me throughout my life. I would like to express my gratefulness to my brother Jalal Shahshenas without whom probably I would have never done this work.

I can not find the proper words to thank my colleague and dear friends; Esra Mete Güneysi, Onur Kayaturan, Özgür Yöney, Benal Mutlu, Birkan Yüksel, and, Erdal Gökgöz for their generous helps and supports during my thesis.

ABSTRACT

DETAILED ASSESSMENT STUDY AND CONCEPT RETROFIT DESIGN OF EXISTING TYPICAL RC BUILDINGS IN TURKEY BY ECCENTRICALLY STEEL BRACED FRAMES

The seismic performance of selected typical exciting RC buildings in Turkey retrofitted by eccentric steel braces is investigated. Briefly, non-ductile reinforced concrete (RC) buildings have been classified into low-rise (4 story), mid-rise (7 story) and high-rise (10 story) buildings. Lateral resisting contribution of the eccentric steel bracing in rehabilitating the building has been examined. The effect of steel bracing distributed over the height of the RC frame on the seismic performance of the rehabilitated building has been studied.

In terms of estimating the actual behavior of the buildings during the seismic activity, to evaluate seismic performance levels of the existing buildings two procedures have been selected from FEMA 356; “Linear Dynamic Analysis” and “Nonlinear Static Analysis”. The performance of the building is evaluated in terms of story drifts and damage indices. The aim of this study is making detailed assessment of existing buildings and introducing eccentrically braced frames as an effective retrofit strategy.

In the case study, unlike most of the rehabilitation methods which aim to locate shear walls in the building, implementing eccentric steel braces into RC buildings. Although there are some studies and even applications of adding EBF and concentric braced frames (CBF) systems into RC buildings, especially in these studies where EBF have been used as retrofit strategy, link element has been designed vertically and out of RC framing system. In this study link element has been modeled and designed as a box section around RC beam, for further studies it is recommended that link element should be RC beam instead of steel beam.

Desirable performance level on the buildings is life safety level and for obtaining this performance level various brace configurations have been analyzed. For controlling the retrofit strategy, pushover procedure has been applied on 2D frame systems to determine the seismic demand and the target displacements; the pushover of the model has been obtained, then it has been converted to modal capacity curve and it has been compared with the earthquake demand in capacity spectrum method in the same diagram. The seismic performance evaluation of the buildings has been done for one direction under 10% in 50 years target performance level according to FEMA 356.

ÖZET

DETAYLI DEĞERLENDİRME ÇALIŞMALARI VE TÜRKİYE’DE Kİ MEVCUT TİPİK BİNALARIN ÇELİK DIŞ MERKEZ ÇERÇEVELER İLE GÜÇLENDİRİLMESİ

Dış merkezli çelik çapraz ile güçlendirilmiş tipik mevcut betonarme binanın sismik performansı araştırılmıştır. Sünek olmayan betonarme binalar kısaca alçak (4 kat), orta (7 kat) ve çok katlı (10 kat) binalar olarak sınıflandırılmıştır. Binaların iyileştirilmesinde dış merkezli çelik çaprazların yanal dayanıma katkısı incelenmiştir. Çelik çaprazların betonarme çerçeve yüksekliği boyunca dağılımının iyileştirilmiş binanın sismik performansı üzerindeki etkisi araştırılmıştır.

Sismik aktiviteler sırasında yapının gerçek davranışının hesaplanması için FEMA 356’dan “Doğrusal Dinamik Analiz” ve “Doğrusal Olmayan Statik Analiz” olmak üzere iki adet prosedür seçilmiştir. Binanın performansı kat kayması ve hasar belirtisi açısından hesaplanmıştır. Bu çalışmanın amacı; mevcut binanın detaylı bir şekilde değerlendirmesinin yapılması ve dış merkezli çapraz çerçevelerin etkin güçlendirme stratejisi olarak sunulmasıdır.

Durum çalışmasında, binaya perde duvar eklenmesini içeren birçok iyileştirme metoduna karşılık, betonarme binaya dış merkezli çelik çaprazların uygulanması denenmiştir. Betonarme binalarda dış merkezli çapraz çerçevelerin ve eş merkezli çapraz çerçevelerin uygulanması üzerinde bazı araştırmalar yapılmış olsada, özellikle dış merkezli çapraz çerçevelerin güçlendirme stratejisi olarak ele alındığı çalışmalarda bağlantı elemanı dikey ve betonarme çerçevenin dışında tasarlanmıştır. Bu çalışmada bağlantı elemanı betonarme kiriş çevresinde kutu kesit olarak tasarlanmıştır. İlave çalışmalara bağlantının çelik kiriş değil betonarme kiriş olarak tasarlanmasını önermektedir.

Binada istenilen performans seviyesi can güvenliği seviyesidir ve bu performans seviyesinin elde edilmesi için çeşitli çapraz düzeni analiz edilmiştir. Güçlendirme stratejisini kontrol etmek amacıyla, sismik gereksinimim ve hedef deplasmanın belirlenmesi için 2 boyutlu çerçeve sistemde itme prosedürü uygulanmıştır. Sonrasında spektral ivme-spektral deplasman diyaframına çevrilmiştir ve aynı diyaframda sismik gereksinim ile karşılaştırılmaştır. Binanın sismik performans değerlendirmesi FEMA 356'ya gore 50 yıllık hedef performans seviyesinin %10'unun altında bir doğrultuda gerçekleştirilmiştir.

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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 i^{th} 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 i^{th} 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 i^{th} 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 ith storey of building
Hi	Height of ith storey of building measured from the top foundation level
h	Effective height of the section
h_i	Height of ith 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	i^{th} storey mass 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 ith 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 ith pushover step along x direction corresponding to first mode
${V_{yN1}}^{(i)}$	Base shear force in ith pushover step along y direction corresponding to first mode
W	Total building weight, Elastic section modulus
W_i	Total weight of ith story of the building including live loads multiplied by related participation factors
w	Buckling coefficient
w_i	Total weight of ith 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
CG	Life Safety Level
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
SAP	Structural Analysis Program
SDOF	Single Degree of Freedom
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

1. INTRODUCTION

1.1. General Considerations

A significant portion of Turkey is subject to frequent earthquakes, most significantly from the North Anatolian Fault Zone, which stretches across the country and is responsible for many of Turkey’s largest historical earthquakes. Turkey ranks high among countries that have suffered centuries of loss of life and property due to earthquakes. In the twentieth century, earthquakes in Turkey caused over 110,000 deaths, 250,000 hospitalized injuries and the destruction of 600,000 housing units. Following the losses suffered during two major earthquakes in 1999, there is broad recognition among government, non-government and academic organizations that extensive response planning is needed in Turkey. Such planning should be based upon detailed risk analyses of likely seismic hazards in Turkey, particularly in and around Istanbul.

In recent decades, earthquake disaster risk for Turkey’s urban centers has increased, mainly due to high rates of urbanization, faulty land-use planning and construction, inadequate infrastructure and services, and environmental degradation. Several studies (Erdik, 2007) have shown that the vulnerability of Turkish building stock is higher than that in California, which shares a comparable level of earthquake hazard. Reasons for this high vulnerability can be traced to several reasons. Poor quality residential construction and development is the result of a high (chronic) rate of inflation (leading to a limited mortgage and insurance market, a major impediment to large scale development and industrialization of the construction sector), excessive urbanization (which created the demand for inexpensive housing), ineffective control/supervision of design and construction, regulations with limited enforcement mechanisms, a lack of accountability.

For Istanbul, a worst-case scenario earthquake of magnitude 7.5 is assumed to take place in the Main Marmara Fault. Figure 1 provides a map of earthquake intensities that would result from the scenario earthquake. Using the damage definitions of 1998 European Macro seismic Scale, a general picture of damage under exposure to these intensity levels can be gained. For the vulnerability class where the general reinforced concrete multistory building stock in Istanbul is located, EMS-1998 provides the following damage definitions:

- Intensity VII: A few buildings sustain moderate damage
- Intensity VIII: Many buildings suffer moderate damage; a few receive substantial to heavy damage
- Intensity IX: Many buildings suffer substantial to heavy damage; a few sustain very heavy damage where “few” describes less than 20 percent and “many” describe 20 percent to 60 percent (Erdik, 2007).

After browsing summarized studies done upon different scenarios of occurring the earthquake and the loss coming up after that, taking precautions in order to bring down the loss as much as possible is inevitable.

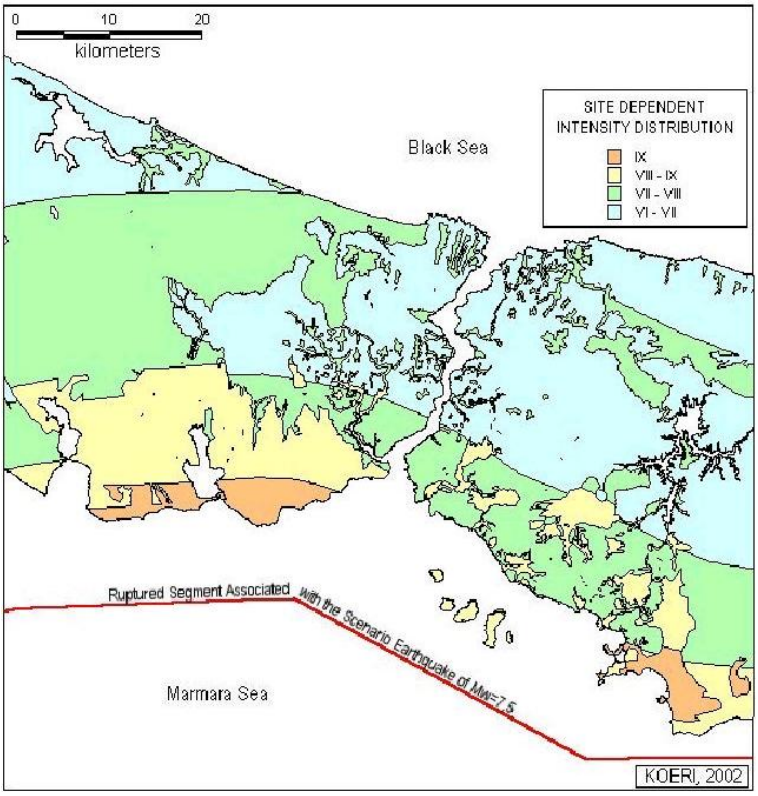


Figure 1.1. Site dependent deterministic intensity distribution (Erdik, 2007)

1.2. Scope and Objective

In this study, the influence and contribution of eccentrically steel braced frames in existing reinforced concrete buildings has been investigated according to FEMA 356. In this study, the buildings which were designed according to the force based approaches are evaluated with the performance based design according to FEMA 356 for determining the seismic performance level. Most of the existing buildings in Istanbul are old and have been designed and constructed according to old-time standards. Even during design and construction of such building all necessary conditions have not been complied. For having a detailed observation upon the current situation of such buildings this study have been carried out.

The first step of the study was to classify the existing buildings in Istanbul. As a matter of fact there are many old-time structures built by reinforced concrete, steel, wood and even masonry which are still operating as resident or Industrial meanings. This study covers only RC buildings. The existing RC buildings are of different height, various numbers of stories, different area and many other differences from architectural and structural planning. Based on the main concept to be after in this study typical buildings were selected and the selected items do not represent real projects or real existing buildings. The following cases were taken into account before selecting the building:

- Selected buildings represent the current situation of the buildings as much as possible
- The worst conditions from the points of soil, foundation and seismic region were considered.
- Realistic and reasonable value of material strengths were selected
- Based on the availability of original design data, following scheme is adopted to conduct the assessment of the buildings.
- Geological Desktop Study, all buildings have been assumed to be on the soil classification D.
- Visual inspection of the buildings in Turkey.

- Specification of materials testing on field and/or laboratory examination, this part also has been assumed as C16 for concrete and S220 for the steel used in buildings.
- Detailed Assessment.

Overall, 3 types of the buildings were selected as low-rise, mid-rise and high-rise buildings. The number of story for low-rise is 4, for mid-rise is 7 and for the high-rise is 10. The floor area of these buildings are the same, in other consideration the effect of gravity on the behavior of EBF systems has been studied.

In order for clear understanding of the future analysis and computations, the existing buildings have been named as R00, R is an abbreviation of retrofit and the numbers coming after that represents the phase and the stage of retrofitting.

At the second step of the study, selected typical buildings type were designed accordance with Turkish old-time design standard in 1975. Most of existing buildings in Istanbul have been designed according to this code. Taken into consideration that even after 1975 the building design standard, most of the designers and contractors did not go along with the conditions stated in that code, most probably due to not being aware of the hazard and the risk of catastrophic events and consequences.

The selected models were designed in accordance with 1975 design code and creep and fatigue factors were considered on the structures due to passing long time of construction. During design it was tried to keep the symmetry and typical situations on the buildings as much as possible. At the same time it was attempted to gain the most realistic prototypes which reflect the current situation of existing buildings as much as possible.

The low, mid and high rise buildings have been assessed in detail according to federal emergency management agency code FEMA 356. The assessment procedures could be seen in following figure the target performance for the buildings is life safety, in order for that all buildings have been analyzed and assessed for this purpose.

At the third step of the study, as a starting point a preliminary configuration of eccentrically braced frames was arranged and applied to each end bay of the buildings. At each end bay only one EBF systems was placed at all story levels. The details of selected EBF like as link and others were put to be done at detailed design part. With taken into account a rescannable link and brace system were modeled and after supplying logical and enough amount of demand capacity, the sections were designed.

At the fourth step of the study, for having an idea of other retrofit methods and also a scale for comparing the EBF method, in the same configuration and same locations concentrically steel braced frames were modeled on an alternative model. The same design procedure has been done on the model retrofitted by CBF as EBF model. From comparison result, the way of progressing was highlighted for future phases of the study. According to many resources it had been observed that EBF systems are more efficient rather than CBF systems against lateral loads, however in the concept of this study it should have been approved by investigating and modeling results.

At fourth step the results of two models were compared and a further step, fifth step, was considered to attempt an alternative brace configuration of EBF. At this stage at each end bay two EBF systems were located at all stories. The reason and intelligence behind adding EBF or CBF systems at the end bays of the structures is due to restriction of architectural planning. No contractor or owner would agree to disorder the schema of the building; likewise this matter is more crucial while dealing with industrial buildings. The analysis results of fifth step showed more filtered results for progressing.

At sixth step of the study, the selected model at fifth step was optimized and a complete retrofit design was carried out. When a building is undergoing the objective of retrofitting, only a unique method of retrofitting is not used, usually a combination of some strategies is taken into account. Thus at sixth step, detailed retrofit concept design has been done on the model in combination of EBF systems, jacketing the relevant columns and locating steel plates at link as a fuse against lateral loads.

At the final step of the study nonlinear static push over analysis has been done on the models for controlling that buildings supply required performance level, life safety.

The nonlinear applications of the SAP 2000 V11.0.0 structural analysis program and the section analysis program XTRACT is used for these analyses. Push over analysis has been done according to FEMA356 coefficient method and base shear vs. displacement of the structures has been selected as scale of comparing.

Scope of the study is defined as the assessment of the structural risk-to-life-safety and the risk associated with the egress routes to the accepted levels of Life Safety as defined in FEMA 356, in order to re-classify or to retrofit the low, mid and high rise structures by EBF systems so that they can be re-classified in the Low Risk category according to the aim of the project.

This study addresses the preliminary assessment of the buildings for a design earthquake with a 10% probability of occurrence in 50 years. Analyses done and results reported in the study are related only to the structural components of the buildings, and do not include the aspects related to the safety of equipment, installations or cladding to be attached to or supported by the structural components. Therefore, investigating the condition of non-structural elements, installations and equipment housed in the structures after the event of an earthquake have not been included in the study.

2. THE DETAILED ASSESSMENT STUDY

2.1. The Assessment Processes

Seismic rehabilitation of an existing building shall be conducted. In the study the highlighted steps have been taken into consideration. Before embarking on a rehabilitation design, seismic deficiencies shall have been identified through a prior seismic evaluation performed using an evaluation methodology considering a combination of building performance and seismic hazard.

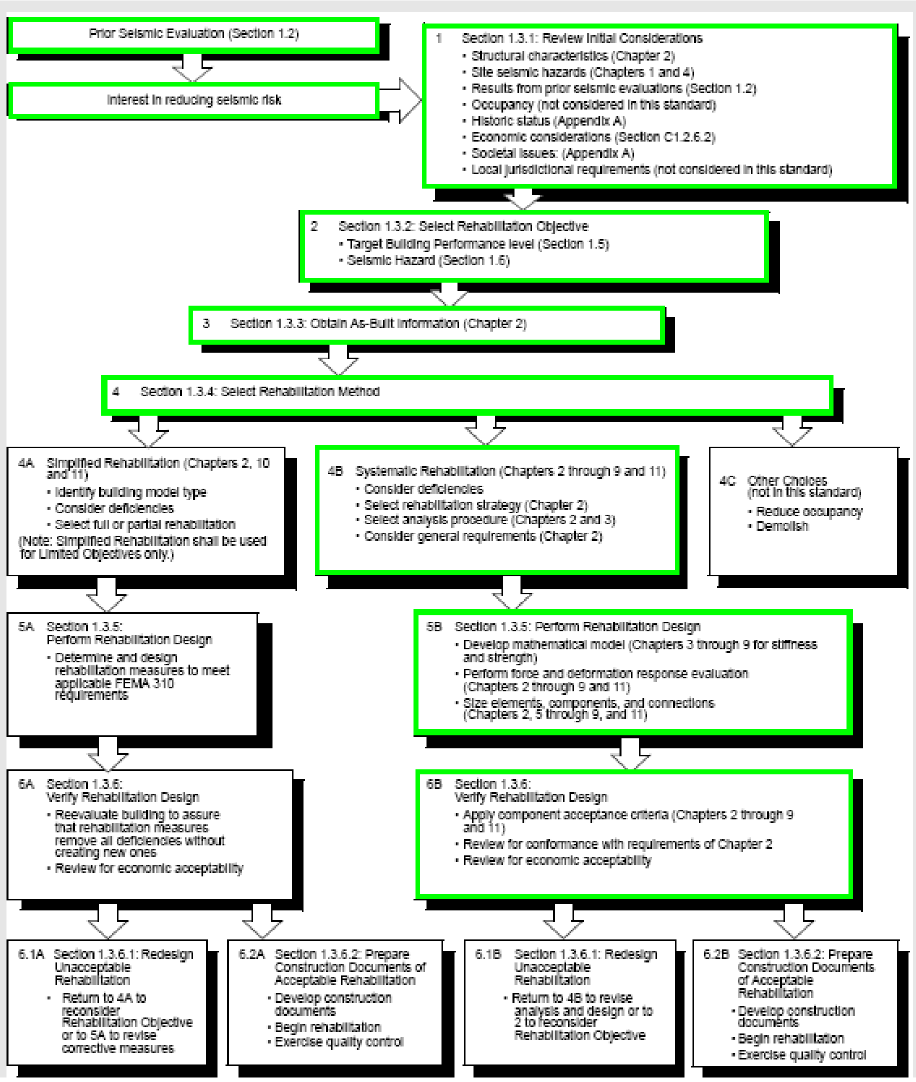


Figure 2.1. The evaluation methodology in the study to be followed (FEMA 356)

2.1.1. Selecting the Objective of Retrofitting

The building owner or code official shall select a seismic Rehabilitation Objective for the building as specified in FEMA 356. However this study is not a real life project so necessary assumption regarding the objectives have been made. The selection of a Rehabilitation Objective shall consist of the selection of a target Building Performance Level from a range of performance levels defined in Section 1.5 and on the selection of an anticipated Earthquake Hazard Level from a range of seismic Obtaining As-Built Information.

Available as-built information for the building shall be obtained and a site visit shall be conducted as specified in FEMA 356, necessary assumptions have been made for as-built information on low, mid and high rise structures in this study.

2.1.2. Verifying Rehabilitation Concept Design

The rehabilitation concept design shall be verified to meet the requirements of FEMA 356 through an analysis of the building, including rehabilitation measures. The analysis shall be consistent with the procedures for the applicable rehabilitation method specified in Section 2.3. A separate analytical evaluation shall be performed for each combination of building performance and seismic hazard specified in the selected Rehabilitation Objective.

2.1.3. Repeating the Processes and Redesign Unacceptable Rehabilitation

If the rehabilitation design fails to comply with the acceptance criteria for the selected Rehabilitation Objective, the rehabilitation measures shall be redesigned or an alternative rehabilitation strategy shall be implemented. This process shall be repeated until the design is in compliance with the acceptance criteria for the selected Rehabilitation Objective.

2.1.4. Rehabilitation Objectives

Recommendations regarding the selection of a Rehabilitation Objective for any building are beyond the scope of FEMA 356. FEMA 274 discusses issues to consider when combining various performance and seismic hazard levels. It should be noted that not all combinations constitute reasonable or cost-effective Rehabilitation Objectives. FEMA 356 is written under the premise that greater flexibility is required in seismic rehabilitation than in the design of new buildings. However, given that flexibility, once a Rehabilitation Objective is selected, FEMA 356 provides internally consistent procedures with the necessary specificity to perform a rehabilitation analysis and design. Building performance can be described qualitatively in terms of the safety afforded building occupants during and after the event; the cost and feasibility of restoring the building to pre-earthquake condition; the length of time the building is removed from service to effect repairs; and economic, architectural, or historic impacts on the larger community. (Ghobarah *et al.,* 2003) These performance characteristics are directly related to the extent of damage that would be sustained by the building. In FEMA 356, the extent of damage to a building is categorized as a Building Performance Level. A broad range of target Building Performance Levels may be selected when determining Rehabilitation Objectives. Probabilistic Earthquake Hazard Levels frequently used in FEMA 356and their corresponding mean return periods (the average number of years between events of similar severity) are as follows:

Table 2.1. Mean return period of various EQ dependent on the exceedance

Earthquake Having Probability of Exceedance	Mean Return Period (years)
50% per 50 year	72
20% per 50 year	225
10% per 50 year	474
2% per 50 year	2475

These mean return periods presented in table in the above are typically rounded to 75, 225, 500, and 2500 years, respectively.

The Rehabilitation Objective selected as a basis for design will determine, to a great extent, the cost and feasibility of any rehabilitation project, as well as the benefit to be obtained in terms of improved safety, reduction in property damage, and interruption of use in the event of future earthquakes. Table below indicates the range of Rehabilitation Objectives that may be used in this standard.

Table 2.2. The range of rehabilitation objectives

	Earthquake Hazard Level (per 50 year)			
Target Building Performance Levels	*50%*	*20%*	*10%*	*2%*
Operational Performance Level (1-A)	a	e	i	m
Immediate Occupancy Performance Level (1-B)	b	f	j	n
Life Safety Performance Level (3-C)	c	g	k	o
Collapse Prevention Performmnce Level (5-E)	d	h	l	p

Each cell in the above matrix represents a discrete Rehabilitation Objective. The Rehabilitation Objectives in the matrix above may be used to represent the three specific Rehabilitation Objectives

- k + p = Basic Safety Objective
- k + p + any of a, e, i, b, f, j, or n = Enhanced Objectives. alone or n alone or m alone = Enhanced Objective
- k alone or p alone = Limited Objectives. c, g, d, h, l = Limited Objectives

2.2. Target Building Performance Levels

Building performance is a combination of the performance of both structural and nonstructural components. Table 2.2 describes the approximate limiting levels of structural and nonstructural damage that may be expected of buildings rehabilitated to the levels defined in the standard. On average, the expected damage would be less.

Performance descriptions in Table 2.2 are estimates rather than precise predictions, and variation among buildings of the same target Building Performance Level must be expected. Building performance in FEMA 356 is expressed in terms of target Building Performance Levels. These target Building Performance Levels are discrete damage states selected from among the infinite spectrum of possible damage states that buildings could experience during an earthquake. The particular damage states identified as target Building Performance Levels in FEMA 356 have been selected because they have readily identifiable consequences associated with the post-earthquake disposition of the building that are meaningful to the building community. These include the ability to resume normal functions within the building, the advisability of post earthquake occupancy, and the risk to life safety. Due to inherent uncertainties in prediction of ground motion and analytical prediction of building performance, some variation in actual performance should be expected.

2.2.1. Structural Performance Levels and Ranges

The Structural Performance Level of a building shall be selected from four discrete Structural Performance Levels and two intermediate Structural Performance Ranges defined in this section. The discrete Structural Performance Levels are Immediate Occupancy (S-1), Life Safety (S-3), Collapse Prevention (S-5), and Not Considered (S-6). Design procedures and acceptance criteria corresponding to these Structural Performance Levels would be as specified in Chapters 4 through 10. The intermediate Structural Performance Ranges are the Damage Control Range (S-2) and the Limited Safety Range (S-4). Acceptance criteria for performance within the Damage Control Structural Performance Range shall be obtained by interpolating the acceptance criteria provided for the Immediate Occupancy and Life Safety Structural Performance Levels. Acceptance criteria for performance within the Limited Safety Structural Performance Range shall be obtained by interpolating the acceptance criteria provided for the Life Safety and Collapse Prevention Structural Performance Levels (Moehle, 1991).

2.2.2. Life Safety Structural Performance Level (S-3)

Structural Performance Level S-3, Life Safety, shall be defined as the post-earthquake damage state that includes damage to structural components but retains a margin against onset of partial or total collapse in compliance with the acceptance criteria specified in FEMA 356 for this Structural Performance Level.

Structural Performance Level S-3, Life Safety, means the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, the overall risk of life-threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure; however, for economic reasons this may not be practical. While the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to re-occupancy.

2.2.3. Seismic Hazard

Seismic hazard due to ground shaking shall be based on the location of the building with respect to causative faults, the regional and site-specific geologic characteristics, and a selected Earthquake Hazard Level. Seismic hazard due to ground shaking shall be defined as acceleration response spectra or acceleration time-histories on either a probabilistic or deterministic basis.

2.2.4. Response Acceleration Parameters

The design short-period response acceleration parameter, S_{XS}, and design response acceleration parameter at a one-second period, S_{X1}, for the BSE-2 Earthquake Hazard Level shall be determined using values of S_S and S_1 taken from approved MCE spectral response acceleration contour maps and modified for site class.

2.2.5. Probabilities of Exceedance Greater than 10% per 50 Years

For probabilities of exceedance greater than 10% per 50 years and when the mapped short-period response acceleration parameter, S_S, is less than 1.5g, the modified mapped short-period response acceleration parameter, S_S, and modified mapped response acceleration parameter at a one-second period, S_1, shall be determined.

2.2.6. Adjustment for Site Class

The design short-period spectral response acceleration parameter, SXS, and the design spectral response acceleration parameter at one-second, SX1, shall be obtained respectively as follows:

$$S_{XS} = F_a S_S \tag{2.1}$$

$$S_{X1} = F_v S_1 \tag{2.2}$$

Where F_a and F_v are site coefficients determined respectively from Tables 2.3 and 2.4, based on the site class and the values of the response acceleration parameters S_S and S_1 for the selected return period.

Table 2.3. Fa values dependent on site class

Values of Fa as a Function of Site Class and Mapped Short-Period Spectral Response Acceleration Ss					
Site Class	Mapped spectral Acceleration at Short-Periods				
	Ss≤0.25	Ss=0.50	Ss=0.75	Ss=1.00	Ss≥1.25
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	*	*	*	*	*

Table 2.4. Fv values dependent on site class

Values of Fv as a Function of Site Class and Mapped Spectral Response Accelerationat One-Second Period S_1					
Site Class	Mapped spectral Acceleration at Short-Periods				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	*	*	*	*	*

2.3. General Response Spectrum

A general response spectrum shall be developed as FEMA 356, in which general horizontal response spectrum shall be developed using equations for spectral response acceleration, Sa, versus structural period, T, in the horizontal direction.

$$S_a = \begin{cases} S_{XS}\left[\left(\frac{5}{B_S}-2\right)\frac{T}{T_S}+0.4\right] & for\ \ 0 < T < T_0 \\ S_{XS}/B_S & for\ \ T < T_S \\ S_{X1}/(B_1 T) & for\ T > T_S \end{cases} \tag{2.3}$$

where T and T_s are given by:

$$T_S = S_{X1}B_S/(S_{XS}B_1) \tag{2.4}$$

$$T_0 = 0.2T_S \tag{2.5}$$

Use of spectral response accelerations calculated using (2.3) in the extreme short period range (T <T_0) shall only be permitted in dynamic analysis procedures and only for modes other than the fundamental mode. BS and B1 are taken from the table below:

Table 2.5. Damping coefficients

Damping Coefficients BS and B1 as a Function of Effective Damping β		
Effective Viscous Damping β	B_S	B_1
≤2	0,80	0,80
5	1,00	1,00
10	1,30	1,20
20	1,80	1,50
30	2,30	1,70
40	2,70	1,90
≥50	3,00	2,00

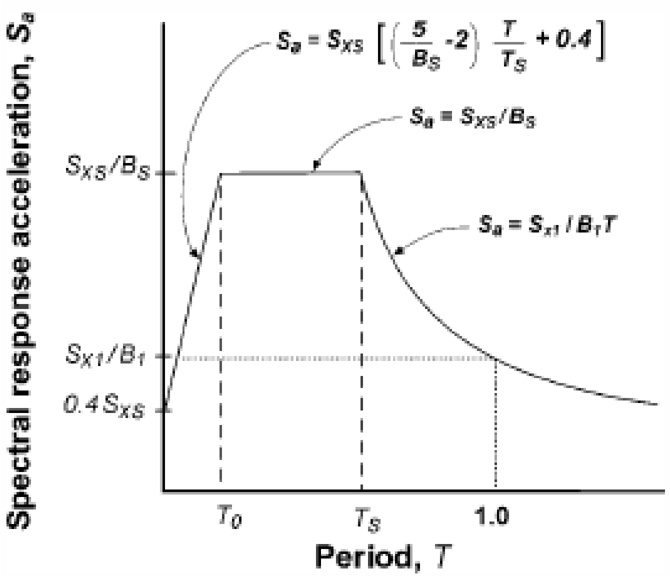


Figure 2.2. FEMA 356 response spectrum curve (FEMA 356)

2.3.1. Minimum Spectral Amplitude

The 5% damped site-specific spectral amplitudes in the period range of greatest significance to the structural response shall not be specified less than 70 percent of the spectral amplitudes of the General Response Spectrum.

2.3.2. Zones of Seismicity

The zone of seismicity shall be defined as High, Moderate or Low. The definition of this classification is as the follow:

2.3.3. Zones of High Seismicity

Buildings shall be considered to be located within zones of high seismicity where the 10%/50 year, design short period response acceleration, S_{XS}, and the 10%/50 year design one-second period response acceleration, S_{X1}, are as:

$$S_{XS} \geq 0.5g \tag{2.6}$$

$$S_{X1} \geq 0.2g \tag{2.7}$$

2.3.4. Zones of Moderate Seismicity

Buildings shall be considered to be located within zones of moderate seismicity where the 10%/50 year, design short-period response acceleration, S_{XS}, and the 10%/ 50 year, design one-second period response acceleration, S_{X1}, are as specified in 2.8 and 2.9:

$$0.167 \leq S_{XS} \leq 0.5g \tag{2.8}$$

$$0.067 \leq S_{X1} \leq 0.2g \tag{2.9}$$

2.3.5. Zones of Low Seismicity

Buildings shall be considered to be located within zones of low seismicity where the 10%/50 year, design short period response acceleration, S_{XS}, and the 10%/50 year, design one-second period response acceleration, S_{X1}, are as specified in 2.10 and 2.11:

$$S_{XS} \leq 0.167g \tag{2.10}$$

$$S_{X1} \leq 0.067g \tag{2.11}$$

2.4. Analysis Procedures

An analysis of the building, including rehabilitation measures, shall be conducted to determine the forces and deformations induced in components of the building by ground motion corresponding to the selected Earthquake Hazard Level, or by other seismic geologic site hazards. The analysis procedure shall comply with one of the following analysis.

Linear analysis subject to limitations specified in Section 2.4.1, and complying with the Linear Static Procedure (LSP) in accordance with Section 3.3.1, or the Linear Dynamic Procedure (LDP) in accordance with Section 3.3.2. The linear procedures maintain the traditional use of a linear stress-strain relationship, but incorporate adjustments to overall building deformations and material acceptance criteria to permit better consideration of the probable nonlinear characteristics of seismic response.

Nonlinear analysis subject to limitations specified in Section 2.4.2, and complying with the Nonlinear Static Procedure (NSP) in accordance with Section 3.3.3, or the Nonlinear Dynamic Procedure (NDP) in accordance with Section 3.3.4. The Nonlinear Static Procedure, often called “pushover analysis,” uses simplified nonlinear techniques to estimate seismic structural deformations. The Nonlinear Dynamic Procedure, commonly known as nonlinear time history analysis, requires considerable judgment and experience to perform, and may be used only within the limitations.

Alternative rational analysis in accordance with Section 2.4.3. The analysis results shall comply with the applicable acceptance criteria.

2.4.1. Linear Procedures

Linear procedures shall be permitted for buildings which do not have an irregularity. For buildings that have one or more of the irregularities linear procedures shall not be used unless the earthquake demands on the building comply with the demand capacity ratio (DCR) requirements. The results of the linear procedures can be very inaccurate when applied to buildings with highly irregular structural systems, unless the building is capable of responding to the design earthquake(s) in a nearly elastic manner. The linear procedures are intended to evaluate whether the building is capable of nearly elastic response.

2.4.2. Limitations on Use of Linear Procedures

The determination of irregularity shall be based on the configuration of the rehabilitated structure. A linear analysis to determine irregularity shall be performed by either an LSP in accordance with Section 3.3.1 or an LDP in accordance with Section 3.3.2. The results of this analysis shall be used to identify the magnitude and uniformity of distribution of inelastic demands on the primary elements and components of the lateral-force resisting system. The magnitude and distribution of inelastic demands for existing and added primary elements and components shall be defined by demand-capacity ratios (DCRs) and computed in accordance with (2.12):

$$DCR = \frac{Q_{UD}}{Q_{CE}} \tag{2.12}$$

Where:
Q_{UD} = Force Due to the gravity and earthquake loads
Q_{CE} = Expected strength of the component or element

DCRs shall be calculated for each action (such as axial force, moment, and shear) of each primary component. The critical action for the component shall be the one with the largest DCR. The DCR for this action shall be termed the critical component DCR. The largest DCR for any element at a particular story is termed the critical element DCR at that story. If an element at a particular story is composed of multiple components, then the component with the largest computed DCR shall define the critical component for the element at that story.

The applicability of linear procedures shall be determined as follows:

- If all component DCRs < 2.0, then linear procedures are applicable.
- If one or more component DCRs exceeds 2.0, and no irregularities are present, then linear procedures are applicable.
- If one or more component DCRs exceed 2.0 and any irregularity is present, then linear procedures are not applicable, and shall not be used.

The Linear Static Procedure shall not be used for a building with one or more of the following characteristics:

- The fundamental period of the building, T, is greater than or equal to 3.5 times Ts.
- The ratio of the horizontal dimension at any story to the corresponding dimension at an adjacent story exceeds 1.4 (excluding penthouses).
- The building has a severe torsional stiffness irregularity in any story. A severe torsional stiffness irregularity exists in a story if the diaphragm above the story under consideration is not flexible and the results of the analysis indicate that the drift along any side of the structure is more than 150% of the average story drift.
- The building has a severe vertical mass or stiffness irregularity. A severe vertical mass or stiffness irregularity exists when the average drift in any story (except penthouses) exceeds that of the story above or below by more than 150%.
- The building has a nonorthogonal lateral-force resisting system.

For buildings in which linear procedures are applicable, but the Linear Static Procedure is not permitted, use of the Linear Dynamic Procedure shall be permitted.

2.4.3. Nonlinear Procedures

Nonlinear procedures shall be permitted for any of the rehabilitation strategies. Nonlinear procedures shall be used for analysis of buildings when linear procedures are not permitted. The NSP shall be permitted for structures in which higher mode effects are not significant. To determine if higher modes are significant, a modal response spectrum analysis shall be performed for the structure using sufficient modes to capture 90% mass participation. A second response spectrum analysis shall also be performed, considering only the first mode participation. Higher mode effects shall be considered significant if the shear in any story resulting from the modal analysis considering modes required for obtaining 90% mass participation exceeds 130% of the corresponding story shear considering only the first mode response. If higher mode effects are significant, the NSP shall be permitted if an LDP analysis is also performed to supplement the NSP. Buildings with significant higher mode effects must meet the acceptance criteria of FEMA 356 for both analysis procedures, except that an increase by a factor of 1.33 shall be permitted in the LDP acceptance criteria for deformation-controlled actions (m-factors). A building analyzed using the NSP, with or without a supplementary LDP evaluation, shall meet the acceptance criteria for nonlinear procedures specified in Section 3.4.3. The NSP is generally a more reliable approach to characterizing the performance of a structure than are linear procedures. However, it is not exact, and cannot accurately account for changes in dynamic response as the structure degrades in stiffness or account for higher mode effects. When the NSP is utilized on a structure that has significant higher mode response, the LDP is also employed to verify the adequacy of the design. When this approach is taken, less restrictive criteria are permitted for the LDP, recognizing the significantly improved knowledge that is obtained by performing both analysis procedures.

The acceptability of force and deformation actions shall be evaluated for each component in accordance with the requirements of Section 3.4. Prior to selecting component acceptance criteria for use in Section 3.4, each component shall be classified as primary or secondary, and each action shall be classified as deformation-controlled (ductile) or force-controlled (nonductile). Component strengths, material properties, and component capacities shall be determined. The rehabilitated building shall be provided with at least one continuous load path to transfer seismic forces, induced by ground motion in any direction, from the point of application to the final point of resistance. All primary and secondary components shall be capable of resisting force and deformation actions within the applicable acceptance criteria of the selected performance level.

2.4.4. Primary and Secondary Elements and Components

Elements and components that affect the lateral stiffness or distribution of forces in a structure, or are loaded as a result of lateral deformation of the structure, shall be classified as primary or secondary, even if they are not part of the intended lateral-force-resisting system. Elements and components that provide the capacity of the structure to resist collapse under seismic forces induced by ground motion in any direction shall be classified as primary. Other elements and components shall be classified as secondary.

2.4.5. Deformation- and Force-Controlled Actions

All actions shall be classified as either deformation controlled or force-controlled using the component force versus deformation curves shown in Figure below.

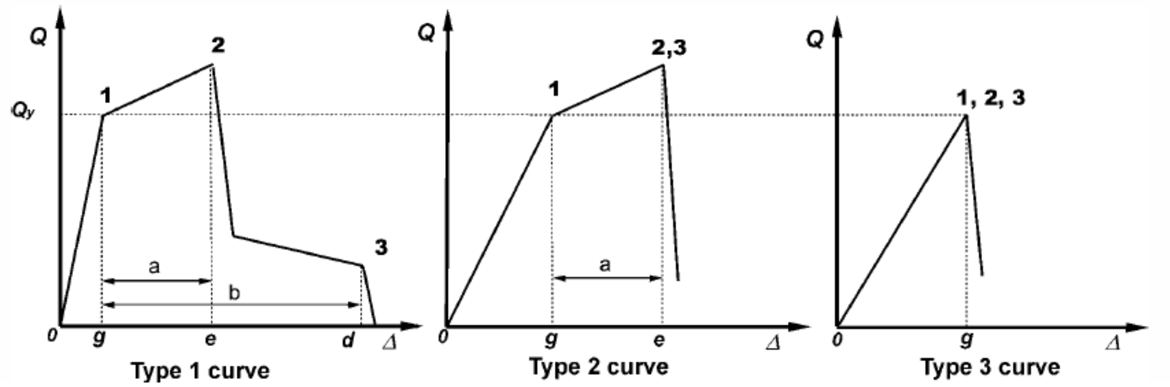


Figure 2.3. FEMA 356 types of component force- deformation curves (FEMA 356)

The Type 1 curve depicted in Figure 2.3 is representative of ductile behavior where there is an elastic range (point 0 to point 1 on the curve) followed by a plastic range (points 1 to 3) with non-negligible residual strength and ability to support gravity loads at point 3. The plastic range includes a strain hardening or softening range (points 1 to 2) and a strength-degraded range (points 2 to 3). Primary component actions exhibiting this behavior shall be classified as deformation-controlled if the strain-hardening or strain softening range is such that e > 2g; otherwise, they shall be classified as force-controlled. Secondary component actions exhibiting Type 1 behavior shall be classified as deformation-controlled for any e/g ratio.

The Type 2 curve depicted in Figure 2.3 is representative of ductile behavior where there is an elastic range (point 0 to point 1 on the curve) and a plastic range (points 1 to 2) followed by loss of strength and loss of ability to support gravity loads beyond point 2. Primary and secondary component actions exhibiting this type of behavior shall be classified as deformation-controlled if the plastic range is such that e > 2g; otherwise, they shall be classified as force controlled.

The Type 3 curve depicted in Figure 2.3 is representative of a brittle or nonductile behavior where there is an elastic range (point 0 to point 1 on the curve) followed by loss of strength and loss of ability to support gravity loads beyond point 1. Primary and secondary component actions displaying Type 3 behavior shall be classified as force-controlled.

Acceptance criteria for primary components that exhibit Type 1 behavior are typically within the elastic or plastic ranges between points 0 and 2, depending on the performance level. Acceptance criteria for secondary elements that exhibit Type 1 behavior can be within any of the performance ranges. Acceptance criteria for primary and secondary components exhibiting Type 2 behavior will be within the elastic or plastic ranges, depending on the performance level. Acceptance criteria for primary and secondary components exhibiting Type 3 behavior will always be within the elastic range.

A given component may have a combination of both force- and deformation-controlled actions. Classification as a deformation-controlled action is not up to the discretion of the user. Deformation-controlled actions have been defined in FEMA 356 by the designation of m-factors or nonlinear deformation capacities. In the absence of component testing justifying Type 1 or Type 2 behavior, all other actions are to be taken as force controlled.

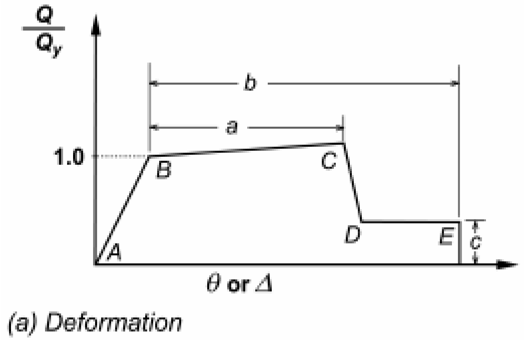
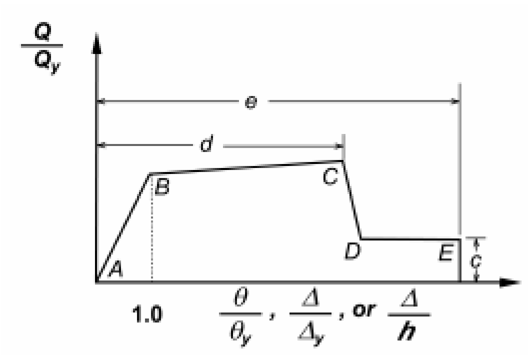
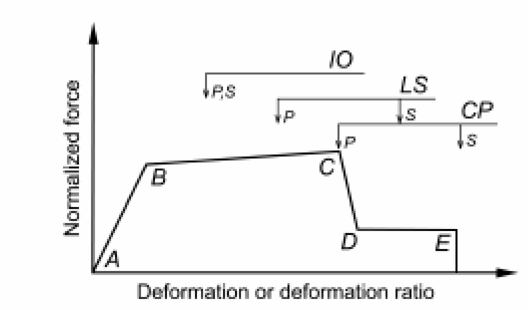


Figure 2.4. Force-deformation curve part a (FEMA 356)

Linear response is depicted between point A (unloaded component) and an effective yield point B. The slope from B to C is typically a small percentage (0-10%) of the elastic slope, and is included to represent phenomena such as strain hardening. C has an ordinate that represents the strength of the component, and an abscissa value equal to the deformation at which significant strength degradation begins (line CD). Beyond point D, the component responds with substantially reduced strength to point E. At deformations greater than point E, the component strength is essentially zero. The sharp transition as shown on idealized curves in Figure 2.4 between points C and D can result in computational difficulty and an inability to converge when used as modeling input in nonlinear computerized analysis software. In order to avoid this computational instability, a small slope may be provided to the segment of these curves between points C and D.



(b) Deformation ratio



(c) Component or element deformation acceptance criteria

Figure 2.5. Force- deformation curves b) Deformation ratio c) Component or element deformation acceptance criteria (FEMA 356)

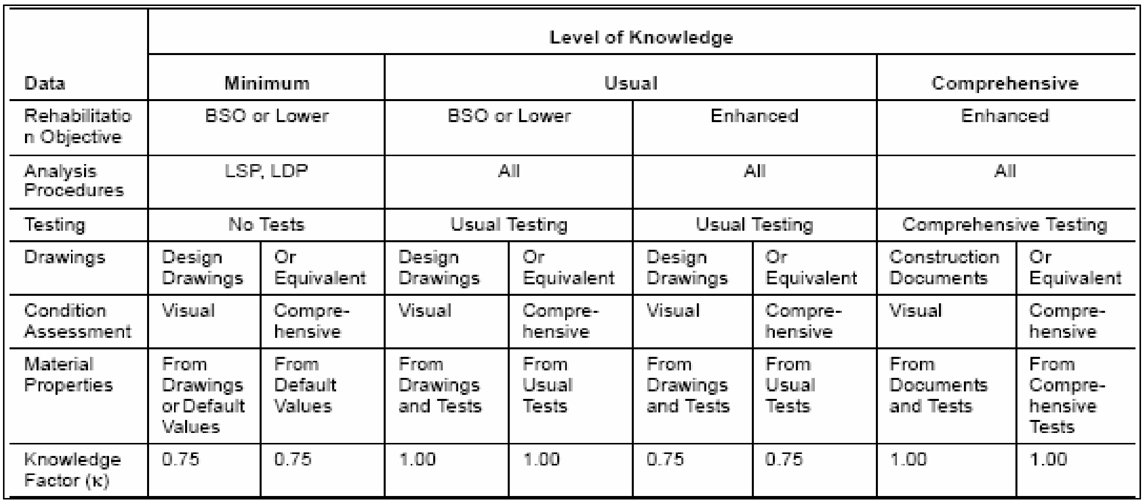
For some components it is convenient to prescribe acceptance criteria in terms of deformation (e.g., θ or Δ), while for others it is more convenient to give criteria in terms of deformation ratios. To accommodate this, two types of idealized force vs. deformation curves are used in Figure 2.4 and Figure 2.5.(b). Figure Figure 2.4 shows normalized force (Q/Q_{CE}) versus deformation (θ or Δ) and the parameters a, b, and c. Figure 2.5 (b) shows normalized force (Q/ Q_{CE}) versus deformation ratio (θ/θy, Δ/Δy, or Δ/h) and the parameters d, e, and c. Elastic stiffnesses and values for the parameters a, b, c, d, and e that can be used for modeling components. Acceptance criteria for deformation or deformation ratios for primary members (P) and secondary members (S) corresponding to the target Building

Performance Levels of Collapse Prevention (CP), Life Safety (LS), and Immediate Occupancy (IO) .

2.4.6. Knowledge Factor

To account for uncertainty in the collection of as-built data, a knowledge factor, κ, shall be selected from Table 2.6 considering the selected Rehabilitation Objective, analysis procedure, and data collection process. Knowledge factors shall be applied on a component basis as determined by the level of knowledge obtained for individual components during data collection. The κ factor is used to express the confidence with which the properties of the building components are known, when calculating component capacities. The value of the factor is established from the knowledge obtained based on access to original construction documents, or condition assessments including destructive or nondestructive testing of representative components. The values of the factor have been established, indicating whether the level of knowledge is “minimum,” “usual,” or “comprehensive.”

Table 2.6. Knowledge factors (FEMA 356)



Data	Level of Knowledge							
	Minimum		Usual				Comprehensive	
Rehabilitation Objective	BSO or Lower		BSO or Lower		Enhanced		Enhanced	
Analysis Procedures	LSP, LDP		All		All		All	
Testing	No Tests		Usual Testing		Usual Testing		Comprehensive Testing	
Drawings	Design Drawings	Or Equivalent	Design Drawings	Or Equivalent	Design Drawings	Or Equivalent	Construction Documents	Or Equivalent
Condition Assessment	Visual	Compre-hensive	Visual	Compre-hensive	Visual	Compre-hensive	Visual	Compre-hensive
Material Properties	From Drawings or Default Values	From Default Values	From Drawings and Tests	From Usual Tests	From Drawings and Tests	From Usual Tests	From Documents and Tests	From Compre-hensive Tests
Knowledge Factor (κ)	0.75	0.75	1.00	1.00	0.75	0.75	1.00	1.00

2.4.7. Selected Analysis Procedures

An analysis of the building shall be performed using the Linear Static Procedure (LSP), Linear Dynamic Procedure (LDP), Nonlinear Static Procedure (NSP), or Nonlinear Dynamic Procedure (NDP) selected based on the limitations specified in Section 2.4.

Four procedures are presented for seismic analysis of buildings: two linear procedures, and two nonlinear procedures. The two linear procedures are termed the Linear Static Procedure (LSP) and the Linear Dynamic Procedure (LDP). The two nonlinear procedures are termed the Nonlinear Static Procedure (NSP) and Nonlinear Dynamic Procedure (NDP).

Linear procedures are appropriate when the expected level of nonlinearity is low. This is measured by component demand to capacity ratios (DCRs) of less than 2.0. Static procedures are appropriate when higher mode effects are not significant. This is generally true for short, regular buildings. Dynamic procedures are required for tall buildings, buildings with torsional irregularities, or non-orthogonal systems. The Nonlinear Static Procedure is acceptable for most buildings, but should be used in conjunction with the Linear Dynamic Procedure if mass participation in the first mode is low.

The term “linear” in linear analysis procedures implies “linearly elastic.” The analysis procedure, however, may include geometric nonlinearity of gravity loads acting through lateral displacements and implicit material nonlinearity of concrete and masonry components using properties of cracked sections. The term “nonlinear” in nonlinear analysis procedures implies explicit material nonlinearity or inelastic material response, but geometric nonlinearity may also be included.

2.4.8. Linear Procedures

For linear procedures, the stability coefficient θ shall be evaluated for each story in the building and for each direction of response using (2.13).

$$\theta_i = \frac{P_i \delta_i}{V_i h_i} \tag{2.13}$$

P_i = Portion of the total weight of the structure including dead, permanent live, and 25% of transient live loads acting on the columns and bearing walls within story level i.

V_i = The total calculated lateral shear force in the direction under consideration at story i due to earthquake response to the selected ground shaking level, as indicated by the selected linear analysis procedure.

h_i = Height of story i, which shall be taken as the distance between either the centerline of floor framing at each of the levels above and below, or the top of floor slabs at each of the levels above and below (or other common points of reference)

δ_i = Lateral drift in story i, in the direction under consideration, at its center of rigidity, using the same units as for measuring h_i.

When the stability coefficient θ_i is less than 0.1 in all stories, the P-Δ effects need not be considered. If the stability coefficient lies between 0.1 and 0.33, seismic forces and deformations in story i shall be increased by the factor $1/(1-\theta_i)$. When the stability coefficient θ_i exceeds 0.33, the structure shall be considered unstable and the rehabilitation design modified to reduce the computed lateral deflections in the story to comply with this limitation.

2.4.9. Nonlinear Procedures

For nonlinear procedures, static P-Δ effects shall be incorporated in the analysis by including in the mathematical model the nonlinear force-deformation relationship of all elements and components subjected to axial forces.

2.4.10. P-Δ Effects

Static P-Δ effects are caused by gravity loads acting through the deformed configuration of a building and result in an increase in lateral displacements. A negative post-yield stiffness may significantly increase inter-story drift and the target displacement. Dynamic P-Δ effects are introduced to consider this additional drift. The degree by which dynamic P-Δ effects increase displacements depends on the following:

- The ratio α of the negative post-yield stiffness to the effective elastic stiffness;
- The fundamental period of the building;
- The strength ratio, R;
- The hysteretic load-deformation relations for each story;
- The frequency characteristics of the ground motion; and
- The duration of the strong ground motion.

Because of the number of parameters involved, it is difficult to capture dynamic P-Δ effects with a single modification factor. Coefficient C3 represents a substantial simplification and interpretation of much analysis data. Dynamic P-Δ effects are automatically captured in the NDP.

2.4.11. Component Gravity Loads for Load Combinations

The following component gravity forces, Q_G, shall be considered for combination with seismic loads. When the effects of gravity and seismic loads are additive, the gravity loads shall be obtained in accordance with 2.13.

$$Q_G = 1.1(Q_D + Q_L + Q_S) \tag{2.14}$$

When the effects of gravity and seismic loads are counteracting, the gravity loads shall be obtained in accordance with 2.14.

$$Q_G = 0.9Q_D \tag{2.15}$$

Where,

Q_D = Dead-load (action).

Q_L = Effective live load (action), equal to 25% of the unreduced design live load, but not less than the actual live load.

Q_S = Effective snow load (action)

2.5. Linear Dynamic Procedure

2.5.1. Basis of the Procedure

If the Linear Dynamic Procedure (LDP) is selected for seismic analysis of the building, the design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements shall be determined using a linearly elastic, dynamic analysis in compliance with the requirements of this section Buildings shall be modeled with linearly elastic stiffness and equivalent viscous damping values consistent with components responding at or near yield level. Results of the LDP shall be checked using the acceptance criteria of Section 3.4.2. Modal spectral analysis is carried out using linearly elastic response spectra that are not modified t account for anticipated nonlinear response (Chopra, 1995).

As with the LSP, it is expected that the LDP will produce displacements that approximate maximum displacements expected during the design earthquake, but will produce internal forces that exceed those that would be obtained in a yielding building.

Calculated internal forces typically will exceed those that the building can sustain because of anticipated inelastic response of components and elements. These design forces are evaluated through the acceptance criteria of Section 3.4.2, which include modification factors and alternative analysis procedures to account for anticipated inelastic response demands and capacities.

2.5.2. Modeling and Analysis Considerations

The ground motion characterized for dynamic analysis shall comply with the following requirements. The LDP includes two analysis methods, namely, the Response Spectrum Method and the Time History Method. The Response Spectrum Method uses peak modal responses calculated from dynamic analysis of a mathematical model. Only those modes contributing significantly to the response need to be considered. Modal responses are combined using rational methods to estimate total building response quantities.

Dynamic analysis using the response spectrum method shall calculate peak modal responses for sufficient modes to capture at least 90% of the participating mass of the building in each of two orthogonal principal horizontal directions of the building. Modal damping ratios shall reflect the damping in the building at deformation levels less than the yield deformation. Peak member forces, displacements, story forces, story shears, and base reactions for each mode of response shall be combined by either the SRSS (square root sum of squares) rule or the CQC (complete quadratic combination) rule. Multidirectional seismic effects shall be considered.

2.5.3. Determination of Forces and Deformations

All forces and deformations calculated using the Response Spectrum shall be multiplied by the product of the modification factors C1, C2, and C3 defined in the following Section and further modified to consider the effects of torsion in accordance with Section 3.2.2.2. Forces and deformations in elements and components shall be calculated for the pseudo lateral load.

2.5.4. Pseudo Lateral Load

The pseudo lateral load in a given horizontal direction of a building shall be determined using (2.16). This load shall be used to design the vertical elements of the lateral-force-resisting system.

$$V = C_1 C_2 C_3 C_m S_a W \tag{2.16}$$

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response, calculated as follows:

$C_1 = 1.5$ for $T<0.1$ second.

C_1 = 1.0 for $T \geq T_S$ second.

Linear interpolation shall be used to calculate for intermediate values of T.

T = Fundamental period of the building in the direction under consideration.

T_S = Characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum.

C_2 = Modification factor to represent the effects of pinched hysteresis shape, stiffness degradation, and strength deterioration on maximum displacement response. For linear procedures C_2 shall be taken as 1.0.

C_3= Modification factor to represent increased displacements due to dynamic P-Δ effects. For values of the stability coefficient θ_i per (2.17) less than 0.1 in all stories, C_3 shall be set equal to 1.0, otherwise C_3 shall be calculated as:

$$1 + 5(\theta - 0.1)/T \qquad (2.17)$$

using θ equal to the maximum value of θ_i of all stories.

C_m= Effective mass factor to account for higher mode mass participation effects obtained from Table 2.7. C_m shall be taken as 1.0 if the fundamental period, T, is greater than 1.0 second.

Table 2.7. Cm factor

No. Of Stories	Concrete Moment Frame	Concrete Shear Wall	Concrete Pier-Spandrel	Steel CBF	Steel EBF	Other
1 _ 2	1,0	1,0	1,0	1,0	1,0	1,0
3 or more	0,9	0,8	0,9	0,9	0,9	1,0

S_a = Response spectrum acceleration, at the fundamental period and damping ratio of the building in the direction under consideration.

W = Effective seismic weight of the building including the total dead load and applicable portions of other gravity loads listed below:

- In areas used for storage, a minimum 25% of the floor live load shall be applicable. The live load shall be permitted to be reduced for tributary area as approved by the code official. Floor live load in public garages and open parking structures is not applicable.
- Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 10 psf of floor area, whichever is greater, shall be applicable.
- Total operating weight of permanent equipment.
- Where the design flat roof snow load calculated in accordance with ASCE 7 exceeds 30 psf, the effective snow load shall be taken as 20% of the design snow load. Where the design flat roof snow load is less than 30 psf, the effective snow load shall be permitted to be zero.

2.5.5. Vertical Distribution of Seismic Forces

The lateral load applied at any floor level x shall be determined in accordance with following equations:

$$F_x = C_{vx} V \tag{2.18}$$

$$C_{vx} = \frac{w_x {h_x}^k}{\sum_{i=1}^{n} w_i {h_i}^k} \tag{2.19}$$

Where:

C_{vx} = Vertical distribution factor

k = 2.0 for T≥2.5 seconds

= 1.0 for T≤0.5 seconds

Linear interpolation shall be used to calculate values of k for intermediate values of T.

V = Pseudo lateral load

w_i = Portion of the total building weight W located on or assigned to floor level i

w_x = Portion of the total building weight W located on or assigned to floor level x

h_i = Height from the base to floor level i

h_x = Height from the base to floor level x

2.5.6. Horizontal Distribution of Seismic Forces

The seismic forces at each floor level of the building shall be distributed according to the distribution of mass at that floor level.

$$F_{px} = \sum_{i=x}^{n} F_i \frac{w_x}{\sum_{i=x}^{n} w_i} \tag{2.20}$$

Where:

F_{px} = Total diaphragm inertial force at level x

F_i = Lateral load applied at floor level i

The seismic load on each flexible diaphragm shall be distributed along the span of that diaphragm, proportional to its displaced shape.

2.5.7. Diaphragms

Diaphragms shall be designed to resist simultaneously

- The seismic forces calculated by the LDP,
- The horizontal forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements above and below the diaphragm.

The seismic forces calculated by the LDP shall be taken as not less than 85% of the forces. Forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements shall be taken to be equal to the elastic forces without reduction, unless smaller forces are justified by an approved rational analysis. Diaphragm actions need not be multiplied by the product of the modification factors C_1, C_2, and C_3.

2.6. Nonlinear Static Procedure

2.6.1. Basis of the Procedure

If the Nonlinear Static Procedure (NSP) is selected for seismic analysis of the building, a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building shall be subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a target displacement is exceeded. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. Because the mathematical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake.

2.6.2. Modeling and Analysis Considerations

The selection of a control node, the selection of lateral load patterns, the determination of the fundamental period, and analysis procedures shall comply with the requirements of this section. The relation between base shear force and lateral displacement of the control node shall be established for control node displacements ranging between zero and 150% of the target displacement, δ_t.

The component gravity loads shall be included in the mathematical model for combination with lateral loads. The lateral loads shall be applied in both the positive and negative directions, and the maximum seismic effects shall be used for design. The analysis model shall be discriticized to represent the load-deformation response of each component along its length to identify locations of inelastic action. All primary and secondary lateral-force-resisting elements shall be included in the model. The force-displacement behavior of all components shall be explicitly included in the model using full backbone curves that include strength degradation and residual strength, if any. Alternatively, the use of a simplified NSP analysis shall be permitted. In a simplified NSP analysis only primary lateral force resisting elements are modeled, the force displacement characteristics of such elements are bilinear, and the degrading portion of the backbone curve is not explicitly modeled. Elements not meeting the acceptance criteria for primary components shall be designated as secondary, and removed from the mathematical model. When using the simplified NSP analysis, care should be taken to make sure that removal of degraded elements from the model does not result changes in the regularity of the structure that would significantly alter the dynamic response.

In pushing with a static load pattern, the NSP does not capture changes in the dynamic characteristics of the structure as yielding and degradation take place. In order to explicitly evaluate deformation demands on secondary elements that are to be excluded from the model, one might consider including them in the model, but with negligible stiffness, to obtain deformations demands without significantly affecting the overall response.

2.6.3. Control Node Displacement

The control node shall be located at the center of mass at the roof of a building. For buildings with a penthouse, the floor of the penthouse shall be regarded as the level of the control node. The displacement of the control node in the mathematical model shall be calculated for the specified lateral loads.

2.6.4. Lateral Load Distribution

Lateral loads shall be applied to the mathematical model in proportion to the distribution of inertia forces in the plane of each floor diaphragm. For all analyses, at least two vertical distributions of lateral load shall be applied. One pattern shall be selected from each of the following two groups:

A modal pattern selected from one of the following:

- A vertical distribution proportional to the values of Cvx. Use of this distribution shall be permitted only when more than 75% of the total mass participates in the fundamental mode in the direction under consideration, and the uniform distribution is also used.
- A vertical distribution proportional to the shape of the fundamental mode in the direction under consideration. Use of this distribution shall be permitted only when more than 75% of the total mass participates in this mode.
- A vertical distribution proportional to the story shear distribution calculated by combining modal responses from a response spectrum analysis of the building, including sufficient modes to capture at least 90% of the total building mass, and using the appropriate ground motion spectrum. This distribution shall be used when the period of the fundamental mode exceeds 1.0 second.

A second pattern selected from one of the following:

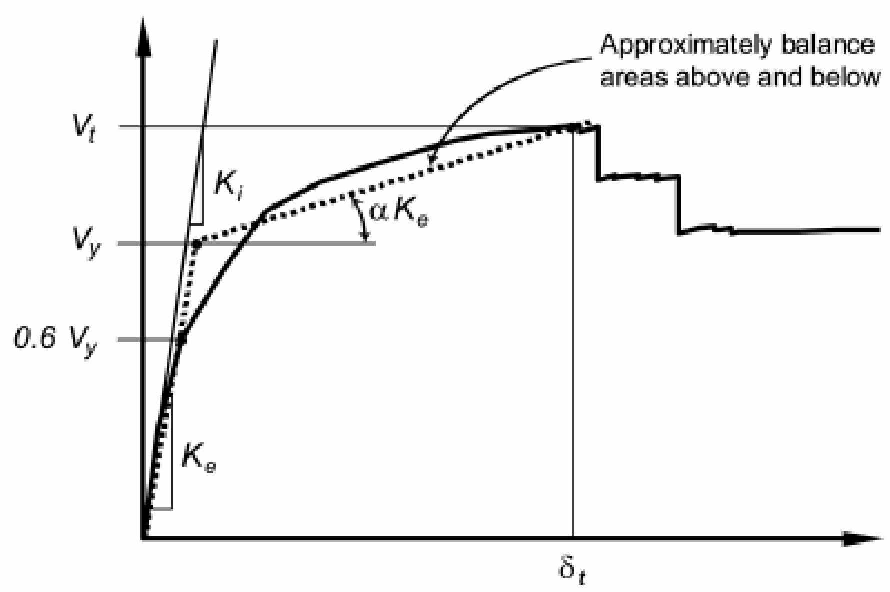
- A uniform distribution consisting of lateral forces at each level proportional to the total mass at each level.
- An adaptive load distribution that changes as the structure is displaced. The adaptive load distribution shall be modified from the original load distribution using a procedure that considers the properties of the yielded structure.

The distribution of lateral inertial forces determines relative magnitudes of shears, moments, and deformations within the structure. The distribution of these forces will vary continuously during earthquake response as portions of the structure yield and stiffness characteristics change. The extremes of this distribution will depend on the severity of the earthquake shaking and the degree of nonlinear response of the structure.

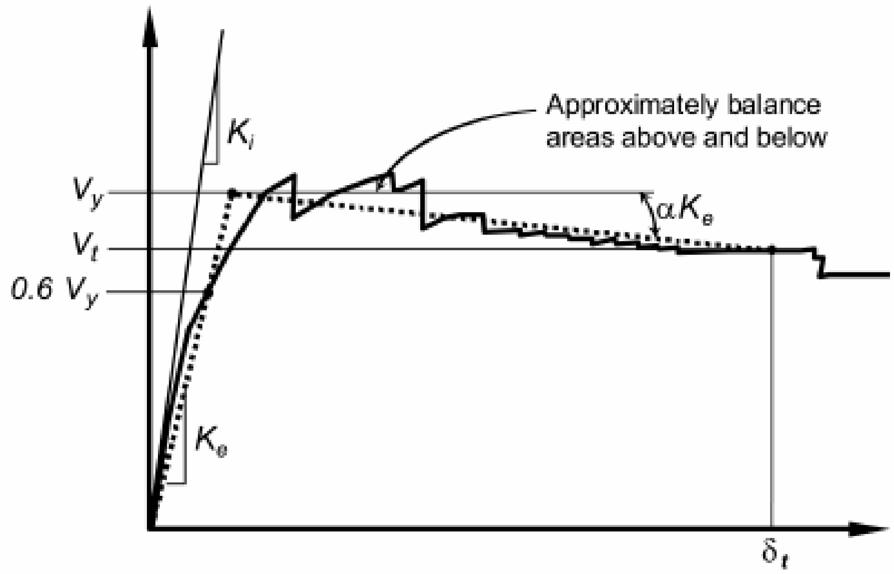
Use of more than one lateral load pattern is intended to bound the range of design actions that may occur during actual dynamic response. In lieu of using the uniform distribution to bound the solution, changes in the distribution of lateral inertial forces can be investigated using adaptive load patterns that change as the structure is displaced to larger amplitudes. Procedures for developing adaptive load patterns include the use of story forces proportional to the deflected shape of the structure, the use of load patterns based on mode shapes derived from secant stiffnesses at each load step and the use of load patterns proportional to the story shear resistance at each step. Use of an adaptive load pattern will require more analysis effort, but may yield results that are more consistent with the characteristics of the building under consideration.

2.6.5. Idealized Force-Displacement Curve

The nonlinear force-displacement relationship between base shear and displacement of the control node shall be replaced with an idealized relationship to calculate the effective lateral stiffness, K_e, and effective yield strength, V_y, of the building as shown in Figure 3-1. This relationship shall be bilinear, with initial slope K_e and post-yield slope α. Line segments on the idealized force-displacement curve shall be located using an iterative graphical procedure that approximately balances the area above and below the curve. The effective lateral stiffness, K_e, shall be taken as the secant stiffness calculated at a base shear force equal to 60% of the effective yield strength of the structure. The post-yield slope, α, shall be determined by a line segment that passes through the actual curve at the calculated target displacement. The effective yield strength shall not be taken as greater than the maximum base shear force at any point along the actual curve.



(a) Positive post-yield slope



(b) Negative post-yield slope

Figure 2.6. Idealized force-displacement curves a) Positive post-yield slope b) Negative post-yield slope (FEMA 356)

2.6.6. Period Determination

The effective fundamental period in the direction under consideration shall be based on the idealized force displacement curve. The effective fundamental period, T_e, shall be calculated in accordance with:

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \tag{2.21}$$

Where:

T_i = Elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis

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

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

2.6.7. Analysis of Mathematical Models

Separate mathematical models representing the framing along two orthogonal axes of the building shall be developed for two-dimensional analysis. A mathematical model representing the framing along two orthogonal axes of the building shall be developed for three-dimensional analysis. The effects of horizontal torsion shall be evaluated. Independent analysis along each of the two orthogonal principal axes of the building shall be permitted unless concurrent evaluation of multidirectional effects is required.

2.6.8. Determination of Forces and Deformations

For buildings with rigid diaphragms at each floor level, the target displacement, δ_t, shall be calculated by an approved procedure that accounts for the nonlinear response of the building. For buildings with non-rigid diaphragms at each floor level, diaphragm flexibility shall be explicitly included in the model. The target displacement shall be calculated as specified for rigid diaphragms, except that it shall be amplified by the ratio of the maximum displacement at any point on the roof to the displacement at the center of mass of the roof (δ_{max}/ δ_{cm}). δ_{max} and δ_{cm} shall be based on a response spectrum analysis of a three-dimensional model of the building. The target displacement so calculated shall be no less than that displacement given by the equation stated in target displacement section (McGuire, 2000).

No line of vertical seismic framing shall be evaluated for displacements smaller than the target displacement. Alternatively, for buildings with flexible diaphragms at each floor level, a target displacement shall be calculated for each line of vertical seismic framing. The target displacement for an individual line of vertical seismic framing shall be as specified for buildings with rigid diaphragms, except that the masses shall be assigned to each line on the basis of tributary area. Forces and deformations corresponding to the control node displacement equaling or exceeding the target displacement shall comply with acceptance criteria.

2.6.9. Target Displacement

The target displacement, δt, at each floor level shall be calculated in accordance with (2.22):

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_a^{\ 2}}{4\pi^2} g \tag{2.22}$$

C_0 is the modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system calculated using one of the following procedures:

- The first modal participation factor at the level of the control node
- The modal participation factor at the level of the control node calculated using a shape vector corresponding to the deflected shape of the building at the target displacement.
- The appropriate value from Table 2.8.

Table 2.8. C_0 factors

Values for Modification Factor C_0			
Number of Stories	Shear Buildings		Other
	Triangular Load Pattern	Uniform Load Pattern	Any Pattern
1	1.0	1.0	1.0
2	1.2	1.2	1.2
3	1.2	1.2	1.3
5	1.3	1.2	1.4
10+	1.3	1.2	1.5

C_1 is the modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response:

$$C_1 = \begin{cases} 1.0 \quad for \;\; T_e \geq T_S \\ C_1 = \left[1.0 + (R-1) {T_S}/{T_e} \right] / R \quad for \;\; T_e < T_S \end{cases} \tag{2.23}$$

Te is the effective fundamental period of the building in the direction under consideration.

T_S is the characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum. R is the ratio of elastic strength demand to calculated yield strength coefficient calculated by 2.23.

$$R = \frac{S_a}{V_y / W} . C_m \tag{2.24}$$

C_2 is the modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response. Values of C_2 for different framing systems and Structural Performance Levels shall be obtained from Table 2.9. Alternatively, use of C2 = 1.0 shall be permitted for nonlinear procedures.

Table 2.9. C_2 factors

Values for Modification Factor C_2				
Structural Performance Level	T≤0.1 Seconds		T≥T_S Seconds	
	Framing Type 1	Framing Type 2	Framing Type 1	Framing Type 2
IO	1.0	1.0	1.0	1.0
LS	1.3	1.0	1.1	1.0
CP	1.5	1.0	1.2	1.0

C_3 is the modification factor to represent increased displacements due to dynamic P-Δ effects. For buildings with positive post-yield stiffness, shall be set equal to 1.0. For buildings with negative post-yield stiffness.

$$C_3 = 1 + \frac{|\alpha|(R-1)^{3/2}}{T_e} \tag{2.25}$$

Where, α is the ratio of post-yield stiffness to effective elastic stiffness.

S_a is the response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration, g.

2.6.10. Consideration of Torsional Effects

The target displacement shall be modified to consider the effects of horizontal torsion. Effects of horizontal torsion shall be considered in accordance with the following requirements:

- Increased forces and displacements due to actual torsion shall be calculated for all buildings.
- The displacement multiplier, η, at each floor shall be calculated as the ratio of the maximum displacement at any point on the floor diaphragm to the average displacement ($\delta_{max}/\delta_{avg}$). Displacements shall be calculated for the applied loads.
- Increased forces and displacements due to accidental torsion shall be considered unless the accidental torsional moment is less than 25 percent of the actual torsional moment, or the displacement multiplier η due to the applied load and accidental torsion is less than 1.1 at every floor.
- For linear analysis procedures, forces and displacements due to accidental torsion shall be amplified by a factor, Ax, when the displacement multiplier η due to total torsional moment exceeds 1.2 at any level (McGuire, 2000).

$$A_x = \left(\frac{\eta_x}{1.2}\right)^2 \leq 3.0 \tag{2.26}$$

- If the displacement modifier η due to total torsional moment at any floor exceeds 1.50, two-dimensional models shall not be permitted and three-dimensional models that account for the spatial distribution of mass and stiffness shall be used.
- When two-dimensional models are used, the effects of horizontal torsion shall be calculated as follows:
    - For the LSP and the LDP, forces and displacements shall be amplified by the maximum value of η calculated for the building.
    - For the NSP, the target displacement shall be amplified by the maximum value of η calculated for the building.
    - For the NDP, the amplitude of the ground acceleration record shall be amplified by the maximum value of η calculated for the building.
- The effects of accidental torsion shall not be used to reduce force and deformation demands on components and elements.

Actual torsion is due to the eccentricity between centers of mass and stiffness. Accidental torsion is intended to cover the effects of the rotational component of the ground motion, differences between computed and actual stiffness, and unfavorable distributions of dead and live load masses. The 10% threshold on additional displacement due to accidental torsion is based on judgment. The intent is to reward those building frames that are torsionally redundant and possess high torsional stiffness. Such structures are likely to be much less susceptible to torsional response than those framing systems possessing low redundancy and low torsional stiffness (Aydinoglu, 2005).

3. FORTH REQUIREMENTS FOR THE SYSTEMATIC REHABILITATION OF CONCRETE LATERAL FORCE RESISTING ELEMENTS

3.1. Material Properties Based on Historical Information

The form, function, concrete strength, concrete quality, reinforcing steel strength, quality and detailing, forming techniques and concrete placement techniques have constantly evolved and have had a significant impact on the seismic resistance of a concrete building. Innovations such as pre-stressed and precast concrete, post tensioning, and lift slab construction have created a multivariate inventory of existing concrete structures.

It is important to investigate the local practices relative to seismic design when trying to analyze a concrete building. Specific benchmark years can be determined for the implementation of earthquake-resistant design in most locations, but caution should be exercised in assuming optimistic characteristics for any specific building.

Particularly with concrete materials, the date of original building construction significantly influences seismic performance. In the absence of deleterious conditions or materials, concrete gains compressive strength from the time it is originally cast and in-place. Strengths typically exceed specified design values (28- day or similar). Early uses of concrete did not specify any design strength, and low-strength concrete was not uncommon. Also, early use of concrete in buildings often employed reinforcing steel with relatively low strength and ductility, limited continuity, and reduced bond development. Continuity between specific existing components and elements (e.g., beams and columns, diaphragms and shear walls) is also particularly difficult to assess, given the presence of concrete cover and other barriers to inspection.

The following component and connection material properties shall be obtained for the as-built structure:

- Concrete compressive strength.
- Yield and ultimate strength of conventional and pre-stressing reinforcing steel and metal connection hardware.

Other material properties that may be of interest for concrete elements and components include:

- Tensile strength and modulus of elasticity, which can be derived from the compressive strength, do not warrant the damage associated with the extra coring required.
- Ductility, toughness, and fatigue properties of concrete.
- Carbon equivalent present in the reinforcing steel.
- Presence of any degradation such as corrosion, bond with concrete and chemical composition.

The effort required to determine these properties depends on the availability of accurate updated construction documents and drawings, quality and type of construction (absence of degradation), accessibility, and condition of materials. The method of analysis selected (e.g., Linear Static Procedure, Nonlinear Static Procedure) may also influence the scope of the testing.

Generally, mechanical properties for both concrete and reinforcing steel can be established from combined core and specimen sampling at similar locations, followed by laboratory testing. Core drilling should minimize damaging the existing reinforcing steel as much as practicable.

3.1.1. Nominal or Specified Properties

Nominal material properties, or properties specified in construction documents, shall be taken as lower bound material properties. Corresponding expected material properties shall be calculated by multiplying lower bound values by a factor taken from

Table 3.1 to translate from lower bound to expected values. Alternative factors shall be permitted where justified by test data.

Table 3.1. Factors to translate lower bound material to expected strength

Material Property	Factor
Concrete Compressive Strength	1,50
Reinforcing Steel Tensile & Strength	1,25
Connector Steel Yield Strength	1,50

3.1.2. Component Properties

The following component properties and as-built conditions shall be established:

- Cross-sectional dimensions of individual components and overall configuration of the structure.
- Configuration of component connections, size of anchor bolts, thickness of connector material, anchorage and interconnection of embedment and the presence of bracing or stiffening components.
- Modifications to components or overall configuration of the structure.
- Current physical condition of components and connections, and the extent of any deterioration present.
- Presence of conditions that influence building performance.

Component properties may be needed to characterize building performance properly in the seismic analysis. The starting point for assessing component properties and condition should be retrieval of available construction documents. Preliminary review of these documents shall be performed to identify primary vertical- (gravity-) and lateral load-carrying elements, systems, and their critical components and connections. In the absence of a complete set of building drawings, the design professional must perform a thorough inspection of the building to identify these elements, systems and components.

3.1.3. Condition Assessment

A condition assessment of the existing building and site conditions shall be performed as specified in this section. The condition assessment shall include the following:

- The physical condition of primary and secondary components shall be examined and the presence of any degradation shall be noted.
- The presence and configuration of components and their connections, and the continuity of load paths between components, elements, and systems shall be verified or established.
- Other conditions including neighboring party walls and buildings, presence of nonstructural components, prior remodeling, and limitations for rehabilitation that may influence building performance shall be reviewed and documented.
- A basis for selecting a knowledge factor shall be formulated.
- Component orientation and physical dimensions shall be confirmed (Pristley, 2003).

The scope of the condition assessment shall include all accessible structural elements and components involved in lateral load resistance. The degree to which the condition assessment is performed will affect the knowledge (κ) factor.

3.1.4. Visual Condition Assessment

Direct visual inspection of accessible and representative primary components and connections shall be performed to identify any configurationally issues, determine whether degradation is present, establish continuity of load paths, establish the need for other test methods to quantify the presence and degree of degradation, and measure dimensions of existing construction to compare with available design information and reveal any permanent deformations.

Visual inspection of the building shall include visible portions of foundations, lateral-force-resisting members, diaphragms (slabs), and connections. As a minimum, a representative sampling of at least 20% of the elements, components, and connections shall be visually inspected at each floor level. If significant damage or degradation is found, the assessment sample of all critical components of similar type in the building shall be increased to 40%. If coverings or other obstructions exist, indirect visual inspection through the obstruction, using drilled holes and a fiberscope, shall be permitted.

3.1.5. Comprehensive Condition Assessment

Exposure is defined as local minimized removal of cover concrete and other materials to allow inspection of reinforcing system details. All damaged concrete cover shall be replaced after inspection. The following criteria shall be used for assessing primary connections in the building for comprehensive data collection:

- If detailed design drawings exist, exposure of at least three different primary connections shall occur, with the connection sample including different types of connections. If no deviations from the drawings exist, it shall be permitted to consider the sample as being representative of installed conditions. If deviations are noted, then at least 25% of the specific connection type shall be inspected to identify the extent of deviation.

- In the absence of detailed design drawings, at least three connections of each primary connection type shall be exposed for inspection. If common detailing among the three connections is observed, it shall be permitted to consider this condition as representative of installed conditions. If variations are observed among like connections, additional connections shall be inspected until an accurate understanding of building construction is gained.

3.1.6. Basis for the Mathematical Building Model

The results of the condition assessment shall be used to quantify the following items needed to create the mathematical building model:

- Component section properties and dimensions.
- Component configuration and the presence of any eccentricities or permanent deformation.
- Connection configuration and the presence of any eccentricities.
- Presence and effect of alterations to the structural system since original construction.
- Interaction of nonstructural components and their involvement in lateral load resistance (Pristley, 2003).

All deviations between available construction records and as-built conditions obtained from visual inspection shall be accounted for in the structural analysis. Unless concrete cracking, reinforcing corrosion, or other mechanisms are observed in the condition assessment to be causing damage or reduced capacity, the cross-sectional area and other sectional properties shall be taken as those from the design drawings. If some sectional material loss has occurred, the loss shall be quantified by direct measurement and sectional properties shall be reduced accordingly, using principles of structural mechanics.

3.1.7. Knowledge Factor

A knowledge factor (κ) for computation of concrete component capacities and permissible deformations shall be selected with the following additional requirements specific to concrete components.

A knowledge factor, κ, equal to 0.75 shall be used if any of the following criteria are met:

- Components are found damaged or deteriorated during assessment, and further testing is not performed to quantify their condition or justify the use of $\kappa=1.0$.
- Component mechanical properties have a coefficient of variation exceeding 25%.
- Components contain archaic or proprietary material and the condition is uncertain.

3.1.8. General Assumptions and Requirements

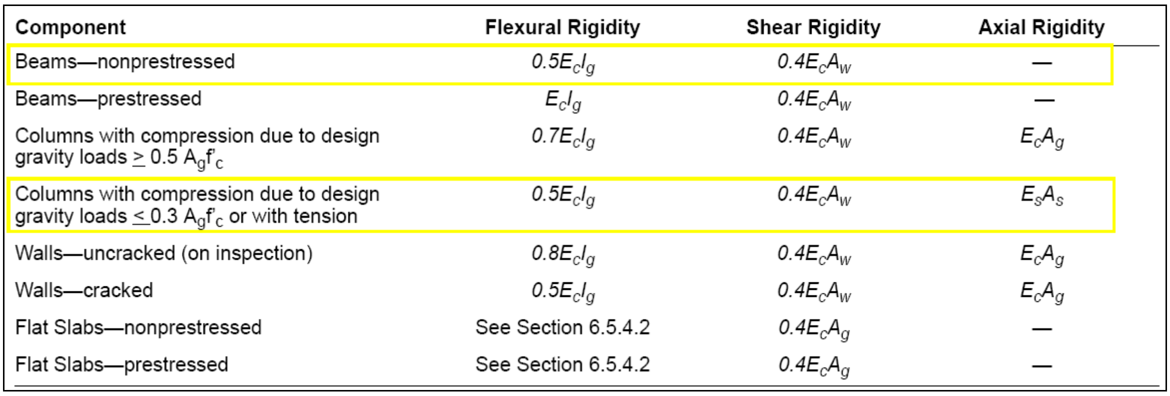
Seismic rehabilitation of concrete structural elements of existing buildings shall comply with the requirements of ACI 318, except as otherwise indicated in FEMA 356. Seismic evaluation shall identify brittle or low-ductility failure modes of force-controlled actions. Evaluation of demands and capacities of reinforced concrete components shall include consideration of locations along the length where lateral and gravity loads produce maximum effects, where changes in cross-section or reinforcement result in reduced strength, and where abrupt changes in cross section or reinforcement, including splices, may produce stress concentrations, resulting in premature failure.

Brittle or low-ductility failure modes typically include behavior in direct or nearly-direct compression; shear in slender components and in component connections, torsion in slender components, and reinforcement development, splicing, and anchorage. It is recommended that the stresses, forces, and moments acting to cause these failure modes be determined from a limit-state analysis considering probable resistances at locations of nonlinear action (Pristley, 2003).

3.1.9. Stiffness

Component stiffnesses shall be calculated considering shear, flexure, axial behavior and reinforcement slip deformations. Consideration shall be given to the state of stress on the component due to volumetric changes from temperature and shrinkage, and to deformation levels to which the component will be subjected under gravity and earthquake loading. The use of higher stiffnesses shall be permitted where it is demonstrate by analysis to be appropriate for the design loading. Alternatively, the use of effective stiffness values in Table 3.2. shall be permitted.

Table 3.2. Cracked section stiffness multipliers (FEMA 356)



Component	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Beams—nonprestressed	$0.5E_cI_g$	$0.4E_cA_w$	—
Beams—prestressed	E_cI_g	$0.4E_cA_w$	—
Columns with compression due to design gravity loads $\geq 0.5\ A_gf'_c$	$0.7E_cI_g$	$0.4E_cA_w$	E_cA_g
Columns with compression due to design gravity loads $\leq 0.3\ A_gf'_c$ or with tension	$0.5E_cI_g$	$0.4E_cA_w$	E_sA_s
Walls—uncracked (on inspection)	$0.8E_cI_g$	$0.4E_cA_w$	E_cA_g
Walls—cracked	$0.5E_cI_g$	$0.4E_cA_w$	E_cA_g
Flat Slabs—nonprestressed	See Section 6.5.4.2	$0.4E_cA_g$	—
Flat Slabs—prestressed	See Section 6.5.4.2	$0.4E_cA_g$	—

3.1.10. Strength and Deformability

Actions in a structure shall be classified as being either deformation-controlled or force-controlled. Components shall be classified as having low, moderate, or high ductility demands. Where strength and deformation capacities are derived from test data, the tests shall be representative of proportions, details, and stress levels for the component. The strength and deformation capacities of concrete members shall correspond to values resulting from earthquake loadings involving three fully reversed cycles to the design deformation level unless a larger or smaller number of deformation cycles is determined considering earthquake duration and the dynamic properties of the structure.

3.1.11. Deformation-Controlled Actions

Strengths used for deformation-controlled actions shall be taken as equal to expected strengths, Q_{CE}, obtained experimentally, or calculated using accepted principles of mechanics. Expected strength is defined as the mean maximum resistance expected over the range of deformations to which the concrete component is likely to be subjected. When calculations are used to define expected strength, expected material properties shall be used. Unless other procedures are specified in FEMA 356, procedures specified in ACI

318 to calculate design strengths shall be permitted except that the strength reduction factor, φ, shall be taken equal to unity.

3.1.12. Force-Controlled Actions

Strengths used for force-controlled actions shall be taken as lower-bound strengths, Q_{CL}, obtained experimentally, or calculated using established principles of mechanics. Lower-bound strength is defined as the mean minus one standard deviation of resistance expected over the range of deformations and loading cycles to which the concrete component is likely to be subjected. When calculations are used to define lower-bound strengths, lower bound estimates of material properties shall be used. Unless other procedures are specified FEMA 356, procedures specified in ACI 318 to calculate design strengths shall be permitted, except that the strength reduction factor, φ, shall be taken equal to unity.

3.1.13. Component Ductility Demand Classification

Components shall be classified as having low, moderate, or high ductility demands, based on the maximum value of the demand capacity ratio (DCR) for linear procedures, or the calculated displacement ductility for nonlinear.

Table 3.3. Ductility classification of components

Max Value of DCR or Displacement Ductility	Descriptor
<2	Low Ductility Demand
2 to 4	Moderate Ductility Demans
>4	High Ductility Demand

3.1.14. Flexure and Axial Loads

Flexural strength and deformation capacity of members with and without axial loads shall be calculated according to the procedures of ACI 318 or by other approved methods. Strengths and deformation capacities of components with monolithic flanges shall be calculated considering concrete and developed longitudinal reinforcement within the effective flange width. Strength and deformation capacities shall be determined considering available development of longitudinal reinforcement. Where longitudinal reinforcement has embedment or development length that is insufficient for development of reinforcement strength, flexural strength shall be calculated based on limiting stress capacity of the embedded bar. Where flexural deformation capacities are calculated from basic principles of mechanics, reductions in deformation capacity due to applied shear shall be taken into consideration. When using analytical models for flexural deformability that do not directly consider effect of shear, and where design shear equals or exceeds $6\sqrt{f_c{}'}\,A_W$, where f_c is in psi and A_w is gross area of web in square inches, the design value shall not exceed eighty percent of the value calculated using the analytical model (Ambrose, 1998).

For concrete columns under combined axial load and biaxial bending, the combined strength shall be evaluated considering biaxial bending. When using linear procedures, the design axial load, P_{UF}, shall be calculated as a force-controlled action. The design moments, M_{UD}, shall be calculated about each principal axis. Acceptance shall be based on the following equation:

$$\left[\frac{M_{UDx}}{m_x \kappa M_{CEx}}\right]^2 + \left[\frac{M_{UDy}}{m_y \kappa M_{CEy}}\right]^2 \le 1 \tag{3.1}$$

M_{UDx} = design bending moment about x axis for axial load P_{UF}, kip-in.
M_{UDy} = design bending moment about y axis fo axial load P_{UF}, kip-in.
M_{CEx} = expected bending moment strength about x axis, kip-in.
M_{CEy} = expected bending moment strength about y axis, kip-in.
m_x = m-factor for column for bending about x axis

m_y = m-factor for column for bending about y axis

3.1.15. Usable Strain Limits

Without confining transverse reinforcement, the maximum usable strain at the extreme concrete compression fiber shall not exceed 0.002 for components in nearly pure compression and 0.005 for other components unless larger strains are substantiated by experimental evidence and approved. Maximum usable compressive strains for confined concrete shall be based on experimental evidence and shall consider limitations posed by fracture of transverse reinforcement, buckling of longitudinal reinforcement, and degradation of component resistance at large deformation levels. Maximum compressive strains in longitudinal reinforcement shall not exceed 0.02, and maximum tensile strains in longitudinal reinforcement shall not exceed 0.05.

3.1.16. Shear and Torsion

Strengths in shear and torsion shall be calculated according to ACI 318 except as modified in FEMA 356. Within yielding regions of components with moderate or high ductility demands, shear and torsional strength shall be calculated according to procedures for ductile components, such as the provisions in Chapter 21 of ACI 318. Within yielding regions of components with low ductility demands and outside yielding regions for all ductility demands, calculation of design shear strength using procedures for effective elastic response such as the provisions in Chapter 11 of ACI 318 shall be permitted. Where the longitudinal spacing of transverse reinforcement exceeds half the component effective depth measured in the direction of shear, transverse reinforcement shall be assumed not more than 50% effective in resisting shear or torsion. Where the longitudinal spacing of transverse reinforcement exceeds the component effective depth measured in the direction of shear, transverse reinforcement shall be assumed ineffective in resisting shear or torsion (Mander, 1998).

For beams and columns in which perimeter hoops are either lap-spliced or have hooks that are not adequately anchored in the concrete core, transverse reinforcement shall be assumed not more than 50% effective in regions of moderate ductility demand and shall be assumed ineffective in regions of high ductility demand. Shear friction strength shall be calculated according to ACI 318, taking into consideration the expected axial load due to gravity and earthquake effects. Where rehabilitation involves the addition of concrete requiring overhead work with dry-pack, the shear friction coefficient μ shall be taken as equal to 70% of the value specified by ACI 318.

3.1.17. Reinforced Concrete Beam-Column Moment Frames

Reinforced concrete beam-column moment frames shall satisfy the following conditions:

- Framing components shall be beams (with or without slabs), columns, and their connections.
- Beams and columns shall be of monolithic construction that provides for moment transfer between beams and columns.
- Primary reinforcement in components contributing to lateral load resistance shall be non-prestressed. Special Moment Frames, Intermediate Moment Frames, and Ordinary Moment Frames as defined in ASCE 7, shall be deemed to satisfy the above conditions. This classification shall include existing construction, new construction, and existing construction that has been rehabilitated (Ambrose, 1998).

The analytical model for a beam-column frame element shall represent strength, stiffness, and deformation capacity of beams, columns, beam-column joints, and other components of the frame, including connections with other elements. Potential failure in flexure, shear, and reinforcement development at any section along the component length shall be considered. Interaction with other elements, including nonstructural elements and components, shall be included. Analytical models representing a beam- column frame using line elements with properties concentrated at component centerlines shall be permitted. Where beam and column centerlines do not intersect, the effects of the eccentricity between centerlines on framing shall be taken into account. Where the centerline of the narrower component falls within the middle third of the adjacent framing component measured transverse to the framing direction, however, this eccentricity need not be considered. Where larger eccentricities occur, the effect shall be represented either by reductions in effective stiffness, strength, and deformation capacity, or by direct modeling of the eccentricity.

The beam-column joint in monolithic construction shall be represented as a stiff or rigid zone having horizontal dimensions equal to the column cross-sectional dimensions and vertical dimension equal to the beam depth, except that a wider joint shall be permitted where the beam is wider than the column and where justified by experimental evidence. The model of the connection between the columns and foundation shall be selected based on the details of the column-foundation connection and rigidity of the foundation-soil system. Action of the slab as a diaphragm interconnecting vertical elements shall be represented. Action of the slab as a composite beam flange shall be considered in developing stiffness, strength, and deformation capacities of the beam component model. Inelastic action shall be restricted to those components and actions listed in Table 3.4, except where it is demonstrated by experimental evidence and analysis that other inelastic action is acceptable for the selected performance level.

3.1.18. Stiffness for Linear Static and Dynamic Procedures

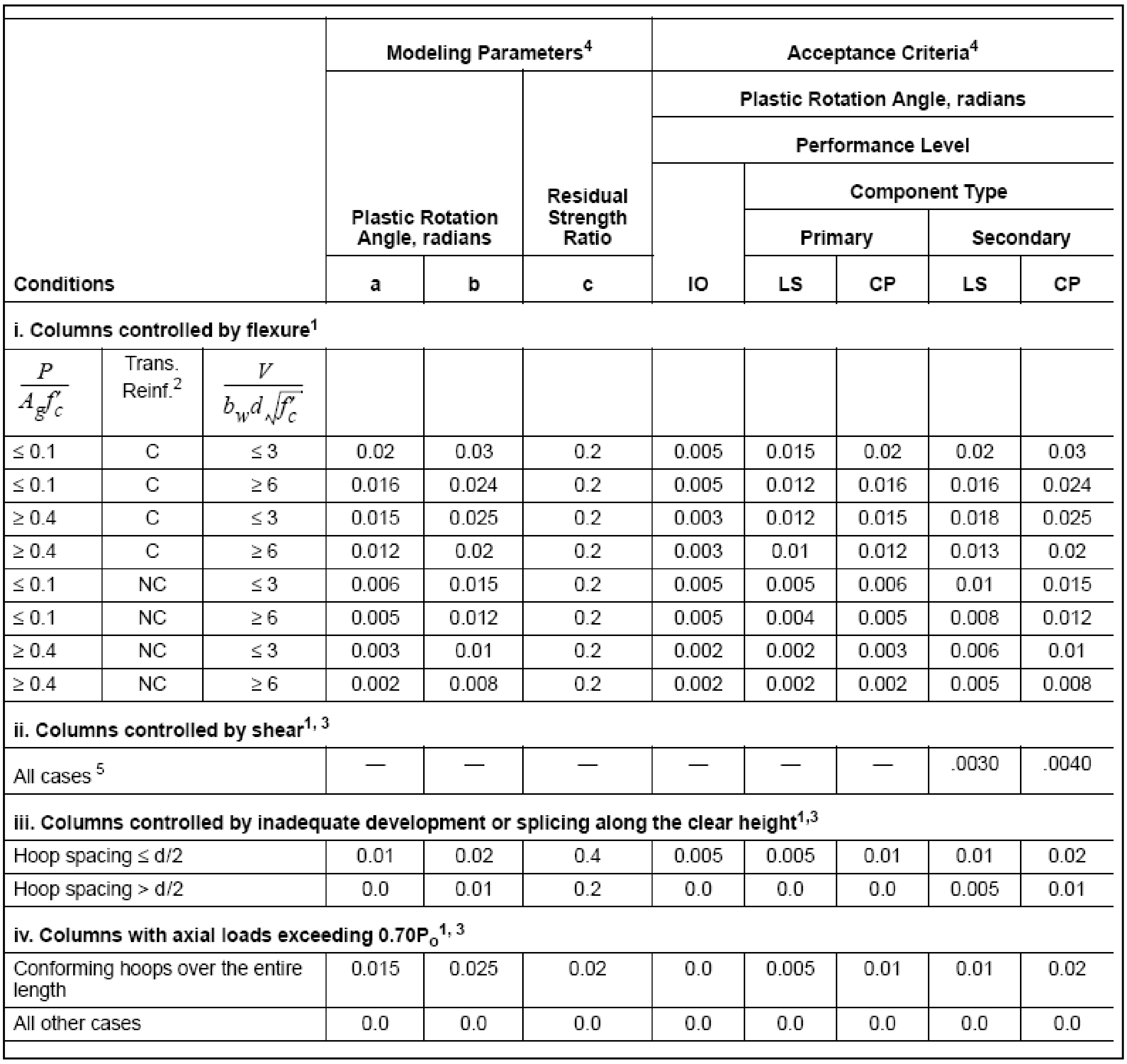
Beams shall be modeled considering flexural and shear stiffnesses, including the effect of the slab acting as a flange in monolithic construction. Columns shall be modeled considering flexural, shear, and axial stiffnesses. Joints shall be modeled as either stiff or rigid components.

3.1.19. Nonlinear Static Procedure

Beams and columns shall be modeled using concentrated plastic hinge models or distributed plastic hinge models. Other models whose behavior has been demonstrated to represent the behavior of reinforced concrete beam and column components subjected to lateral loading shall be permitted. The beam and column model shall be capable of representing inelastic response along the component length, except where it is shown by equilibrium that yielding is restricted to the component ends. Where nonlinear response is expected in a mode other than flexure, the model shall be established to represent these effects. Monotonic load-deformation relations shall be according to the generalized load-deformation relation, except that different relations shall be permitted where verified by experiments. The overall load-deformation relation shall be established so that the maximum resistance is consistent with the design strength.

For beams and columns, the generalized deformation shall be either the chord rotation or the plastic hinge rotation. For beam-column joints, the generalized deformation shall be shear strain. Values of the generalized deformation at points B, C, and D shall be derived from experiments or rational analyses, and shall take into account the interactions between flexure, axial load, and shear.

Table 3.4. Modeling parameters and numerical acceptance criteria for nonlinear procedures (FEMA 356)



			Modeling Parameters[4]			Acceptance Criteria[4]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
Conditions			a	b	c	IO	LS	CP	LS	CP
i. Columns controlled by flexure[1]										
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03
≤ 0.1	C	≥ 6	0.016	0.024	0.2	0.005	0.012	0.016	0.016	0.024
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.003	0.012	0.015	0.018	0.025
≥ 0.4	C	≥ 6	0.012	0.02	0.2	0.003	0.01	0.012	0.013	0.02
≤ 0.1	NC	≤ 3	0.006	0.015	0.2	0.005	0.005	0.006	0.01	0.015
≤ 0.1	NC	≥ 6	0.005	0.012	0.2	0.005	0.004	0.005	0.008	0.012
≥ 0.4	NC	≤ 3	0.003	0.01	0.2	0.002	0.002	0.003	0.006	0.01
≥ 0.4	NC	≥ 6	0.002	0.008	0.2	0.002	0.002	0.002	0.005	0.008
ii. Columns controlled by shear[1, 3]										
All cases [5]			—	—	—	—	—	—	.0030	.0040
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]										
Hoop spacing ≤ d/2			0.01	0.02	0.4	0.005	0.005	0.01	0.01	0.02
Hoop spacing > d/2			0.0	0.01	0.2	0.0	0.0	0.0	0.005	0.01
iv. Columns with axial loads exceeding 0.70P_o[1, 3]										
Conforming hoops over the entire length			0.015	0.025	0.02	0.0	0.005	0.01	0.01	0.02
All other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

3.1.20. Acceptance Criteria for Linear Static and Dynamic Procedures

All actions shall be classified as being either deformation-controlled or force-controlled. In primary components, deformation controlled actions shall be restricted to flexure in beams (with or without slab) and columns. In secondary components, deformation-controlled actions shall be restricted to flexure in beams (with or without slab), plus restricted actions in shear and reinforcement development. All other actions shall be defined as being force controlled actions. Where the calculated DCR values exceed unity, the following design actions shall be determined using limit analysis principles as: (1) moments, shears, torsions, and development and splice actions corresponding to development of component strength in beams and columns; (2) joint shears corresponding to development of strength in adjacent beams and columns; and (3) axial load in column and joints, considering likely plastic action in components above the level in question.

Design actions shall be compared with design strengths. Where the average DCR of columns at a level exceeds the average value of beams at the same level, and exceeds the greater of 1.0 and m/2 for all columns, the level shall be defined as a weak column element. For weak column elements, one of the following shall be satisfied.

- The check of average DCR values at the level shall be repeated, considering all primary and secondary components at the level with a weak column element. If the average of the DCR values for vertical components exceeds the average value for horizontal components at the level, and exceeds 2.0, the structure shall be reanalyzed using a nonlinear procedure, or the structure shall be rehabilitated to remove this deficiency.
- The structure shall be reanalyzed using either the NSP or the NDP.
- The structure shall be rehabilitated to remove the weak story.

3.1.21. Nonlinear Static and Dynamic Procedures

Where the generalized deformation is taken as rotation in the flexural plastic hinge zone in beams and columns, the plastic hinge rotation capacities shall be as defined by Table 3.5. For columns designated as primary components and for which calculated design shears exceed design shear strength as defined, the permissible deformation for the Collapse Prevention Performance Level shall not exceed the deformation at which shear strength is calculated to be reached; the permissible deformation for the Life Safety Performance Level shall not exceed three quarters of that value. Where inelastic action is indicated for a component or action not listed in these tables, the performance shall be deemed unacceptable. Alternative approaches or values shall be permitted where justified by experimental evidence and analysis.

Table 3.5. Numerical acceptance criteria for linear procedures (FEMA 356)

			***m*-factors**[4]				
			Performance Level				
				Component Type			
				Primary		**Secondary**	
Conditions			**IO**	**LS**	**CP**	**LS**	**CP**
i. Columns controlled by flexure[1]							
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$					
≤ 0.1	C	≤ 3	2	3	4	4	5
≤ 0.1	C	≥ 6	2	2.4	3.2	3.2	4
≥ 0.4	C	≤ 3	1.25	2	3	3	4
≥ 0.4	C	≥ 6	1.25	1.6	2.4	2.4	3.2
≤ 0.1	NC	≤ 3	2	2	3	2	3
≤ 0.1	NC	≥ 6	2	1.6	2.4	1.6	2.4
≥ 0.4	NC	≤ 3	1.25	1.5	2	1.5	2
≥ 0.4	NC	≥ 6	1.25	1.5	1.75	1	1.6
ii. Columns controlled by shear[1,3]							
Hoop spacing ≤ d/2, or $\frac{P}{A_g f'_c} \le 0.1$			–	–	–	2	3
Other cases			–	–	–	1.5	2
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]							
Hoop spacing ≤ d/2			1.25	1.5	1.75	3	4
Hoop spacing > d/2			–	–	–	2	3
iv. Columns with axial loads exceeding $0.70P_o$[1,3]							
Conforming hoops over the entire length			1	1	2	2	2
All other cases			–	–	–	1	1

4. ECCENTRICALLY BRACED FRAMES

In the below typical configuration of eccentrically braced frames has been monitored. The crucial components of EBF system are link, brace and beam respectively. EBF has recently emerged as the braced framing system of choice in regions of high seismicity. Provisions for the analysis, design and detailing of EBF have been introduced into seismic regulations and guidelines. The high elastic stiffness of the CBF and the ductility and stable energy dissipation capacity of the moment resisting frame are characteristics of the EBF.

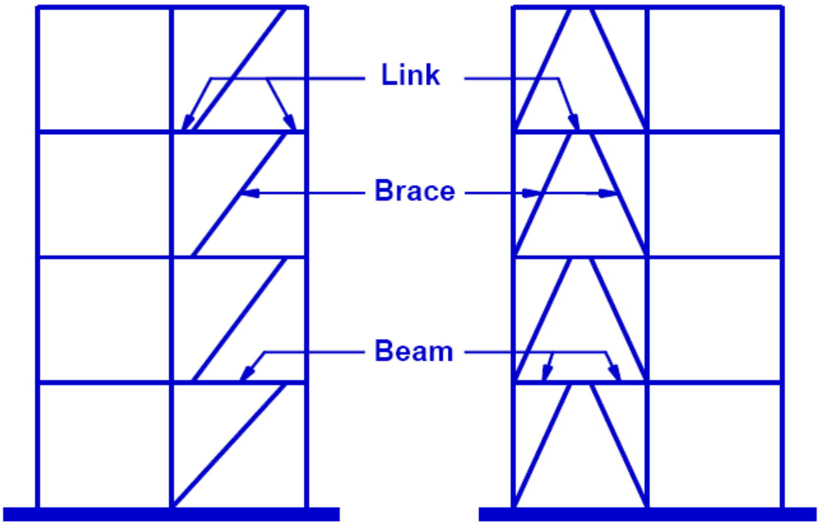


Figure 4.1. Typical eccentrically braced frames and major components

4.1. Introduction into EBF systems

The eccentrically braced frame (EBF) is a hybrid lateral force-resisting system composed of two conventional framing systems:

- The moment resisting frame
- Concentrically braced frame (CBF)

Figure 4.2. Example of EBF application (SEAOC 2000)

The EBF serves to combine many of the individual advantages of each conventional framing system and minimizes their respective disadvantages.

Specially, EBF possess:

- High elastic stiffness.
- Stable inelastic response under cyclic lateral loading.
- Excellent ductility and energy dissipation capacity.

The key distinguishing feature of an EBF is that at least one end of each brace is connected so as to isolate a segment of beam called a link. Common EBF arrangements are illustrated in each figure the links are identified by link length e. the split-K configuration is particularly advantageous because of symmetrical configuration and because the links are not directly connected to the columns, thus avoiding the problems associated with developing full moment connections to the column (Youssef, 2006).

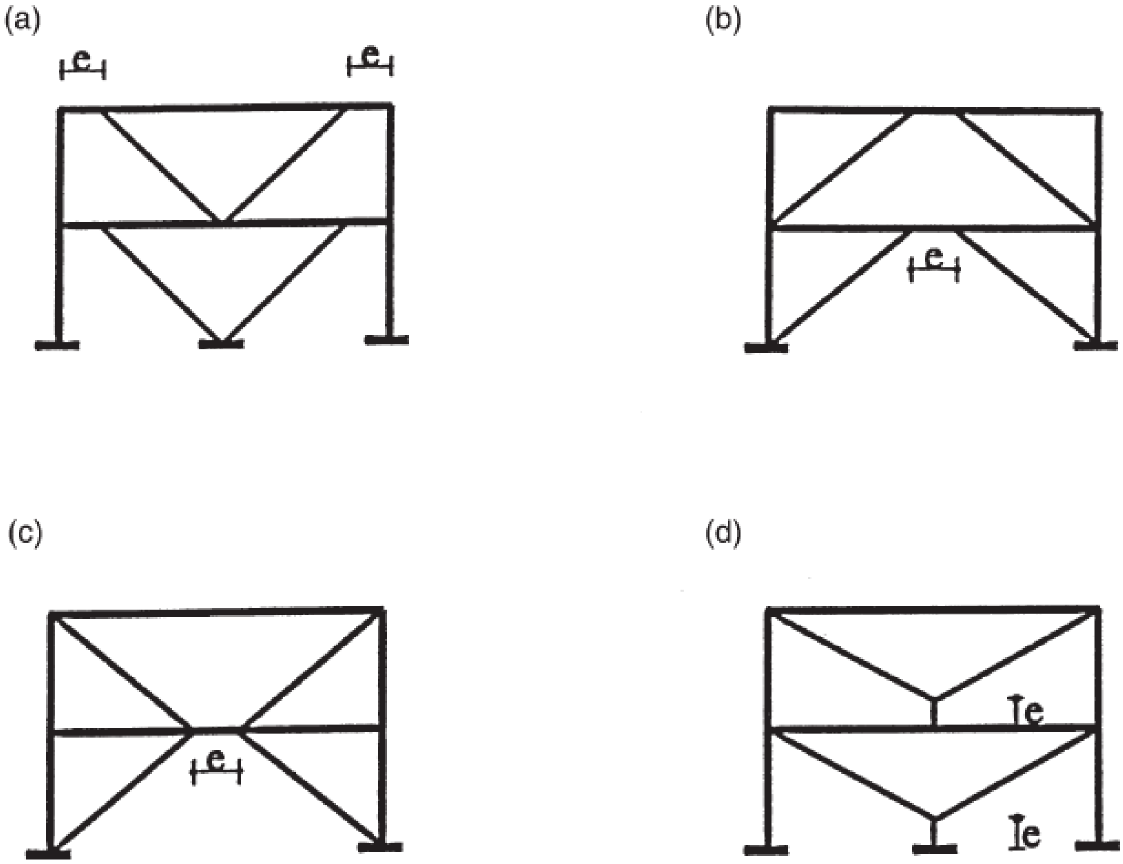


Figure 4.3. Typical EBF Configurations a) V-type with two links b) inverted V-type with single link c) X-type d) Y-type with vertical link

Eccentric braced steel frames (EBF) are very efficient structures for resisting earthquakes as they combine the ductility of that is characteristic of moment frames and the stiffness associated with braced frames. In the EBF inelastic activity is confined to a small length of the floor beams which yields mostly in shear (therefore called the shear link). Capacity design approach is followed in an attempt to limit the inelastic activity to the shear links only while all other frame members are designed to behave elastically (Ambrose, 1998).

Figure 4.4. Example of EBF application (SEAOC 2000)

4.1.1. EBF Design Philosophy

The EBF design concept is simple and straight forward; restrict the inelastic action to the links, and design the framing around the links to sustain the maximum forces that can be delivered by the links.

Design using this strategy should ensure that the links act as ductile seismic fuses and preserve the integrity of the surrounding seismic framing. This concept of capacity design provides an elegant means of limiting forces in selected framing components. For design using linear static procedures, links are typically proportioned for forces substantially smaller than those calculated by analysis for elastic response. Other components in the seismic framing system are then designed for the forces generated by the fully yielded and strain-hardened links. That is all other elements are designed for the capacity of the connected links.

The key point of EBF design is to limiting inelastic actions in the link beam and key to the success of this approach is the deformation capacity of the link. Links must be properly designed and detailed to achieve the large requisite plastic deformations. A simple relationship between frame shear force and link shear force can be developed for common EBF configurations for the purpose of preliminary design. The relationships depend only on frame geometry and are independent of whether the link response is elastic or inelastic. Where P is the lateral force, h is the story height and L is the width of braced bay, for the split-K-braced EBF and assuming that the moment at the center of the link is equal to zero, the link shear force (V_L) can be estimated as:

$$V_L = \frac{Ph}{L} \tag{4.1}$$

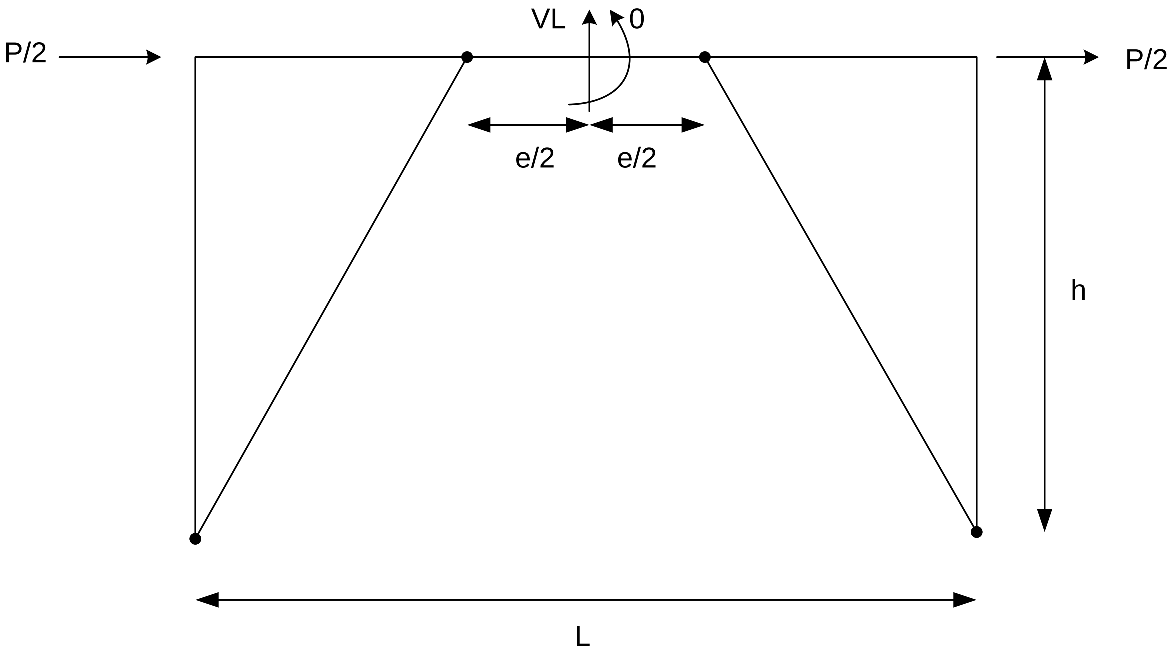


Figure 4.5. Forces appeared in the link

The relationship between story plastic drift (Δ_p) and link plastic rotation (γ_p) can be simply derived. Recognizing that elastic deformations in the framing outside the links are small compared with the plastic deformations, the framing outside the links can be assumed to be rigid.

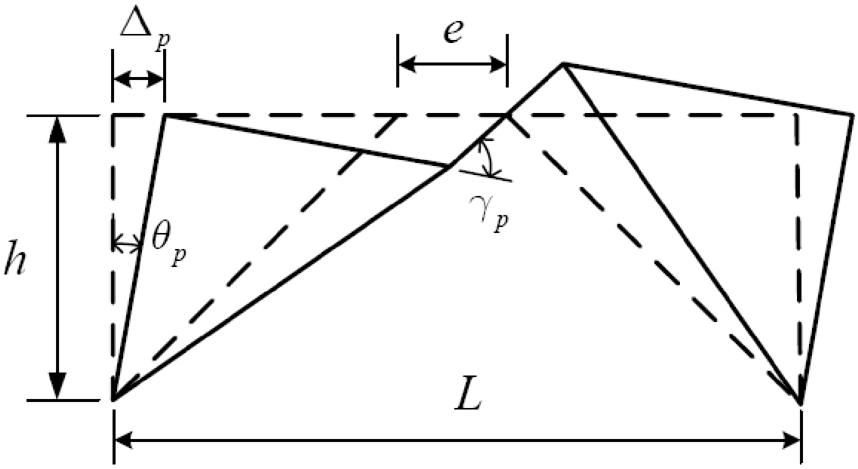


Figure 4.6. Rotation of link element under lateral effect

$$\Delta_p = \frac{\gamma_p e h}{L} \tag{4.2}$$

$$\gamma = \frac{L}{e}\theta \tag{4.3}$$

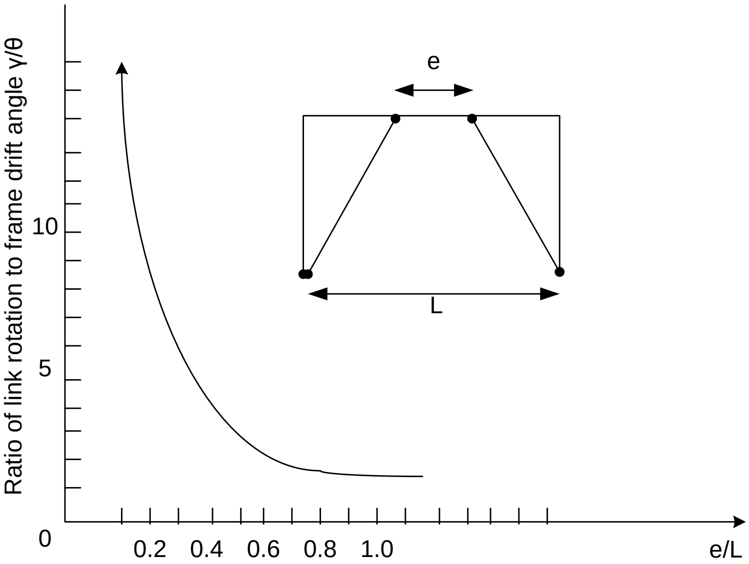


Figure 4.7. The relation of link rotation to frame drift

Link rotation demand grows quickly as the link length decreases with respect to the frame dimension L. The limiting case of e/L equal to 1.0 corresponds to a moment-resisting frame; that is g/q approaches 1.0 as e/L approaches 1.0. Although large link rotation demands can be realized by short (shear) links, figure above demonstrates that links should not be too short, or else the link rotation demands will become excessive even for a shear yielding link.

4.1.2. Link Behavior and Length

Link plastic rotation angle (γ_p) can be easily estimated by frame geometry assuming rigid-plastic behavior of the frame members. The relationship between plastic story drift angle and link plastic rotation angle for the three types of EBFs was determined in previous part. Depending on the section properties of the link, link may yield either in shear extending over the full length of the link or in flexure at the link ends, or the combination of shear and flexural yielding. Note that link plastic rotation angle is the same whether the link yields in shear or in flexure.

The length and geometry of the links in an EBF will dictate the behavior of the frame. Short links are shear-critical and long links are flexure-critical. Architectural concerns with the use of short links are similar to those associated with the use of concentric braces. Long links have architectural and planning advantages by providing significant openings in the frame for doors, windows, and mechanical equipment. The link in an EBF is designed to act as a ductile element and act as a fuse for the structure, and dissipating much of the energy input to the building by the earthquake.

The most critical factor affecting the inelastic behavior of a link is its length. Link length controls the yielding mechanisms and the ultimate failure mode.

For short links, shear dominates the link response; for longer links, flexure controls link response. The figure below illustrates typical distributions of actions, axial force, shear force and bending moment in the EBF systems under lateral loads. For very short links, the link shear force reaches the plastic shear capacity before the end moments reach the plastic moment capacity, and the link yields in shear, forming a shear hinge or shear link.

$$V_p = 0.55 d t_w F_y \tag{4.4}$$

$$M_p = ZF_y \tag{4.5}$$

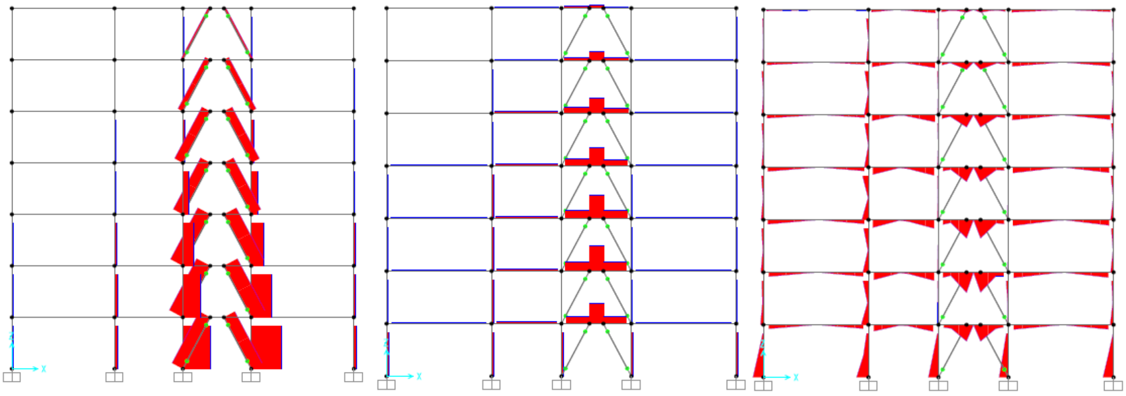


Figure 4.8. Contour of the forces appeared in the link and surrounded elements under lateral effects

For longer links, the end moments reach Mp, forming flexural hinges before shear yielding can occur; these links are termed flexural links. Material strain hardening can substantially influence link behavior, and both shear and flexural yielding can occur over a wide range of link lengths. Rules have been developed to guide designers' selection of sections to ensure shear yielding. The following equations for the link length (e) can be used to classify links:

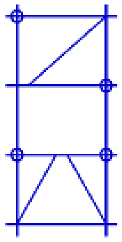
$$Short\ Links\,(shear) \qquad e \le \frac{1.6M_p}{V_p} \tag{4.6}$$

$$Intermediate\ Links \qquad \frac{1.6M_p}{V_p} < e < \frac{2.5M_p}{V_p} \tag{4.7}$$

$$Moment\,(long)\,Links \qquad e \ge \frac{2.5M_p}{V_p} \tag{4.8}$$

Some designers prefer EBF systems without taken moment connections at ends of the beam connected, such choice may act a little far from the design philosophy of EBF systems. However at design codes and guidelines for such conditions different and reduced seismic modification factors are valid. R and C_d values for eccentric bracing systems vary depending on moment resisting capacity of connections away from link beam; locations identified in diagram. Connections at links must be fully restrained.

Table 4.1. Referred R and C_d Factors of EBF systems in structural engineers association of California (SEAOC, 2000)

Eccentric Bracing System	R	Cd	
Building frame system or part of dual system w/ special moment frame			
With moment resisting connections at coloumns from links	8	4	
Without moment resisting connections at coloumns from links	7	4	

4.1.3. Design Procedure of EBF systems

Basic design procedure and special requirements for eccentrically braced frames (EBFs) is:

- Elastic Analysis
- Check rotation angle; re-proportion as required
- Design check for strength
- Design connection details

Elastic analysis of frame determines member forces and elastic deflections; inelastic deflection determined using C_d. EBF deformed this inelastic amount as though if rigid, ideally plastic mechanism; rotation angle of link beam determined. Columns, beams, and braces designed to remain elastic at force level that yields link beam; approximates behavior of rigid, ideally plastic mechanism. Ductility assured and buckling prevented by designing stiffeners and connections at link beam to accommodate link beam rotation in mechanism.

5. CASE STUDY

5.1. Introduction

The aim of the study is to highlight the contribution of eccentrically braced frames as a retrofitting strategy used for strengthening existing reinforced concrete buildings in Istanbul. The study consists of three singular buildings, and these are buildings in which seismic loads are fully resisted by frames. Since the main purpose is to bring out the influence of EBF systems on existing buildings, typical structures were selected for inspecting and keeping the symmetry for having most straight results was of consideration. Most of these structures have been designed and constructed using former specifications and codes like as “AFET BÖLGELERİNDE YAPILACAK YAPILAR HAKKINDA YÖNETMELİK_1975”. Even though there were such codes which were not so meaningful from seismic point of view, most designers at that time disobey those codes and came up with the designs which depended mostly on the experience of designer for seismic effects.

The scope of this study is defined as the assessment of the structural risk-to-life-safety to Collapse Prevention response levels and the risk associated with the egress routes to the accepted levels of Life Safety as defined in the Federal Emergency Management Agency document - FEMA 356, in order to re-classify or to retrofit the structures so that they can be re-classified in the Low Risk category according to the aim of the project.

This report addresses the preliminary assessment of the buildings listed above for a design earthquake with a 10% probability of exceedance in 50 years. Analyses done and results reported in the following sections are related only to the structural components of the buildings, and do not include the aspects related to the safety of equipment, installations or cladding to be attached to or supported by the structural components. Therefore, this study does not investigate the condition of non-structural elements, installations and equipment housed in the structures after the event of an earthquake.

Three types of the buildings are low-rise, mid-rise and high rise reinforced concrete buildings. It has been assumed that the original structural and architectural design drawings are not available, so knowledge factor could be taken as 0.75 for considering such buildings in Istanbul and taken into account the worst condition of assessment. During preparing draft drawings of the buildings the matter of symmetry was of priority. The existing concrete compressive strength and the yield strength of the reinforcing bars are taken as C16 and S220. The reason under such selection was to keeping the buildings designed according to Turkish Design Code 1975, in other hand keeping the present situation of existing buildings in Istanbul as realistic as far as possible.

Usually, when steel classification of an existing building is examined, visual inspection and exposure results are privileged for engineers. After inspections and experiences upon existing buildings in Istanbul, it was assumed to take S220 for proceeding. The minimum reinforcement ratio and the ultimate load design procedures are the main options for considering the design of an exiting building.

During the selection of the buildings the following articles were considered as much as possible:

- Most of the existing buildings in Turkey have been designed using former code in 1975
- The code drafted in 1975 is not a best ideal code from seismic point of view
- The designers did not comply with the code
- Most of the designs based on the experience of the designer upon seismic loads
- There are some structures which even do no meet the gravity conditions
- During the construction there were infirmity of detailing
- Negligence on the material, the quality of concrete and steel

After including the above factors the highlights were drafted, three types of the structures were chosen as;

- Low rise buildings, 4 story

- Mid rise buildings, 7 story
- High rise buildings, 10 story

These buildings on-plan geometry is same and the only difference is the number of stories, the reason of such selection is to examine the effect of gravity matter on the behavior of EBF systems.

5.1.1. Assessment Methodology

Based on the availability of original design data, following scheme is adopted to conduct the assessment of the buildings on site.

- Geological Desktop Study, all buildings have been assumed to be on the soil classification D.
- Visual inspection of the buildings in Istanbul. .
- Specification of materials testing on field and/or laboratory examination, this part also has been assumed as C16 for concrete and S220 for the steel used in buildings.
- Detailed Assessment (Maheri and Hadjipour, 2003).

In the following sections the building will be investigated for the performance based design process according to the Linear Dynamic analysis and Nonlinear Static Analysis, which are explained in FEMA 356. Then, if the buildings are decided that incapable to comply with the desired performance levels or to resist the earthquake ground motion, they would be strengthened by EBF systems and this strategy would be compared to CBF systems.

5.1.2. Buildings Description

The structural form Low, Mid and High rise buildings are rectangular. The structure have respectively 4, 7 and 10 stories above ground started as 1st, 2nd and roof floors. The overall on plan sizes of the structure is 20x18 m on a mesh of 4x5 gridlines (E-W dir x N-S dir) with 4m intervals and nominally 6 m intervals respectively. In section, the nominal floor height is 3.00 m.

The structural system of the building is moment resisting frame. The first and the last span along the length of the structure in x-direction is 6 m and the rest are 4 m. and the span lengths in y-directions are 6 m. There is no reinforced concrete shear walls along the height of the structure there are some infill walls made of unreinforced masonry infill in interior part of the structures as architectural meanings for parting the rooms.

All columns are square formed and the reinforcement ratio is between 0.75-1.00 in accordance with the 1975 Turkish code. Column sizes vary between 500x500, 400x400, 350x350, 600x600mm.all beams were selected to be 300mm in width and 500 as depth, the reason is that at this stage the performance of the building is of interest so local mechanism is not of interest at preliminary procedure. The stirrups are Ø8 at a spacing of 250 mm provided along the entire length of the column. For the concrete cover; the clear cover is 20 mm and cover to the center of longitudinal reinforcement is taken as approximately 40 mm for all columns. In particular after retrofitting the system by EBF, the behavior of the beam especially in link region would be of considerable interest from ductility and strength points of view.

The floor type is joist system and the nominal spacing of the joists is 500 mm. The direction of the slabs is in W-E direction; theoretically the slabs are one way slab.

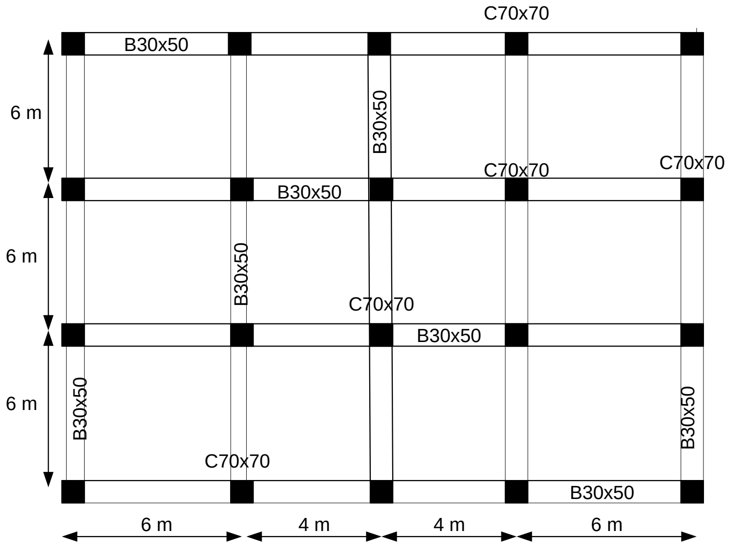


Figure 5.1. Typical plan view of low, mid and high rise buildings

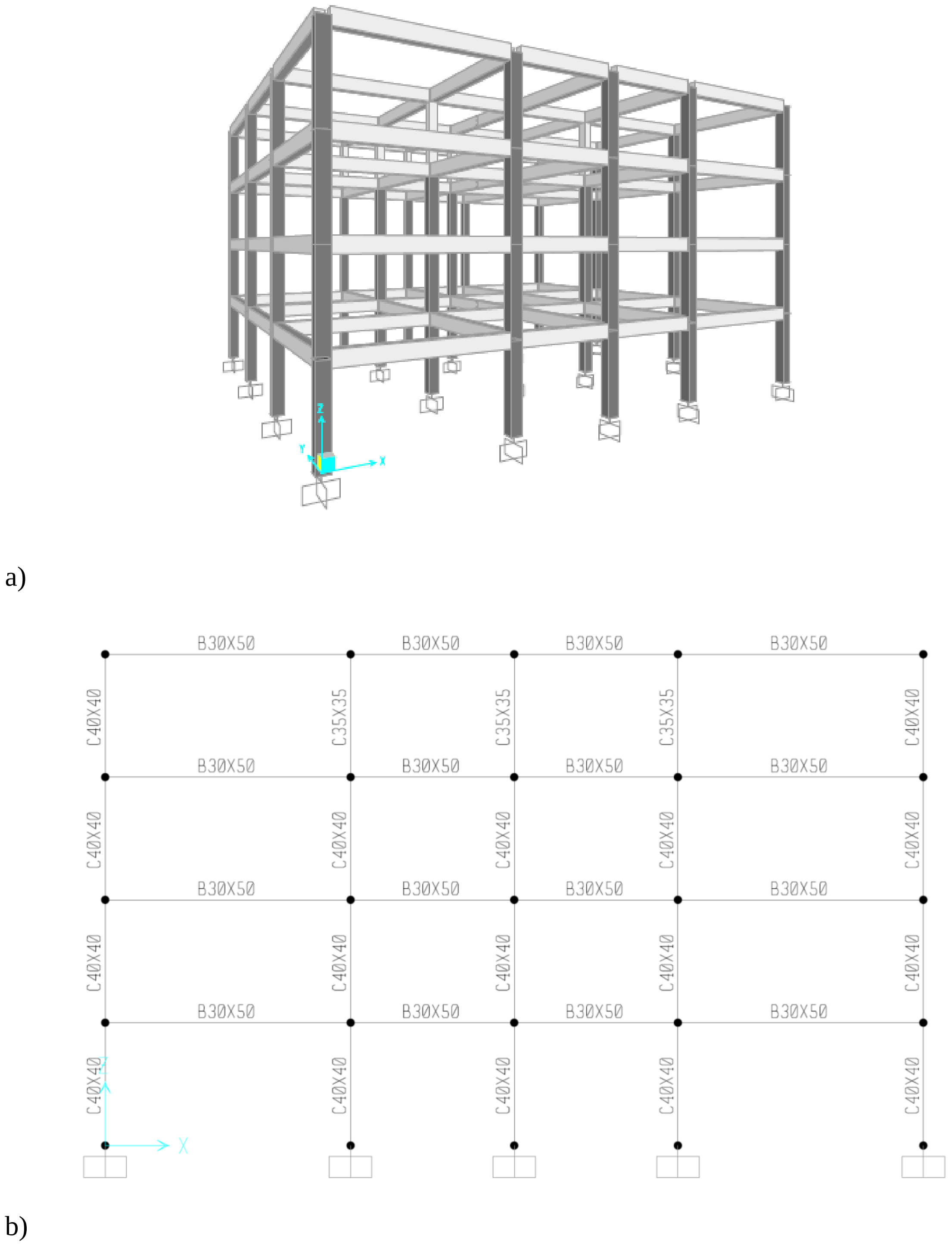
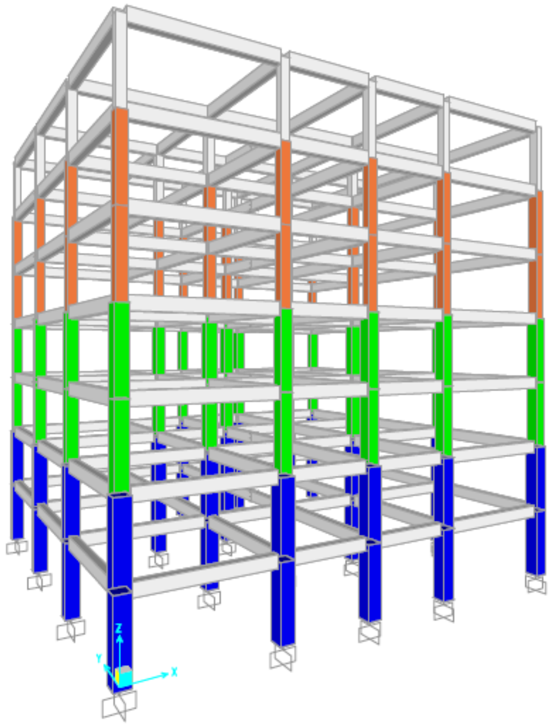
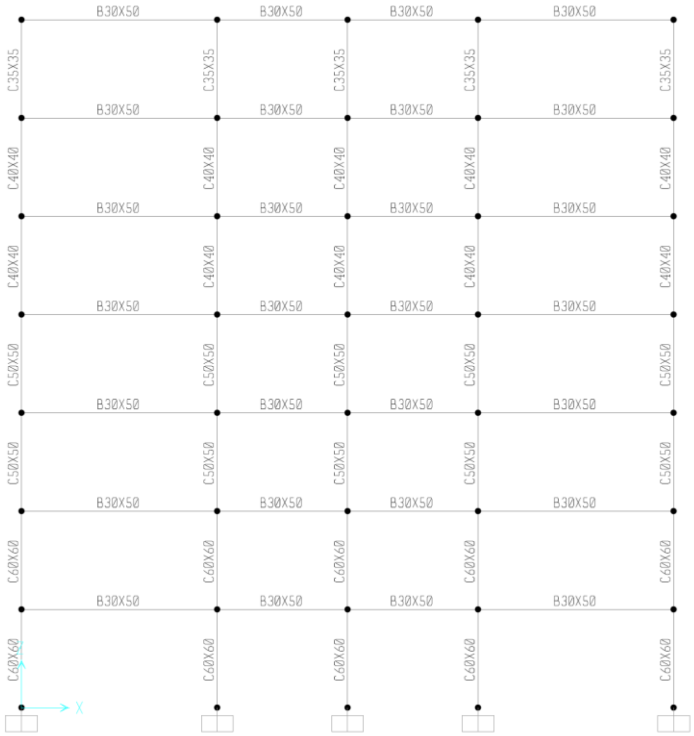


Figure 5.2. Sections of low-rise designed according to 1975 Turkish Code

a) 3D perspective view of low-rise building b) Assigned sections and dimensions



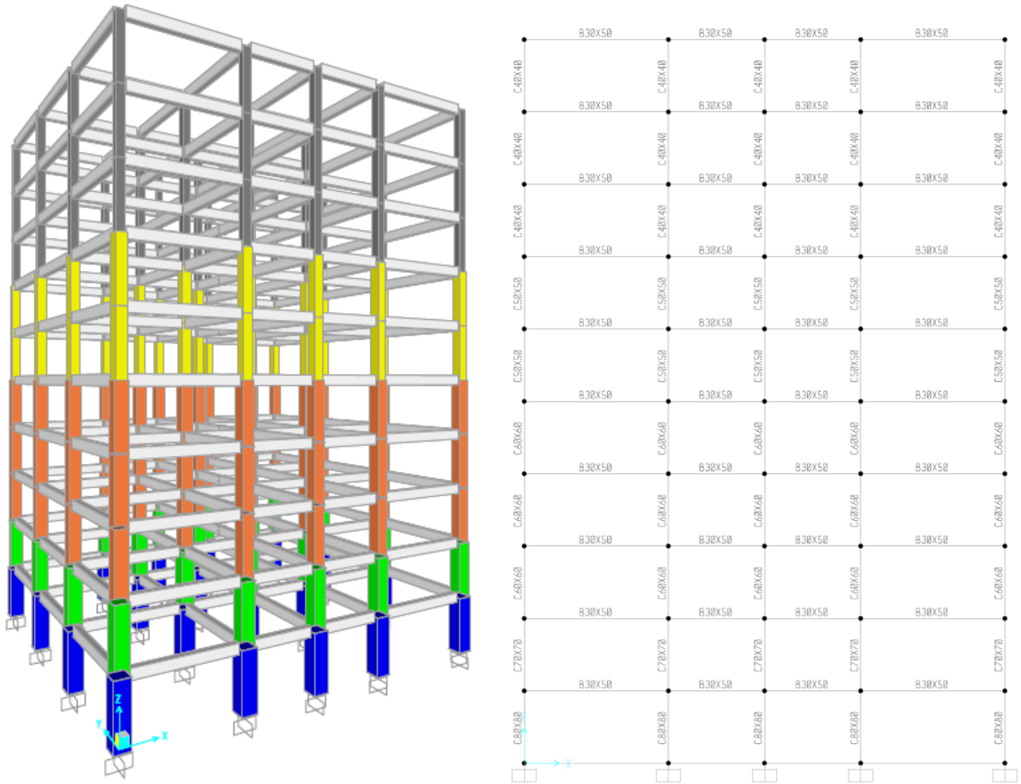
a)



b)

Figure 5.3. Sections of mid-rise designed according to 1975 Turkish Code

a) 3D perspective view of mid-rise building b) Assigned sections and dimensions



a) b)

Figure 5.4. Sections of high-rise designed according to 1975 Turkish Code
a) 3D perspective view of high-rise building b) Assigned sections and dimensions

5.2. Description of approaching

At this study for seeing the results easily some names and abbreviations have been used for calling the structures and the processes. "R" is the abbreviation of "Retrofit" and the numbers coming after this word mean the step in which the analysis is placed. So, the existing buildings which have undergone no retrofit application are R00, which represents no retrofit. At the same time the numbers present of the method of retrofitting, in which at this study there are only two methods, EBF and CBF methods and in the last phase jacketing the columns.

R00 = the existing structures.
R1A = the structures retrofitted by EBF systems as configuration number one.
R2B = the structures retrofitted by CBF systems as configuration number two.

R2C = the structures retrofitted by EBF systems as configuration number two.

R3A = the structures retrofitted by EBF systems as configuration number two and optimized by column jacketing.

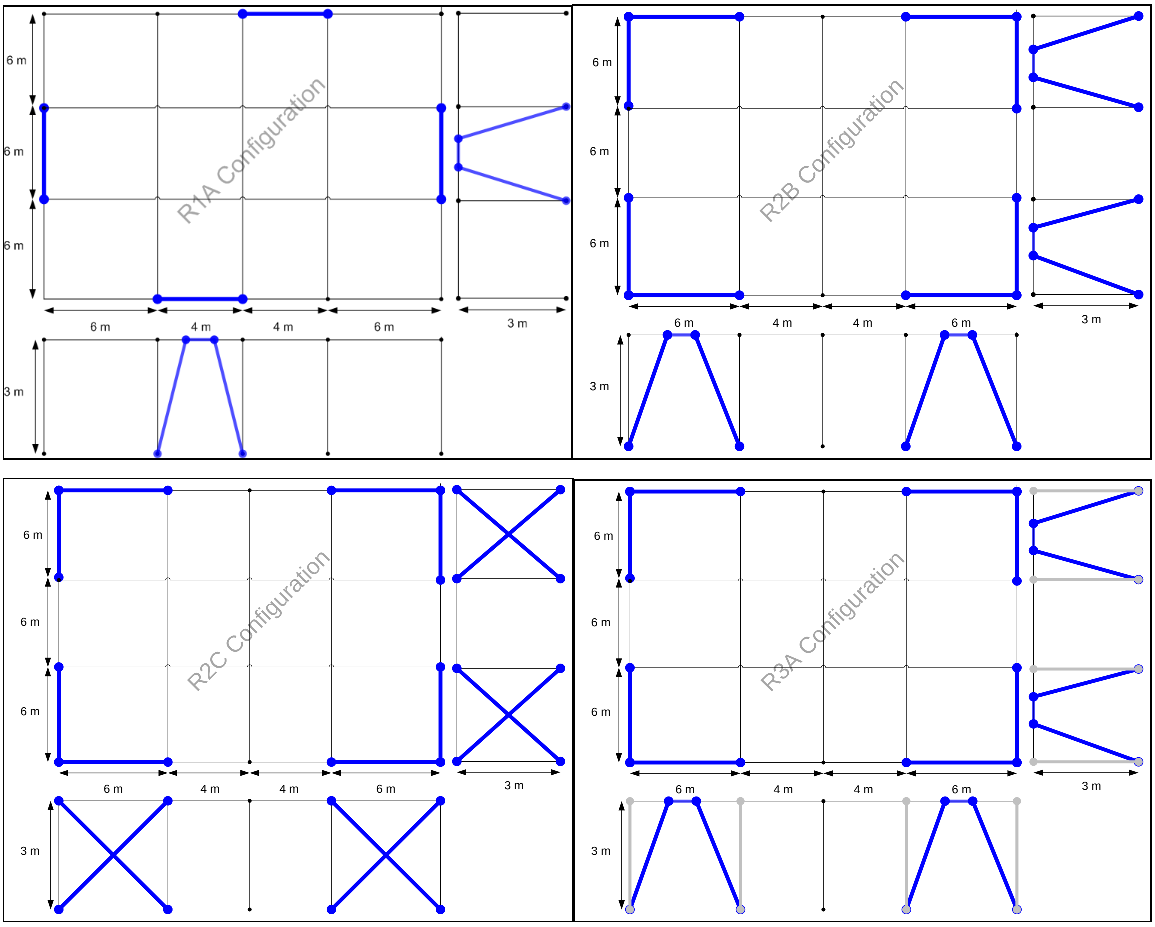


Figure 5.5. R1A, R2B, R2C and R3D configurations

5.2.1. Modeling the Structures

The structural systems have been modeled and analyzed 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. All floors have been defined as rigid diaphragm. The end offsets have been also assigned to beams automatically however for columns at j-end the offset value has been put manually as the depth of the connected beam has been considered. The concrete class and reinforcing steel class have been chosen as C16 (compressive strength of concrete f_{ck} = 16 MPa) and S220 (f_{yk} =220 MPa) respectively, however expected values have been put into the program as 1.5 multiplier is for concrete and 1.15 for the steel yield strength according to ACI318-05. According to FEMA 356 soil classification and seismicity region, it has been assumed that building is located on first seismic zone and SD soil type.

5.2.2. Structural Condition Surveys

For taking into consideration the most hazardous condition in Turkey, the below spectrum was selected for analyzing and assessing the buildings designed by 1975 code. As it can be easily observed is that the constant acceleration region starts from T=0.12, the reason beyond this is come up from the characteristics of FEMA 356 special for selected seismic region and soil classification, the plateau is ended at T_S=0.6 the following spectrum have been carried out for 10% probability of exceedance in 50 years.

$$S_1 = 0.6 \tag{5.1}$$

$$S_S = 1.5 \tag{5.2}$$

$$F_a = 1.0 \tag{5.3}$$

$$F_v = 1.5 \tag{5.4}$$

$$S_{XS} = F_a S_S = 1.0 \times 1.5 = 1.5 \tag{5.5}$$

$$S_{X1} = F_v S_1 = 1.5 \times 0.6 = 0.9 \tag{5.6}$$

$$T_0 = 0.2 \times T_S = 0.12 \tag{5.7}$$

$$T_S = \frac{S_{D1}}{S_{DS}} = 0.6 \tag{5.8}$$

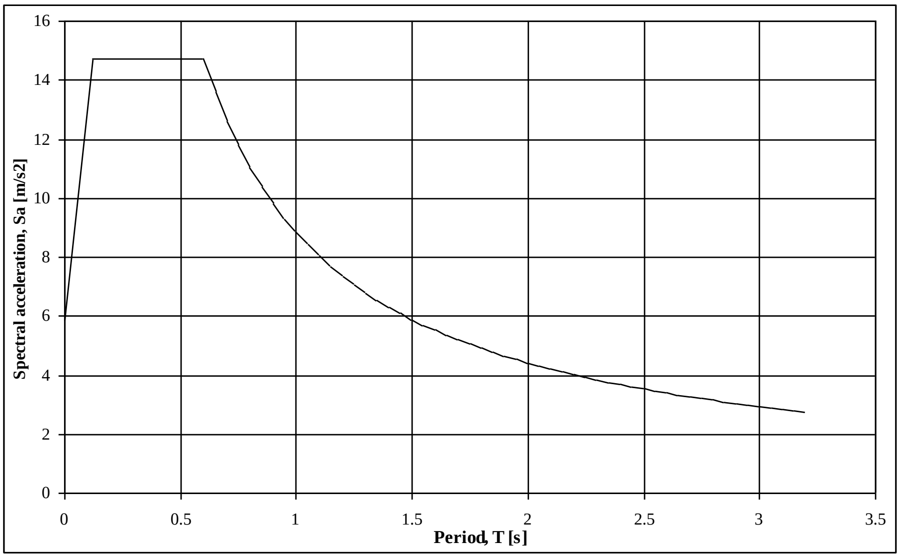


Figure 5.6. FEMA 356 elastic design / evaluation spectra
'For SS =1.50g, S1=0.60g, soil profile SD, Fa=1.00, Fv=1.50, and 5% damping'

Based on the assumption and not any available data the site class is defined as type SD, according to UBC'97 and FEMA codes.

5.2.3. Loading analysis

Vertical loads determined according to ACI31805 and UBC 97 are given below. They are used in determining the elastic seismic loads for finding the desired checks for the structural system model.

All stories,

$$g = 4.50\ kN/m^2 \quad \text{(Including slabs)} \tag{5.9}$$

$$q = 5.00\ kN/m^2 \quad \text{(Including infill walls)} \tag{5.10}$$

Roof story,

$$g = 1.50\ kN/m^2 \tag{5.11}$$

$$q = 0.75\ kN/m^2 \quad \text{(Just snow load)} \tag{5.12}$$

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.

5.2.4. Designing R00 structures in accordance with 1975 Turkish Design Code

After completing the modeling of the structures in SAP2000 program they have been checked and when it is required, they have been designed according to 1975 Turkish Code. Design combinations of 1975 Turkish Code have been a implemented at design stage of existing buildings. It has been tried to carry out reasonable and as far as possible realistic DCR results from the design models of existing structures. As a matter of fact the columns reinforcement ratios were kept between 0.75 – 1.00. The designed columns can be seen from the above figure.

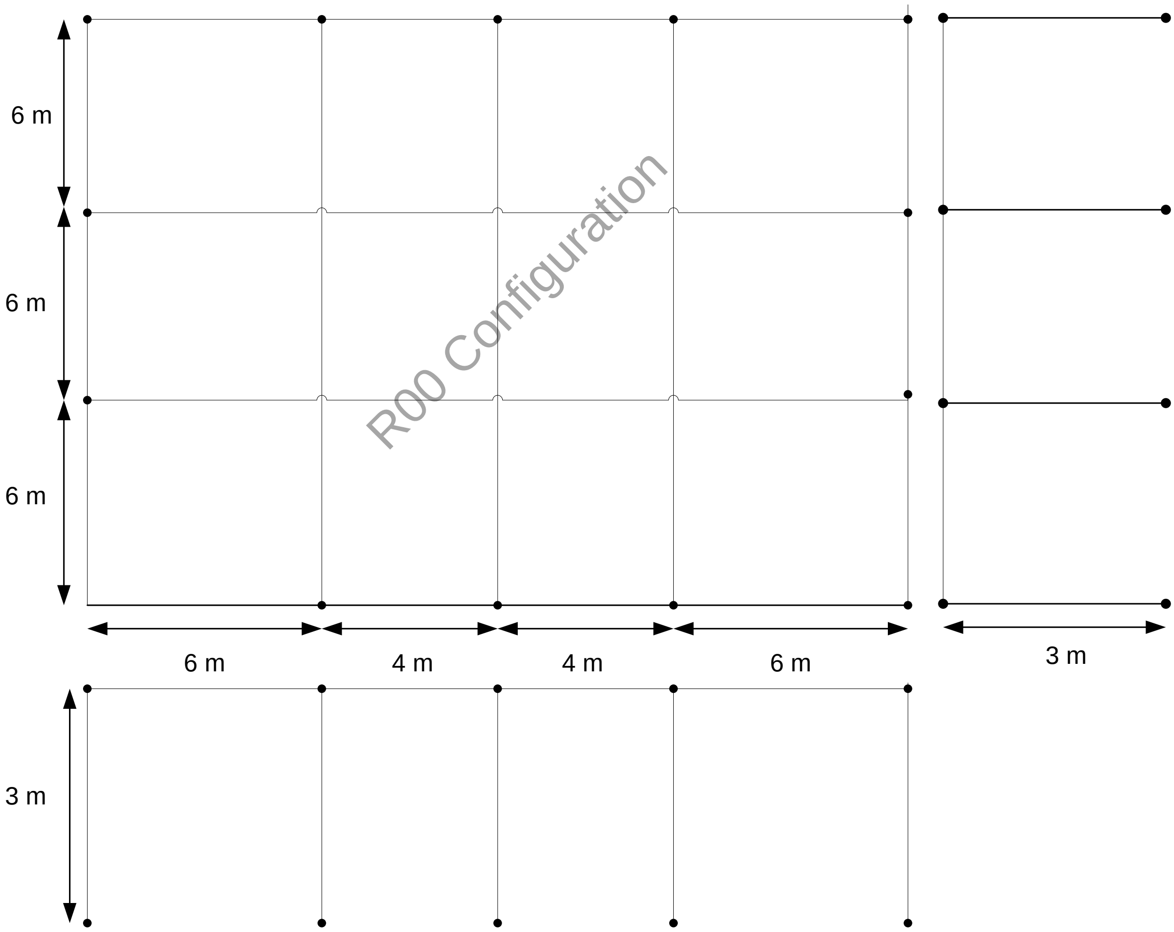


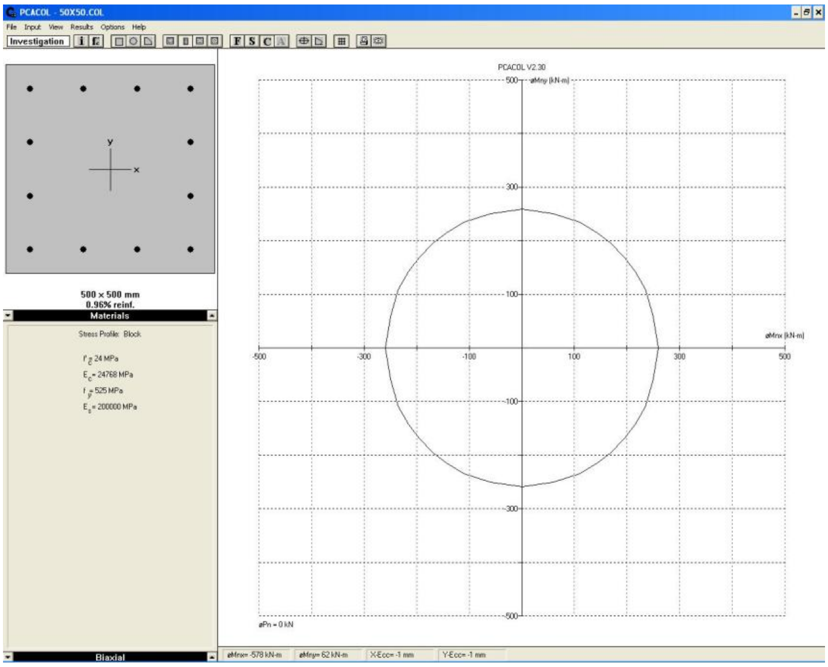
Figure 5.7. R00 configuration (Existing Building)

5.2.5. R00 Detailed Assessment Study

As it was stated before, R00 presents existing buildings. For having a better idea of current condition of low, mid and high rise buildings, they have been examined in accordance with the provisions of FEMA 356. in the assessment phase of the buildings, after completing the models in SAP2000, the required assessment combinations have been put into the model and from the model output analysis have been exported into excel program. At this study only columns of the structures have been taken into account and linear analysis results have been carried out only from columns. At this study three computer programs have been used:

- SAP2000 v.11
- PCACOL
- m-factor program (VB-Excel)

As a matter of fact using default values was avoided as much as possible. So, the capacity of the columns has been derived by using PCACOL program. This program is for deriving moment capacity of different reinforced concrete sections under various axial forces. The interface of the program is as follow;



500 x 500 mm
0.96% reinf.

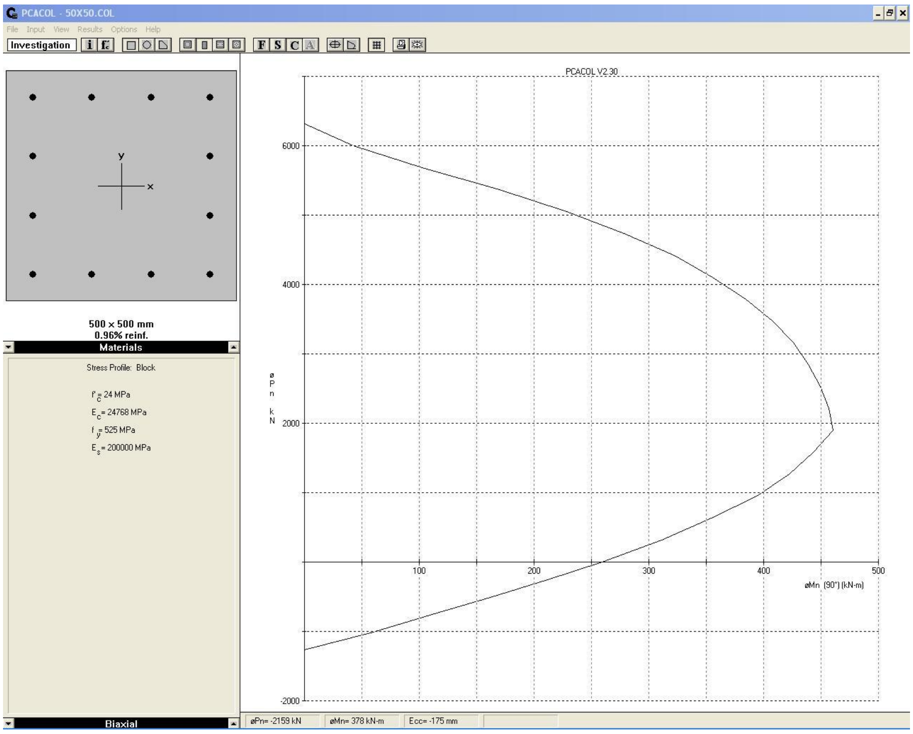
Stress Profile: Block

f'_c = 24 MPa
E_c = 24768 MPa
f_y = 525 MPa
E_s = 200000 MPa

øPn = 0 kN

øMnx= -578 kN-m
øMny= 62 kN-m
X-Ecc= -1 mm
Y-Ecc= -1 mm

Figure 5.8. M2 and M3 axis capacity view of PCA column



500 × 500 mm
0.96% reinf.

Stress Profile: Block

f'_c = 24 MPa
E_c = 24768 MPa
f_y = 525 MPa
E_s = 200000 MPa

øPn= -2159 kN
øMn= 378 kN-m
Ecc= -175 mm

Figure 5.9. P (Axial Load) and M3 capacity view of PCA column

From PCACOL program the moment capacity of the sections under no axial forces have been derived, during exporting of SAP2000 the axial forces of the columns under gravity loads have also been taken out and exported to excel, during assessment analyzing of these columns in excel, each columns' capacity under relevant axial load has been computed.

The second program used for analyzing is a Visual Basic program developed in Excel. The outputs of SAP2000, which mostly are force and moments of the elements under FEMA 356 assessment combinations, and PCACOL results of moment capacity of different sections, are transferred to m-factor program. Using m-factor program which is in summary collected and built up of FEMA 356 provisions, analysis of many elements could be done at the same time and easily.

5.2.6. Detailed Assessment

For the evaluation of the structures the linear procedures specified in FEMA 356 were adopted. Following sections outline the basics related to linear procedures and assumptions regarding the input parameters.

5.2.7. FEMA 356 Assessment Procedures

FEMA 356 assessment procedures using linear analysis methods require the analysis to be undertaken under the elastic spectrum loads and application of base shear modifiers to calibrate the results with the full non-linear analysis methods. In this approach response modification factors are not applied to the design response spectrum, and the force / displacement modifications, and allowance for ductility and over-strength are made at member level. Although it is applied at member level, the knowledge factor, κ, is an important demand modifier, and reflects the reliability of the information on the present state of the material strengths and system details.

Application of FEMA 356 analysis and capacity check rules require separate treatment of brittle and ductile failure modes. It is also crucial to investigate whether a given mode can be reached before the contradicting behavior takes over. An illustrative case would be the failure of a column in shear, under compressive stresses or in a flexural manner, where highest ductility can only be reached if both the axial and shear forces are low and the shear resistance at the column ends allow the development of flexural hinges at member ends. Sets of rules developed in this regard are given in the relevant chapters of FEMA 356, particularly Chapter 6 for reinforced concrete structures. Acceptance criteria tables list the probable failure modes and give the associated m-factors applicable.

In addition to failure mode distinction, it is also required to classify the existing and newly added structural elements as primary or secondary, where the members classified as secondary cannot be allowed to carry more than 25% of the total lateral load, and thus, are assumed adequate at lower performance limits and are granted higher ductility modification factors as compared to the primary elements. Furthermore, primary elements are not allowed to fail in a brittle manner, since this would mean sudden loss of the structures lateral resistance and therefore its stability.

5.2.7.1. Performance Level. Performance level targeted for the low-mid-high rise structures is set at the life safety level for structural elements. This issue requires further discussion and clarification, since some of the structures or extensions that form a part of the egress path may thus be required to have a higher overall performance. At the current stage of the assessment all structures are investigated considering the Life safety scope only.

5.2.7.2. Gravity Loading. Gravity loading of the structure is assigned complying with the rules of the FEMA 356 Chapter 3. Following paragraphs outline the approach used in the modeling of the assumed weight and mass of the structures for structural analysis.

5.2.7.3. Dead Load. Dead loads of the structural elements that are physically represented in the structural models are taken automatically. Mass contribution of these elements is also calculated by the software employed for analysis (SAP2000). Elements that do not contribute to the structural resistance of the structures are modeled as external loads and distributed on the structural elements with respect to their tributary areas and application points. Mass due to this type of dead loads is defined in addition to the above given element mass and assigned at the nearest column end nodes.

5.2.7.4. Live Loads. Live load on the floors was assumed as 5kN/m^2 which is used with 30% contribution to seismic weight of the structure.

5.2.7.5. Snow Load. Snow load on the roof was assumed as 0.75kN/m^2 which is used with 30% contribution to seismic weight of the structure.

5.2.7.6. Seismic Loading. When using linear procedures for analysis, FEMA 356 requires the applied base shear to be modified to account for the displacement, hysteresis, and second order effects explicitly. These modifications are required for deformation controlled structural response, where the non-linear and linear behavior regimes may result in substantially different displacement responses.

5.2.8. Modification Factors

Before starting analysis the elements in m-factor program, required modifications should be done in the Model. The torsional effects of the seismic loads on the diaphragm rigidities of the floors should be influenced on the structures. Likewise, pseudo lateral force modification factors should be computed for $V = C_1 C_2 C_3 C_m S_a W$. These factors should be calculated for each horizontal direction separately. Knowing base shear, story shear, gravity forces at each story and dynamic periods of the structures are necessary for calculating modification factors.

5.2.8.1. Coefficient C1. This coefficient accounts for the observed difference in peak displacement response amplitude for non-linear response as compared with linear response, as observed for buildings with relatively short initial vibration periods. For structures with a fundamental period less than 0.1 sec, this coefficient takes on the value 1.5. If the fundamental period is beyond the second corner period, TS, of the spectrum the value becomes 1.0 and for the range in between interpolation is required.

5.2.8.2. Coefficient C2. This coefficient adjusts design values based on the shape of the hysteresis characteristics of the building. For structures evaluated using linear procedures this coefficient takes on the value 1.0. Thus, no modification of the base shear required.

5.2.8.3. Coefficient C3. P-Δ effects caused by gravity loads acting through the deformed configuration of a building always result in an increase in lateral displacements. If P- Δ effects result in a negative post-yield stiffness in any one storey, such effects may significantly increase the inter-storey drift and the target displacement. The degree by which dynamic P- Δ effects increase displacements depends on a number of parameters and it is difficult to capture dynamic P- Δ effects with a single modification factor. The coefficient C3 of FEMA 356, introduced for this purpose represents a substantial simplification and interpretation of large amount of analysis data. Although regarded as approximate, coefficient C3 is an important base shear modifier. The stability coefficient, θi, is the ratio of the moment due to the eccentric weight of all the stories above a given floor level due to calculated lateral drift under earthquake loading, to the resisting moment formed by the storey sway mechanism at considered level. Its value is found using the highest stability coefficient calculated for all of the stories of a given structure. If this value exceeds 0.33 the structure is considered potentially unstable.

5.2.8.4. Coefficient Cm. This coefficient adjusts the mass participation of the higher modes for mid-rise structures having fundamental period lower than 1.0 second.

5.2.8.5. Coefficient J. Force-delivery reduction factor, J, is a correction for the force-controlled modes of load transfer. It is greater than or equal to 1.0, should be taken as the smallest DCR of the components in the load path delivering force to the component in question. The default values of J equal to 2.0 in Zones of High Seismicity, 1.5 in Zones of Moderate Seismicity, and 1.0 in Zones of Low Seismicity. J must be taken as 1.0 for the Immediate Occupancy Structural Performance Level and in cases where the forces are delivered by components that remain elastic. For the low, mid, and high-rise Building the J-factor is taken as 2.

5.2.9. Base Shear – Linear Static Procedure

Base shear under the elastic lateral forces is defined with the pseudo lateral load equation (3-10) of FEMA 356 Chapter 3. The coefficients defined above are used to scale the spectral acceleration applicable. The formula is repeated below for convenience.

$$V_B = C_1 C_2 C_3 C_m S_a W \tag{5.13}$$

Reference to the value represented by the above given formula has only been done in this report to define the FEMA 356 acceptable level of the performance under Life Safety and to scale down the base shear obtained from the linear dynamic procedure as described below.

Table 5.1. Story forces of existing buildings

		Stroy No	*Story Height (m)*	*Area (m2)*	*Dead Load (kN)*	*Live Load (kN)*	*Stroy Shear Forces in X Direction (kN)*	*Stroy Shear Forces in Y Direction (kN)*
Loe-Rise	R00	*1st*	3	360	2507	1436	2179	1657
		2nd	3	360	2457	1411	1462	819
		3rd	3	360	2383	1389	1373	822
		4th	3	360	1644	214	760	455
		Total	12	1440	8992	4451		
Mid-Rise	R00	*1st*	3	360	2788	1430	5078	4632
		2nd	3	360	2770	1426	2942	1720
		3rd	3	360	2588	1415	2553	1502
		4th	3	360	2584	1415	2920	1862
		5th	3	360	2438	1409	2051	1284
		6th	3	360	2372	1379	1965	1333
		7th	3	360	1633	209	1110	799
		Total	21	2520	17173	8684		
High-Rise	R00	*1st*	3	360	2788	1430	4547	4011
		2nd	3	360	2774	1428	2736	1608
		3rd	3	360	2579	1410	2481	1464
		4th	3	360	2595	1416	3113	2066
		5th	3	360	2438	1400	2410	1583
		6th	3	360	2412	1382	2670	1879
		7th	3	360	1686	230	2008	1385
		8th	3	360	2395	1395	2662	1966
		9th	3	360	2330	1359	2289	1726
		10th	3	360	1600	194	1462	1238
		Total	30	3600	23597	11645		

The story area at all structures is 360 m^2 and the floor of these stories has been assigned as diaphragm in X (U1) and Y (U2) directions. As it can be seen from the table above, self weight (Dead Load) and assumed live load at all stories has been taken almost the same. During distributing the applied loads to the frames the in-situ reinforced concrete slab has been assumed as 2-way, so all connected beams have been considered to carry gravity load.

The aim of preparing the table above is to obtain the story and total base shear of the structures. The story shear is a crucial point to interrogate the seismic behavior of a building. When story shear of a system consisted of only columns as primary components against lateral forces is examined, the columns are expected to carry all appeared lateral force as story shear. In a combination of shear wall and column system, each element that resist about 75% of coming shear force is assumed as primary component and the other as secondary component. Therefore, before looking at the table above it should be stated that the reference point, in other words the scale of differentiating, is story shear.

Since the columns at each story are the only elements for resisting the lateral force, so at developed m-factor program the type of primary has been selected before running the program. In the table above the total seismic weight of the building and therefore the total C coefficient could be carried out:

Table 5.2. Base shear to seismic weight ratio of existing buildings

Structure	Total DL	Total LL	Seismic W	Base Shear X	Base Shear Y	C=V/W	
						X%	Y%
Low	8992	4451	12553	2179	1657	0,17	0,13
Mid	17173	8684	24120	5078	4632	0,21	0,19
High	23597	11645	32913	4547	4011	0,14	0,12

After obtaining the ratio of the base shear to total weight of the buildings, the comments could be derived. The average ratio of two directions in Low-Rise building shows that about 15 percent of the total weight of the building is applied as lateral load, it could be stated that this building could resist against only 15 % of its own weight as lateral load during earthquake. This ratio is 20% for Mid-Rise and 13 % for High-Rise buildings. Although it is so early to make any comment on the seismic performance of the buildings however it can be seen that mid-rise building has more intention to remain stiffer than the other two structures. Even this fact needs to be proved and should not be relied on at this phase of the assessment.

The displacement of center of mass located at the center of floor diaphragms for each building is as shown in table below. This table is necessary for computing C factor and the multiplier of applied load by dynamic analysis. The mentioned factors are also necessary for calculating tortional multiplier due to 5% eccentricity of each diaphragm floor.

In following chapters displacements will be used for drift calculation of each story which is a scale of determining seismic performance level.

Table 5.3. Story displacements of existing buildings

Structure			*Displacement*	
			D_x [mm]	*D_y [mm]*
Low-Rise	*R00*	*1*	55	57
		2	63	67
		3	42	46
		4	19	20
Mid-Rise	*R00*	*1*	27	30
		2	45	51
		3	53	59
		4	46	52
		5	51	53
		6	34	35
		7	17	18
High-Rise	*R00*	*1*	18	29
		2	40	44
		3	51	58
		4	50	57
		5	45	52
		6	46	51
		7	39	43
		8	43	44
		9	28	28
		10	9	10

According to FEMA 356 which refers to UBC 1997 the drift ratio of each story level is not allowed to be more than 2% of the story height. The elastic displacements of existing buildings in the table above are higher than the allowed value and do not fulfill FEMA 356 conditions.

The table of mass participation ratio has been prepared for all buildings in order for seeing each buildings natural frequency and checking whether it is short period or long period. There is the risk of locating at the peak of spectrum which is called as plateau. So the highest acceleration could be applied to the structure at the case of plateau which is not desirable for designers. However computing the periods of the building is also required for calculating reduction factors in following tables.

The table shows that from Low to High rise buildings the first natural period of the buildings are augmenting toward the end of curve in spectrum. The first and at the same time the highest period is created in Y-direction so as expected the base shear at this direction is higher in comparable to X direction.

Table 5.4. Mass participation ratios of existing buildings

		Mode	Period (s)	Participation Mass Ratio			Sum of Participation Mass Ratio		
				UX	UY	UZ	SumUX	SumUY	SumUZ
Low	R00	1	0,96	0,00	0,84	0,00	0,00	0,84	0,00
		2	0,89	0,85	0,00	0,00	0,85	0,84	0,00
		3	0,86	0,00	0,00	0,00	0,85	0,84	0,00
Mid	R00	1	1,43	0,00	0,75	0,00	0,00	0,75	0,00
		2	1,30	0,75	0,00	0,00	0,75	0,75	0,00
		3	1,26	0,00	0,00	0,00	0,75	0,76	0,00
High	R00	1	1,97	0,00	0,76	0,00	0,00	0,76	0,00
		2	1,75	0,76	0,00	0,00	0,76	0,76	0,00
		3	1,69	0,00	0,00	0,00	0,76	0,76	0,00

After taking story forces and periods of the buildings from SAP 2000, some modifications should be done on the applied loads to prepare the model for the assessment analysis. One of the modifications as it could be understood from its name is C modification factors to be considered at load combinations. Under these combinations the frame outputs (Axial Load, Shear, and Moment) of all columns are carried out into m-factor program. As a matter of fact, established combinations are required combinations for force and displacement control as necessary for linear dynamic analysis.

The factors obtained from 1/ ($C_1C_2C_3J$) are included into force control combination considered for each direction separately. No reduction is considered for displacement control due to the fact that structure should be analyzed elastically under displacement control processes. For existing mid-rise building force controlled reduction factor has been calculated as 0.5 and included to FC combinations in the assessment model.

Table 5.5. Modification factors for low rise building

											X-dir.	Y-dir.
Storey	Θ_{ix}	Θ_{iy}	C_{1x}	C_{1y}	C_2	C_{3x}	C_{3y}	C_m	ΠC_x	ΠC_y	$1/(C_1*C_2*C_3*J)$	$1/(C_1*C_2*C_3*J)$
1	0.023	0.026	1	1	1	1	1	0,9	0,9	0,9	0,50	0,50
2	0.024	0.028	1	1	1	1	1	0,9	0,9	0,9	0,50	0,50
3	0.016	0.019	1	1	1	1	1	0,9	0,9	0,9	0,50	0,50
4	0.008	0.009	1	1	1	1	1	0,9	0,9	0,9	0,50	0,50

The reduction factor for mid-rise building has been calculated as table below. Different factors have been calculated for each story separately opposite to low-rise building. For each story a single model can not be evaluated and it is time taking procedure. At upper stories the reduction factor decreases into 0.17 and it shows that the capability of resisting is going down as story numbers go up. For overcoming the matter of many models and single models for each story the highest obtained reduction factor has been taken into account for mid-rise building which is 0.5 reduction factor.

Table 5.6. Modification factors for mid rise building

											X-dir.	Y-dir.
Storey	Θ_{ix}	Θ_{iy}	C_{1x}	C_{1y}	C_2	C_{3x}	C_{3y}	C_m	ΠC_x	ΠC_y	$1/(C_1*C_2*C_3*J)$	$1/(C_1*C_2*C_3*J)$
1	0,020	0,023	1	1	1	1	1	0,9	0,9	0,9	0,50	0,50
2	0,061	0,098	1	1	1	1	1	0,9	0,9	0,9	0,50	0,50
3	0,055	0,083	1	1	1	1	1	0,9	0,9	0,9	0,50	0,50
4	0,048	0,074	1	1	1	1	1	0,9	0,9	0,9	0,50	0,50
5	0,039	0,055	1	1	1	1	1	0,9	0,9	0,9	0,50	0,50
6	0,020	0,025	1	1	2	1	1	0,9	1,8	1,8	0,25	0,25
7	0,025	0,031	1	1	3	1	1	0,9	2,7	2,7	0,17	0,17

The reduction factors at upper stories at high rise building are obeying almost the same rule as mid-rise. A preliminary consequence could be carried out relevant into the number of stories and the amount of gravity, at upper stories the reduction factors decrease however the highest obtained reduction factor is recommended to use at model which at the most cases is the reduction factor at base story.

At high-rise building the reduction factor of force controlled combinations is also 0.5 since the maximum reduction factor is this value as shown in the table below.

Table 5.7. Modification factors for high rise building

											X-dir.	Y-dir.
Storey	Θ_{ix}	Θ_{iy}	C_{1x}	C_{1y}	C_2	C_{3x}	C_{3y}	C_m	ΠC_x	ΠC_y	$1/(C_1*C_2*C_3*J)$	$1/(C_1*C_2*C_3*J)$
1	0,009	0,010	1	1	1	1	1	0,8	0,8	0,8	0,50	0,50
2	0,012	0,013	1	1	1	1	1	0,8	0,8	0,8	0,50	0,50
3	0,012	0,013	1	1	1	1	1	0,8	0,8	0,8	0,50	0,50
4	0,011	0,012	1	1	1	1	1	0,8	0,8	0,8	0,50	0,50
5	0,010	0,011	1	1	1	1	1	0,8	0,8	0,8	0,50	0,50
6	0,009	0,010	1	1	2	1	1	0,8	1,6	1,6	0,25	0,25
7	0,007	0,008	1	1	3	1	1	0,8	2,4	2,4	0,17	0,17
8	0,006	0,006	1	1	4	1	1	0,8	3,2	3,2	0,13	0,13
9	0,005	0,005	1	1	5	1	1	0,8	4	4	0,10	0,10
10	0,002	0,003	1	1	6	1	1	0,8	4,8	4,8	0,08	0,08

5.2.10. Linear Dynamic Analysis

Due to the irregular structural configuration of the buildings in the inventory, all of the structures are analyzed using Linear Dynamic Procedure (LDP) of FEMA 356. Results of all of the analyses are checked for participating mass and stability coefficient, and base shear modifiers are changed accordingly. Accidental torsion effects are considered by adding static equivalent torsional moments where applicable. Static P-Δ effects are accounted for by using iterative analyses including the effects of the axial compressive loads on the stiffness of the structure.

5.2.11. Stiffness Modeling

The stiffness values of the reinforced concrete structural elements are modified based on the reduction of the section stiffnesses to account for the non-linear behavior. No similar modification was applied to steel members. FEMA 356 requirements for RC elements are:

- Modulus of elasticity for the concrete is taken from ACI 318-02,
- Element stiffnesses are modified using the Table 6.5 of FEMA 356.

5.2.12. Multi Directional Seismic Effects

For all framed structures, multi-directional excitation effects are considered using 30% of the main direction spectra and applying it across the considered earthquake direction. Spectral response combinations are done using SRSS procedures for individual directional (X, Y, Z) spectra. Vertical acceleration has not been considered.

5.2.13. Load Combinations

Gravity load case is established by combining the individual contributions of dead and live loads, where the live loads are reduced by a multiplier of 0.25. Following combinations were used for gravity,

$$Q_G = 1.1 \times (Q_D + 0.25 Q_L) \tag{5.14}$$

$$Q_G = 0.9 \times Q_D \tag{5.15}$$

And the spectral load input was combined with the gravity loads using the following formulation,

$$Q_{UD} = Q_G \pm Q_E \tag{5.16}$$

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3 J} \tag{5.17}$$

5.2.14. Demand-to-Capacity Ratios

FEMA 356 procedures define the degree and distribution of the ductility demand using demand-to-capacity ratios (DCR's). Since the element level demand and resistance modifiers are not included in their definition, the DCR's are not used to determine the acceptability of component behavior. Demand-to-capacity ratio for a given behavior mode is defined as,

$$DCR = {}^{Q_{UD}}\!\!\diagup\!{}_{Q_{CE}} \tag{5.18}$$

Where

Q_{UD} = Force calculated due to the gravity and earthquake loads,

Q_{CE} = Expected strength of the component or element.

When evaluating the DCR's and the behavior under deformation-controlled actions the expected strength, Q_{CE}, shall be used. Q_{CE} is defined as the statistical mean value of yield strengths, Q_y, for a population of similar components, and includes consideration of strain hardening and plastic section development.

In the following paragraphs, DCR's are used to demonstrate the member performance level and to explore the retrofit requirement for the investigated structures, Tables 5.8. Since no knowledge factor, κ, or m-factor has been employed the final results of the detailed assessment may be different for structures assigned retrofit requirement with marginal occurrence.

Table 5.8. DCR range of existing buildings

Structure			*Number of Columns with*				*% Columns with*				*Total # Existing*
			DCR<1	*1≤DCR≤2*	*2≤DCR≤4*	*DCR>4*	*DCR<1*	*1≤DCR≤2*	*2≤DCR≤4*	*DCR>4*	
Low-Rise	*R00*	*1*	0	0	0	20	0	0	0	100	20
		2	0	0	10	10	0	0	50	50	20
		3	0	0	18	2	0	0	90	10	20
		4	0	0	20	0	0	0	100	0	20
Mid-Rise	*R00*	*1*	0	0	20	0	0	0	100	0	20
		2	0	13	7	0	0	65	35	0	20
		3	0	2	18	0	0	10	90	0	20
		4	0	4	16	0	0	20	80	0	20
		5	0	0	18	2	0	0	90	10	20
		6	0	0	19	1	0	0	95	5	20
		7	0	3	17	0	0	15	85	0	20
High-Rise	*R00*	*1*	2	11	7	0	10	55	35	0	20
		2	2	12	6	0	10	60	30	0	20
		3	0	20	0	0	0	100	0	0	20
		4	1	19	0	0	5	95	0	0	20
		5	4	16	0	0	20	80	0	0	20
		6	4	16	0	0	20	80	0	0	20
		7	0	10	10	0	0	50	50	0	20
		8	0	12	8	0	0	60	40	0	20
		9	0	20	0	0	0	100	0	0	20
		10	0	3	3	0	0	15	15	0	20

5.2.15. Member Level Acceptance Criteria

Capacity checks of the primary and secondary elements are done in accordance with the rules of FEMA 356 section 6 and 7. Actions listed below are classified as displacement controlled (DC) and the allowed m-factors are used to reduce the element level demand. These m-factors change per element and per loading level, and are controlled for,

Moment acting on columns by,

- the level of gravity shear stress,
- axial load acting on the column,
- shear capacity requirements to develop plastic moments at column end zones,
- splice lengths,
- Detailing of the column transverse reinforcement.

Moment and shear acting on shear walls by,

- the level of shear stress resisted by concrete,
- axial load acting on the wall,
- Detailing of the wall transverse reinforcement and formation of confined end zones.

Moment acting on beams by,

- the level of shear stress resisted by concrete,
- amount of reinforcement as compared to the balanced case,
- shear capacity requirements to develop plastic moments at beam end zones,
- Detailing and spacing of the beam transverse reinforcement.

Moment and shear acting on slab-column joints by,

- the level of shear stress resisted by concrete,
- detailing of the continuity reinforcement,
- Embedment into the slab-column joint.

Actions on structural elements other than these are considered as force controlled and no m-factor is used. However, for the performance acceptance of foundations and columns, the earthquake actions are reduced using the J-factors in the corresponding load combinations. The acceptance limits can be summarized as in the following formulation,

Deformation Controlled Case : $m \kappa Q_{CE} \geq Q_{UD}$,
Force Controlled Case : $\kappa Q_{CL} \geq Q_{UF}$,
with force and resistance of the components defined as in the following.

When evaluating the acceptance of the behavior under deformation-controlled actions the expected strength, Q_{CE}, is used, where Q_{CE} is defined as the nominal component capacity calculated with expected material properties. When evaluating the behavior of force-controlled actions, a lower bound estimate of the component strength, Q_{CL}, is used, where Q_{CL} is defined as the nominal component capacity calculated with statistical mean minus one standard deviation of the material strengths. For the simplified control of low demand cases and elements a ratio of Q_{CE} / Q_{CL} =1.25 - 1.4 was assumed depending on axial load level throughout the analyses.

Based on the methodology outlined above, following sections further discuss the structural analysis and retrofit design at the element level. In the following, the failure modes for columns are identified as, compressive/tensile failure, demand shear failure and flexural failure. If the columns are found inadequate to develop plastic hinges at the ends due to insufficient transverse reinforcement, but are safe under the shear demand these failures are flagged as flexural failures. Where more than one failure is flagged as the result of the acceptance criteria, priority is given to compressive/tensile failure over others, and demand shear failure over the flexural.

Table 5.9. Acceptance criteria values of exiting buildings

Structure			*Failure Mode*			*Failed Columns*	*Total Columns*	*% Failed*
			Axial	*Shear*	*Flexure*			
Low-Rise	*R00*	*1*	0	20	0	20	20	100
		2	0	16	4	20	20	100
		3	0	10	10	20	20	100
		4	0	0	20	20	20	100
Mid-Rise	*R00*	*1*	0	0	20	20	20	100
		2	0	0	19	19	20	95
		3	0	2	17	19	20	95
		4	0	1	15	16	20	80
		5	0	13	7	20	20	100
		6	0	4	16	20	20	100
		7	0	0	19	19	20	95
High-Rise	*R00*	*1*	0	0	17	17	20	85
		2	0	0	17	17	20	85
		3	0	2	14	16	20	80
		4	0	1	12	13	20	65
		5	0	0	11	11	20	55
		6	0	13	0	13	20	65
		7	0	0	13	13	20	65
		8	0	0	8	8	20	40
		9	0	0	0	0	20	0
		10	0	0	0	0	20	0

5.2.16. Risk levels of phase 1 and Assessment process of phase 2

The objective of the Phase 1 study is to,

- Identify the basic characteristics of buildings,
- Designate the buildings vulnerable under seismic actions,
- Estimate the probable damage and loss,

The Phase 1 inspections were executed without making detailed use of design drawings and calculations and any geotechnical data. According to this assessment, the Capacity/Demand ratios of all buildings on site were defined and the following buildings were classified at risk levels.

Assessment procedures, in general, require the check of the crucial detailing, strength and stability requirements of a structural configuration, regardless of their age, construction type, original design codes, etc., but considering structural condition, material quality, configurations regularity and redundancy explicitly. In what follows, the “FEMA

356 – Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency (2000)” is taken as a frame of reference for structural calculations of the assessment works. Parts of the document are summarized below in order to highlight the important points on the adopted methodology.

6. RETROFITING THE BUILDING BY ECCENTRICALLY BRACED FRAMES

There are different strategies of retrofitting a reinforced concrete building. Usually after detailed assessment of a building, different methods are tried to strengthen the structure. Most of the designers take architectural and structural issues into account before making decision upon the way of retrofitting. Existing reinforced concrete (RC) frame buildings with nonductile detailing represent a considerable hazard during earthquakes. The nonductile behavior of RC frames is due to inadequate transverse reinforcement in beams, columns and joints; bond slip of beam bottom reinforcement at the joint; and lack of confinement of the column lap splice area. It is important to develop effective and economic seismic rehabilitation systems for nonductile RC buildings before an earthquake occurs, since the potential for damage and loss of life during future seismic events is unacceptably high.

Different rehabilitation systems have been developed to upgrade the seismic performance of existing undamaged structures before being subjected to an earthquake. The two main approaches are:

- Add new structural elements such as structural walls or steel bracing.
- The selective strengthening of deficient structural elements such as the use of concrete and steel jackets and fiber reinforced polymers, FRP wrapping.

The use of steel bracing systems for seismic rehabilitation of RC frames offers some advantages such as:

- The ability to accommodate openings
- Minimal added weight to the structure
- External steel systems have been constructed with minimum disruption to the function of the building and its occupants

The use of eccentric steel bracing in the rehabilitation of RC structures has lagged behind concentric steel bracing applications due to the lack of sufficient research and information about the design, modeling and behavior of the combined concrete and steel system. To facilitate the application of eccentric bracing in rehabilitation, further research is needed in several areas such as testing of the RC beam–steel link connection details and design as well as the development and implementation of link element models in analysis software. In eccentrically braced frames (EBFs), forces are transferred to the brace members through bending and shear forces developed in the ductile steel link. The link is designed to act as a fuse by yielding and dissipating energy while preventing buckling of the brace members. Well-designed links provide a stable source of energy dissipation.

Different brace patterns are used in eccentrically braced steel frames. Examples of these patterns include V-bracing, K-bracing, X-bracing and Y-bracing as shown in Figure 6.1.

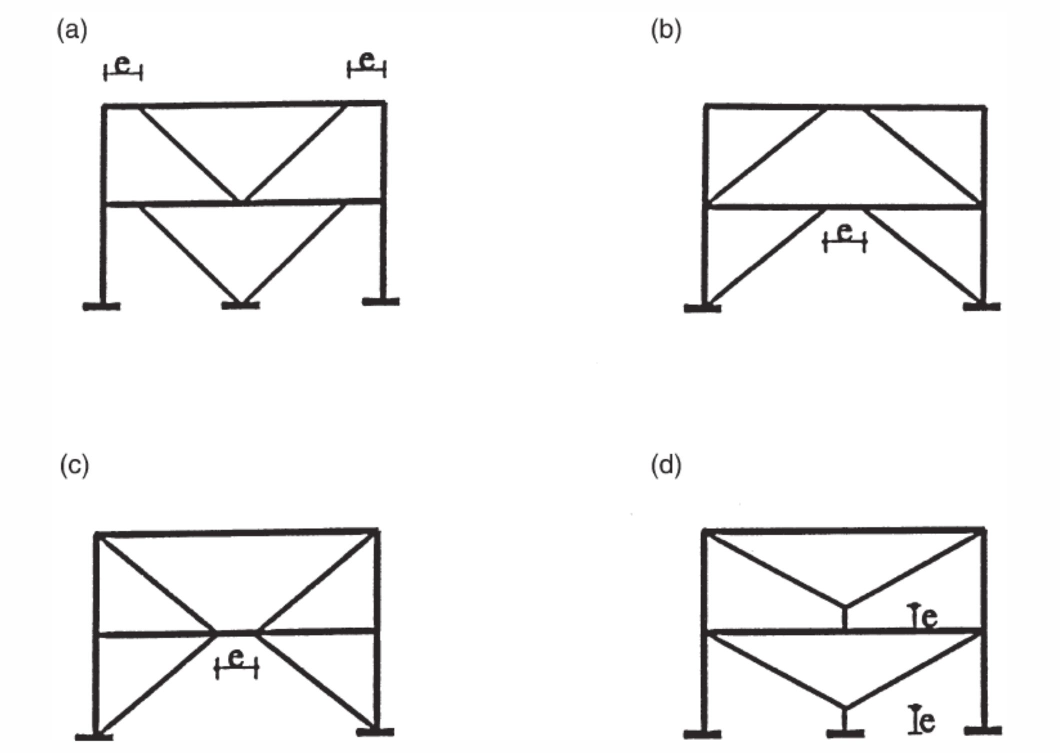


Figure 6.1. Different types of EBF systems a) V-type with two links b) inverted V-type with single link c) X-type d) Y-type with vertical link

Most of these patterns utilize short beam segments as active links. The analysis, design and performance of eccentrically braced steel frames have been studied by several researchers (Ghobarah *et al.*, 2003). In RC frames, the concrete beams are incapable of performing as a ductile link for the steel bracing system that is inserted in the frame bays.

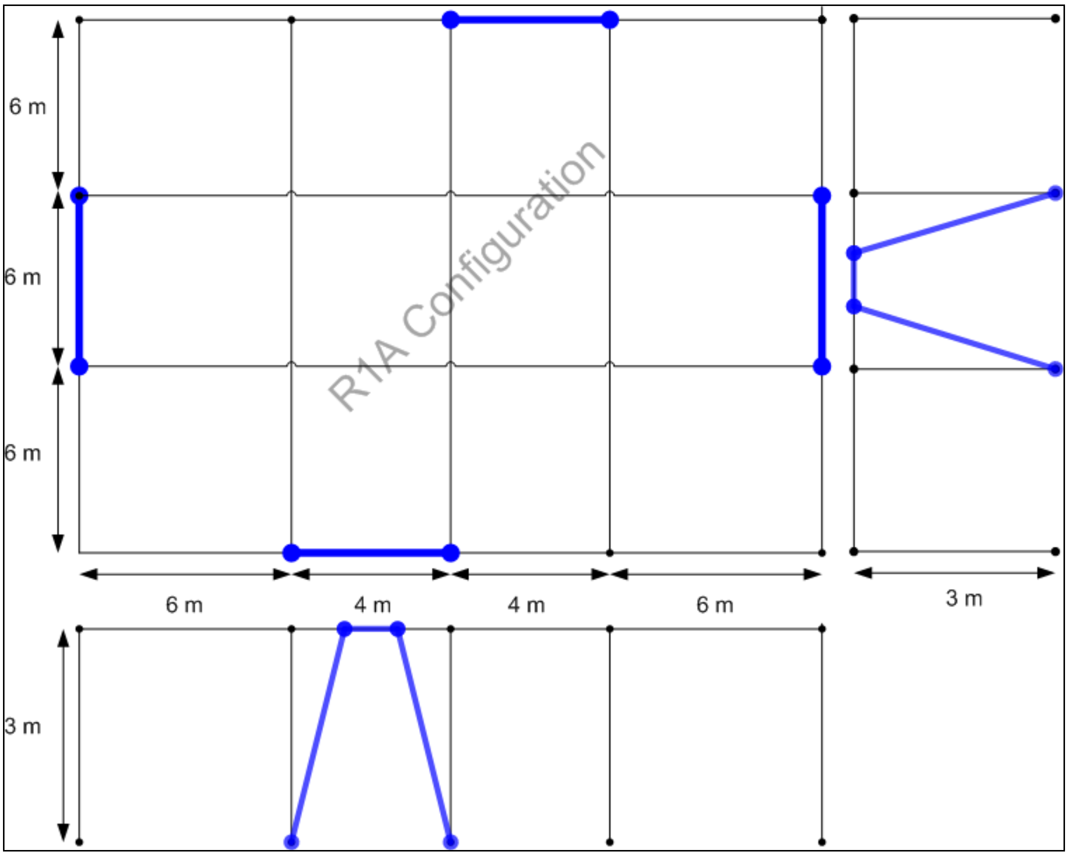


Figure 6.2. Plane and section views of R1A configuration

6.1. Retrofit strategy - R1A

From the assessment of R00 structures, it was observed that existing buildings do not satisfy desirable performance (Life Safety) according to FEMA 356 because there are shear, flexure and even axial failures at columns. These failures would cause collapse of the structure during earthquake with 10% of exceedance in 50 years.

For starting point, EBF systems were located at end bays in which at each end bays there is only one EBF along the building. EBF systems were located at middle span at y-direction symmetrically and in x-direction asymmetrically. The reason of such configuration was that, it was only trial 1 of a preliminary design phase, so the only mission which that configuration was after, was to proving that EBF systems would have contribution for the building against lateral forces. R1A configuration can be seen from figure.

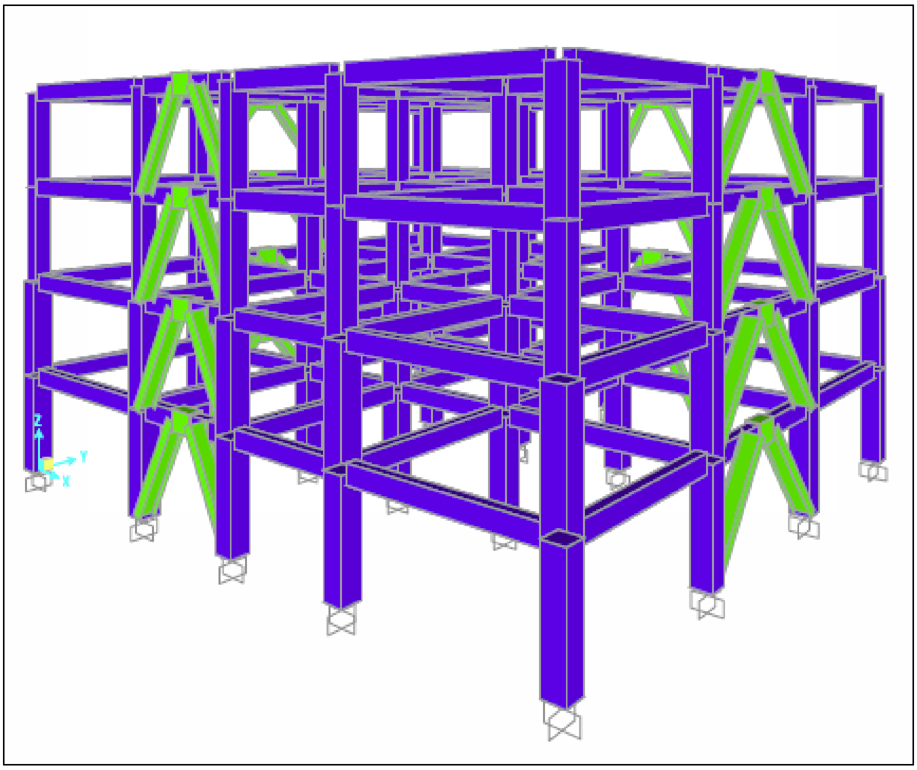


Figure 6.3. 3D view of R1A configuration low-rise building

Since reinforced concrete beam at existing structure-R00 is not supposed to have enough ductility and strength for behaving like as link beam, it was decided to locate steel plates at this part of the beam, as if a box is located at link. However, the behavior of steel box should have been examined before that. For seeing the behavior of steel plates covered RC beam, an external model composed of two story and one bay frame was developed out of the R1A models. At mid-rise model R1A, frame in direction X was departed and instead of steel box, steel plates were modeled by shell area sections. This external model's mission was only to see the distribution of shear caused by earthquake on the plates.

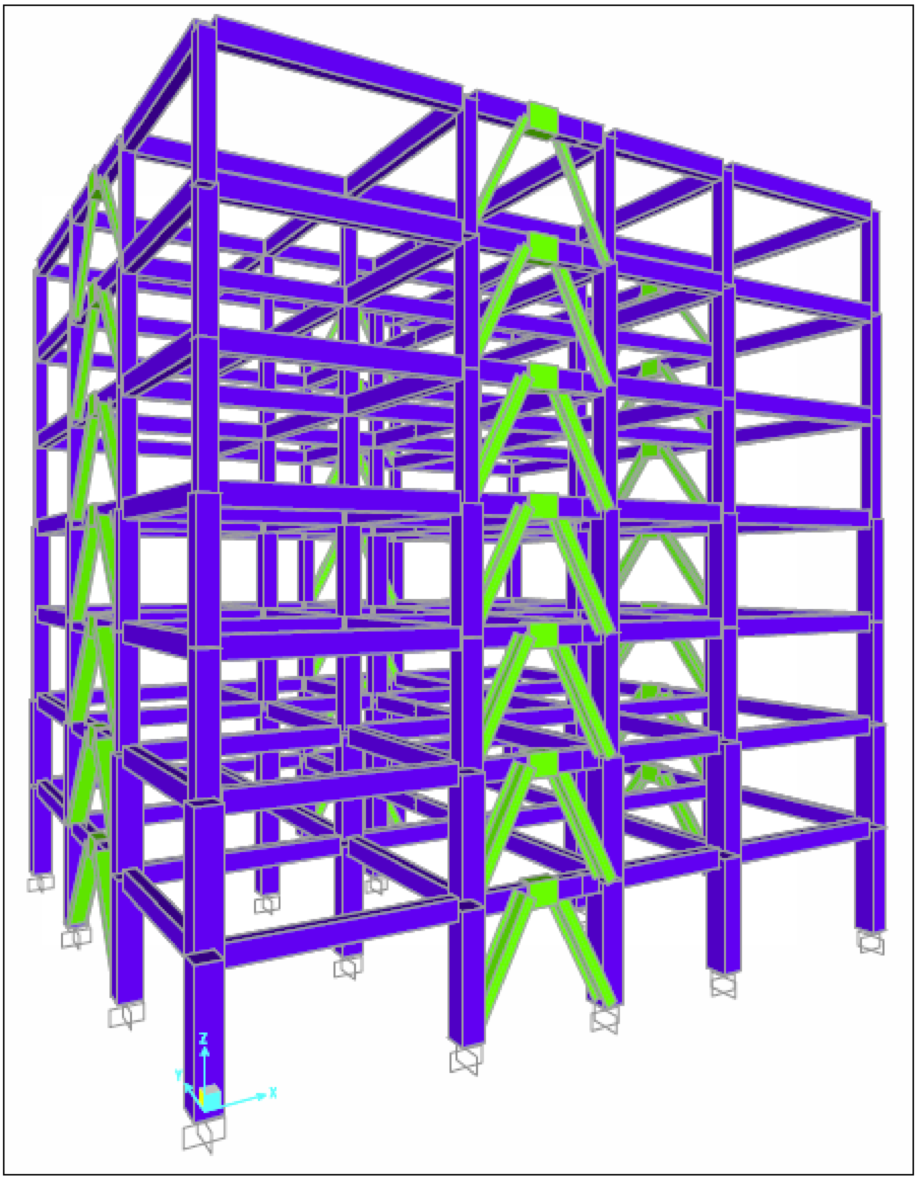


Figure 6.4. 3D view of R1A configuration mid-rise building

The connected braces to link elements are performing like strong axis as out-of-plane and do not carry any moment into the link. So connection between brace and link should be as if the effective length of the braces do not change as they have been modeled as k=1. So at external model the connection between steel plates, beam and braces are fully rigid. As it stated before, the worst condition was selected for seeing the behavior of steel plates as link, therefore, the base and first story of mid-rise R1A model were chosen. Most of the designers locate an I-section profile as link element and design the system like that braces and all elements except link remain elastic but the link behave plastically. The most desirable plastic behavior is plastic shear. So for this external model the stresses coming up from shear were examined. The stress analysis results for S12 as absolutely maximum, for the lateral force in X-direction is as follow:

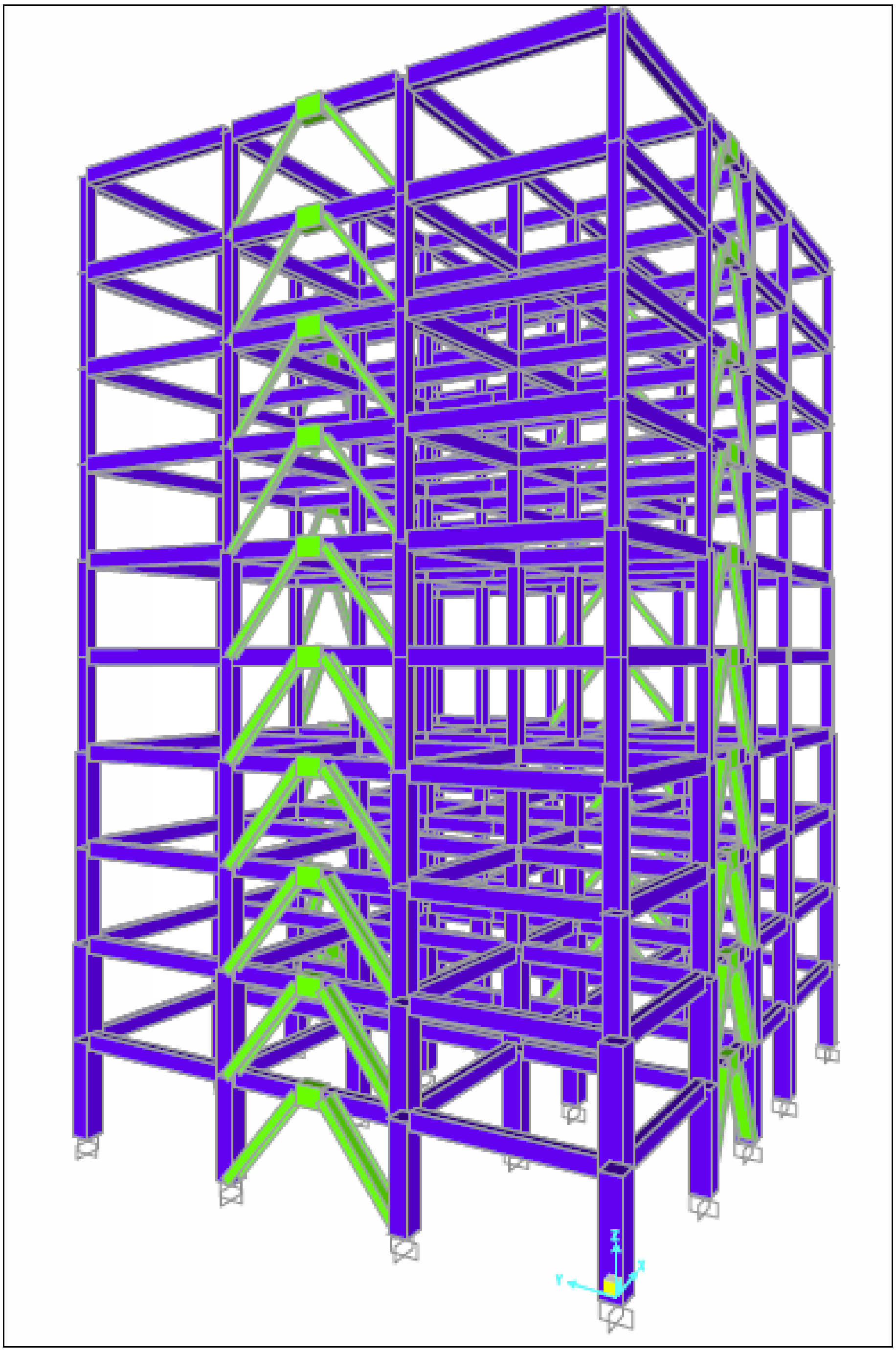


Figure 6.5. 3D view of R1A configuration high-rise building

6.2. The Stress Analysis of Steel Plates Covered RC Beam

6.2.1. EBF in Steel Structures

A typical configuration of EBF in steel structures has been shown in figure below. It would be of benefit to investigate the behavior of EBF in steel structures and after that implementing it in RC structures. In EBF systems, braces are responsible for resisting the lateral forces and transferring it into link elements. The link element, by absorbing energy acts like a fuse against lateral loads for the system. Thus plastic behavior is expected in link elements; desirable plastic action is the plastic behavior due to shear which mostly happens in short link components. The design philosophy is designing the link as inelastic and keeping the rest of the framing system behaves elastically. The link component should have enough ductility to act plastically so the stiffeners are compulsory elements of design and required at codes. Some highlight factors of EBF systems are:

- Eccentric bracing is commonly used in seismic regions and allows for doorways and corridors in the braced bays
- The difference between Chevron bracing and eccentric bracing is the space between the bracing members at the top gusset connection
- In an eccentrically braced frame bracing members are connected to separate points on the beam/girder
- The beam/girder segment or “link” between the bracing members absorbs energy from seismic activity through plastic deformation

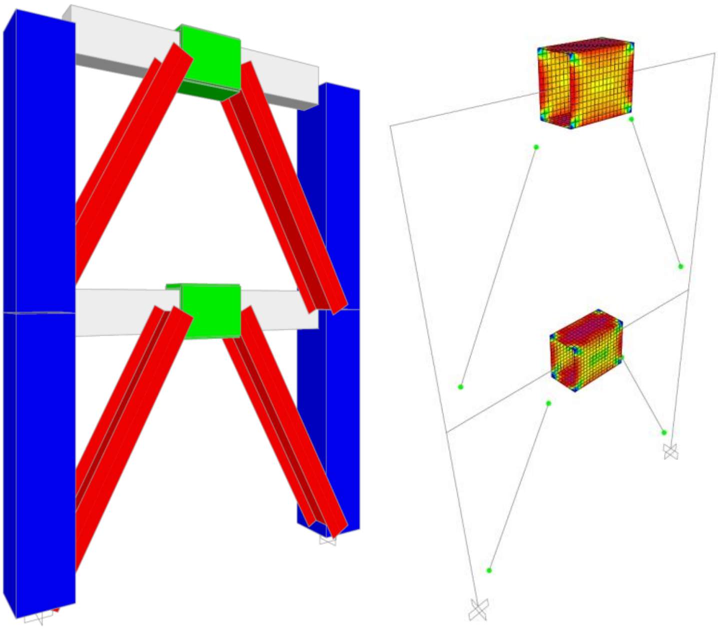


Figure 6.6. 3D view of steel plates stress analysis

6.2.2. EBF in RC Structures

Designing EBF systems at steel structures is almost difficult in application and complicated, however the result is so effective. For this study in order to obtain the same result as much as steel structures, the modeling and detailing concept are so crucial. The key point of designing EBF is to provide enough ductility at the link element which would behave inelastic after absorbing the energy. In previous section it has been shown that the shear forces of the link are higher than other elements. Usually in steel structures the section profile of the link element is I-profile (wide flange I section) and the web of I-profile is responsible for overcoming shear. The reason that stiffeners are required in the web is to providing sufficient stiffness at the web against shear. However, in this study there is no accessibility to steel wide flange in the middle of existing RC beam. Moreover it is obvious that the existing beam does not have mentioned ductility. On the other hand, one of the scopes outlined at this study was to locating link element in the framing system, not vertically nor exteriorly. Two options were considered during the modeling, the first was to crush the RC beam and construct a new RC beam which has sufficient longitudinal and transverse reinforced rebar to ensure the required stiffness and ductility and using the RC beam as link component, and the second was similar to the first choice except link element. At the second option, after constructing new RC beam, around the link segment could be covered by steel plates with length e (length of link). In this study the first option has not been applied. The second option has been decided for proceeding and models have been evaluated according to this option. The first option which seems more economically and risky is recommended for future study.

Selecting the second option needs to be approved. In particular, it is required to be proved that this system will work and will act as a fuse which carries the story shear. A prototype model was created in SAP2000 and 2D frames from mid-rise and high-rise models were isolated. Only base and fist story of 2D frame were taken into account as shown in figure above the aim of this prototype model was to observe the distribution of coming shear in the steel plates and defining preliminary required thickness of the plates to be designed. As it can be seen from the stress outputs of the model, steel plate are accepting the shear and react against the story shear.

Plates have been modeled by steel shell area surrounded RC beam and rigidly connected to the beam. Plates are expected to be welded in construction site. The key to accomplishing such model is detailing. So, all processes like connection to floor diaphragm, welding the plates to each other, anchoring into RC beam and even connection between brace and bottom steel plate are of considerable importance. Otherwise the objectives of the study would never be carried out.

It can be seen from figure below hat S12, shell stresses under Sx combination (factored spectral dynamic force) has been contoured between [0 4] range. For seeing the maximum shear appeared on the plates absolute maximum stress outputs have been shown in the figure. The unit of presenting the figures is MPa (N/mm^2)

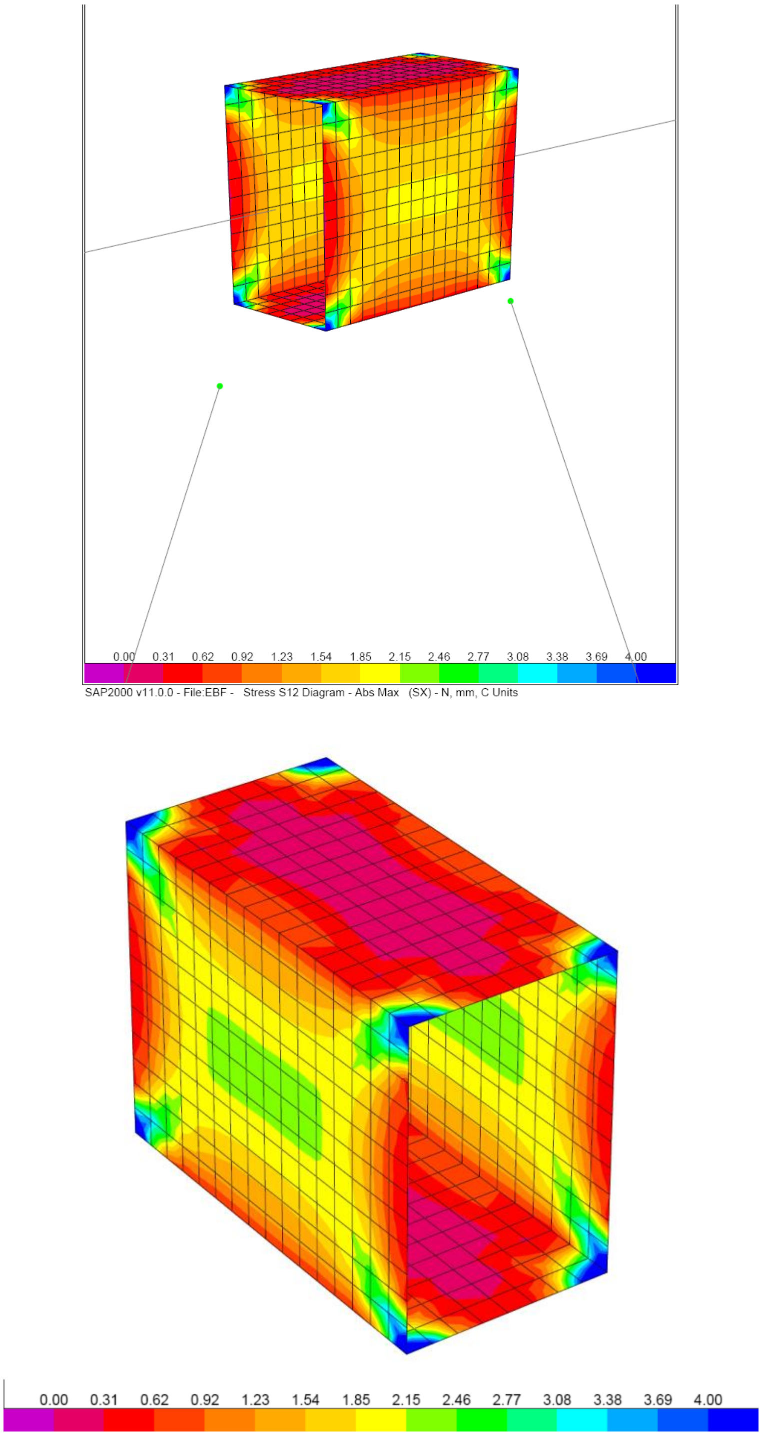


Figure 6.7. Shear stress (S12) analysis of steel plates

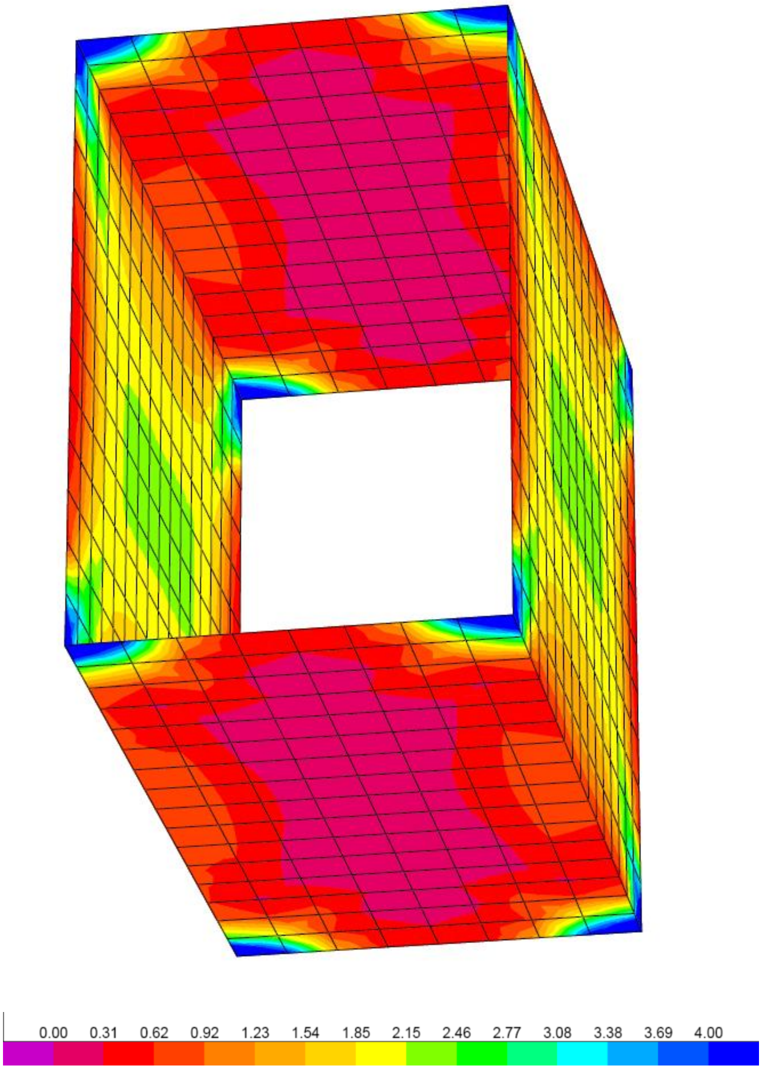


Figure 6.8. The range of shear stress on steel plates

6.2.3. Results from Stress Analysis

From stress analysis it was obtained that plates should be welded so carefully to each other, because shear stresses appeared in the edge of the plates is about 4 MPa. So before determining the thickness and area of weld it would be ensured that 4 MPa value has been taken into account.

As it can be seen from the figures, the top and bottom plates do not take so much shear pressure and it is mostly the vertical plates carrying the shear pressure. Similar to I-profile the vertical steel plates are acting as the web of box section. The maximum appeared stress on the vertical plates is 3 MPa. During plastic design of web area this value should be considered.

A crucial remark should be stated here before detailing the connection between steel plates and braces, as it is known the braces are located strong-axis as out-of-plane, so the flange of braces must be in-plan with vertical plates as much as possible. Since if the flange locate at middle of bottom plate they would causes unbalanced flexural during being buckled, therefore local steel plate buckling would be inevitable at such condition. When the flange is in the axis of vertical plates the brace would transfer the force directly to the web of box section which is desirable matter of design. For accomplishing in-plane brace flange and vertical plates, before constructing the new RC beam, the geometry of braces should be determined and the about 2 cm less than depth of connected brace could be taken as the width of new RC beam.

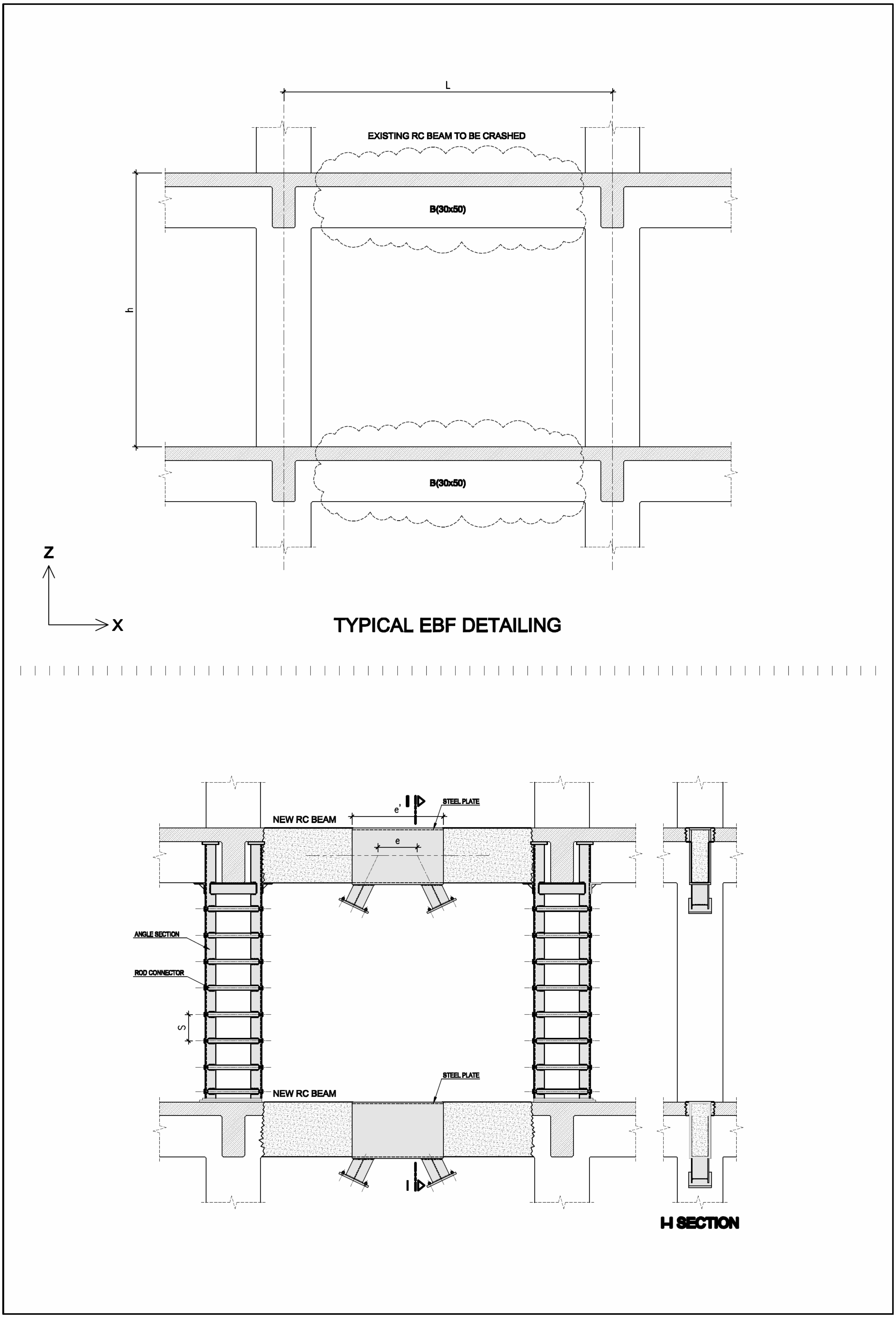


Figure 6.9. Typical EBF detailing

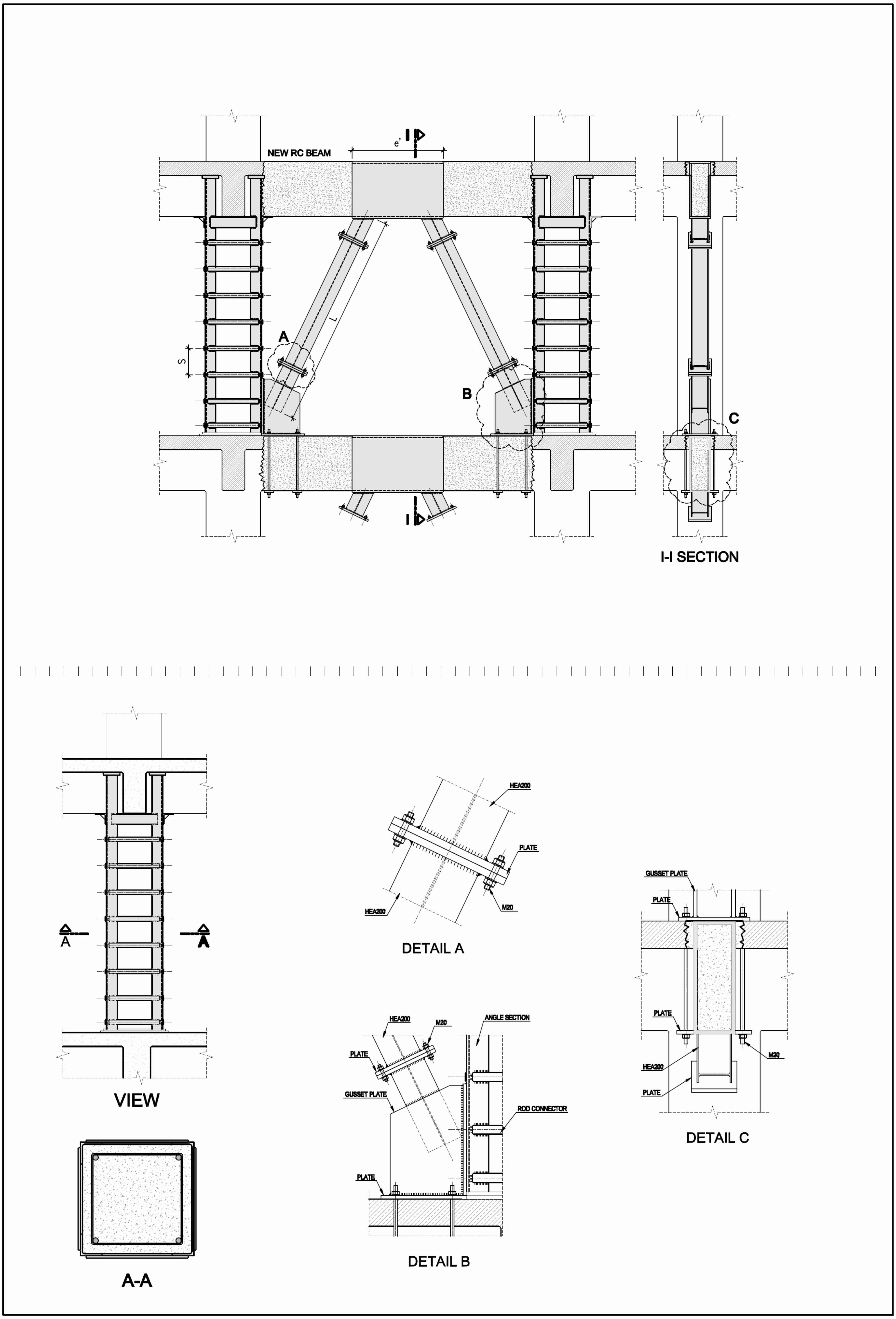


Figure 6.10. Typical column jacketing

7. COMPARISON THE LINEAR DYNAMIC ANALYSIS RESULTS

7.1. R00 and R1A structures

At this part of the study, the results of linear dynamic analysis would be presented in comparable form. In other words, by considering the main objective which is to compare the contribution of EBF system, it would be effective when the results tables are presenting next to each other. R00 which represents the results of the existing buildings would be differentiated with R1A which represents the results of first selected EBF configuration.

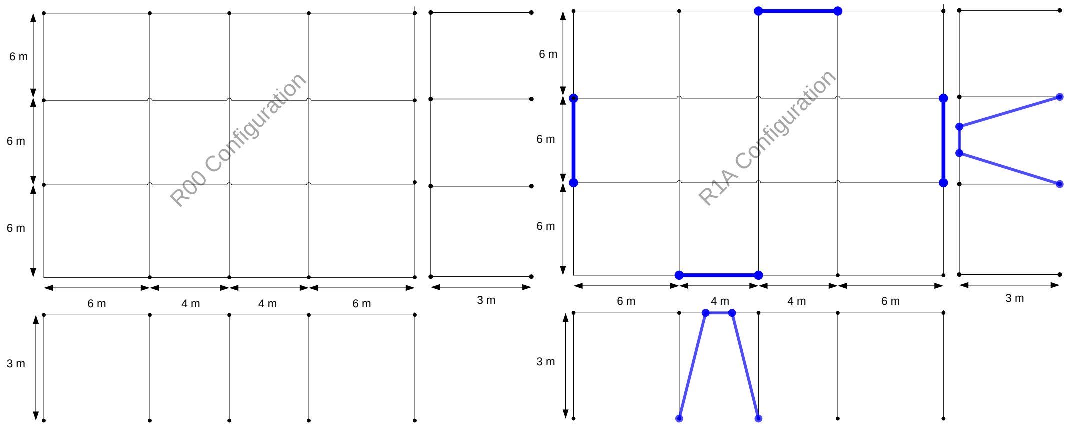


Figure 7.1. R00 and R1A on-plane and sections

The first brace configuration was designed to be located at end bays of the buildings. As a matter of fact, located braces have been designed according to UBC 1997 - load resistance factor design the main criteria for designing the braces was to keep the demand capacity ratio of these braces between the range of 0.7 and 0.9. The reason of such DCR arrangement was to ensuring that braces would yield before connected framing elements. In other words, the most desirable sequence of yielding components is link element, braces, beams and at last columns. However this sequence is only an initial priority for designers and the design is never done on sequential process. Finally, wide steel profiles with yield strength of 460 MPa were selected to be located as their strong-axis is out-of-plane of frame. Lateral buckling would be resisted by locating the braces out-of-plane. So, various types of H-profiles were designed in the model as their DCR remain between 0.7 and 0.9.

As same as assessment stage of the buildings, the first table carried out of retrofitted models is the table of story forces. These tables provide a preliminary idea of the behavior of the buildings and make it possible to state the first opinion of retrofit strategy which has been effective or not.

Table 7.1. Story forces of R00 and R1A

Structure		*Stroy No*	*Story Height (m)*	*Area (m2)*	*Dead Load (kN)*	*Live Load (kN)*	*Stroy Shear Forces in X Direction (kN)*	*Stroy Shear Forces in Y Direction (kN)*
Low-Rise	R00	*1st*	3	360	2507	1436	2179	1657
		2nd	3	360	2457	1411	1462	819
		3rd	3	360	2383	1389	1373	822
		4th	3	360	1644	214	760	455
		Total	12	1440	8992	4451		
	R1A	*1st*	3	360	2470	1441	6147	5695
		2nd	3	360	2461	1436	5345	4956
		3rd	3	360	2448	1437	3853	3606
		4th	3	360	1692	226	1780	1698
		Total	12	1440	9071	4540		
Mid-Rise	R00	*1st*	3	360	2788	1430	5078	4632
		2nd	3	360	2770	1426	2942	1720
		3rd	3	360	2588	1415	2553	1502
		4th	3	360	2584	1415	2920	1862
		5th	3	360	2438	1409	2051	1284
		6th	3	360	2372	1379	1965	1333
		7th	3	360	1633	209	1110	799
		Total	21	2520	17173	8684		
	R1A	*1st*	3	360	2745	1436	7542	6926
		2nd	3	360	2739	1433	7145	6538
		3rd	3	360	2577	1430	6426	5851
		4th	3	360	2574	1429	5556	5064
		5th	3	360	2446	1429	4639	4267
		6th	3	360	2443	1429	3419	3199
		7th	3	360	1679	219	1670	1600
		Total	21	2520	17203	8805		
High-Rise	R00	*1st*	3	360	2788	1430	4547	4011
		2nd	3	360	2774	1428	2736	1608
		3rd	3	360	2579	1410	2481	1464
		4th	3	360	2595	1416	3113	2066
		5th	3	360	2438	1400	2410	1583
		6th	3	360	2412	1382	2670	1879
		7th	3	360	1686	230	2008	1385
		8th	3	360	2395	1395	2662	1966
		9th	3	360	2330	1359	2289	1726
		10th	3	360	1600	194	1462	1238
		Total	30	3600	23597	11645		
	R1A	*1st*	3	360	2927	1435	8985	8051
		2nd	3	360	2918	1430	8660	7737
		3rd	3	360	2911	1427	8054	7178
		4th	3	360	2721	1424	7354	6550
		5th	3	360	2713	1420	6675	5962
		6th	3	360	2709	1419	6047	5411
		7th	3	360	2552	1418	5419	4857
		8th	3	360	2547	1416	4584	4144
		9th	3	360	2542	1415	3345	3069
		10th	3	360	1828	207	1550	1454
		Total	30	3600	26367	13012		

In the table above R00 is the result of existing buildings and R1A is the results of the first selected brace configuration as retrofit strategy. At low-rise building the base shear of building before retrofitting was 2179 KN in X-direction and after rehabilitation it has raised into 6147 KN, approximated 3 times the former. At mid rise and high rise buildings this ratio is about 1.5 and 2 respectively. So the first impression is that, EBF system has had some contribution for the building and has been theoretically approved by taken into account story forces.

The second scale of comparison is story displacements, the floor diaphragm rigidity and story stiffness could be outcome by story displacements and drifts. As in many codes tone of the criteria for determining the structural performance level is story drifts and a restriction which has been experimentally set is considered as boundaries of displacements.

For measuring the story displacements, due to floor diaphragm rigidity the center of this diaphragm has been taken for. In the low-rise building the story displacements has fallen overall through the height, for instance at second story the displacement before retrofitting was 67 mm and decreased into 17 mm. an average ratio of 3 has been saved as story displacement. This ratio at mid-rise and high-rise buildings has been computed as 2 and 1.5 respectively. Observing this ratios helps to have another scale of comment upon the retrofit strategy. So far, speaking only on story forces and story displacements which are outputs of the program has shown that R1A configuration has provided strength and rigidity for the buildings.

The third step of preliminary observing the result of rehabilitation is bringing out the modal analysis results. The first 3 natural dynamic periods of buildings has been presented in the table below. In the low-rise building the first period which is occurring in Y-direction is 0.96 s before retrofitting, and after that, is 0.47 s but in X-direction. It could be understood that the manner of natural stability of own structure could be changed after implementing EBF system into structural system. At mid-rise and high-rise buildings the ratio of 2 and 1.5 is dominated for periods before and after the retrofitting. So reducing the natural periods is a sign of raising the rigidity of the systems.

Table 7.2. Story displacements of R00 and R1A

Structure			*Displacement*	
			D_x *[mm]*	D_y *[mm]*
Low-Rise	*R1A*	*1st*	19	15
		2nd	23	17
		3rd	20	15
		4th	13	10
	R00	*1st*	55	57
		2nd	63	67
		3rd	42	46
		4th	19	20
Mid-Rise	*R1A*	*1st*	17	16
		2nd	26	23
		3rd	29	25
		4th	29	25
		5th	29	24
		6th	24	20
		7th	18	15
	R00	*1st*	27	30
		2nd	45	51
		3rd	53	59
		4th	46	52
		5th	51	53
		6th	34	35
		7th	17	18
High-Rise	*R1A*	*1st*	20	20
		2nd	26	26
		3rd	27	28
		4th	28	28
		5th	27	28
		6th	28	29
		7th	27	28
		8th	24	26
		9th	21	23
		10th	11	14
	R00	*1st*	18	19
		2nd	40	44
		3rd	51	58
		4th	50	57
		5th	45	52
		6th	46	51
		7th	39	43
		8th	43	44
		9th	28	28
		10th	9	10

Table 7.3. Participation mass ratios of R00 and R1A

Structure		Mode	Period (s)	Participation Mass Ratio			Sum of Participation Mass Ratio		
				UX	UY	UZ	SumUX	SumUY	SumUZ
Low-Rise	R1A	1	0,47	0,84	0,00	0,00	0,84	0,00	0,00
		2	0,41	0,00	0,84	0,00	0,84	0,84	0,00
		3	0,31	0,00	0,00	0,00	0,84	0,84	0,00
	R00	1	0,96	0,00	0,84	0,00	0,00	0,84	0,00
		2	0,89	0,85	0,00	0,00	0,85	0,84	0,00
		3	0,86	0,00	0,00	0,00	0,85	0,84	0,00
Mid-Rise	R1A	1	0,81	0,75	0,00	0,00	0,75	0,00	0,00
		2	0,70	0,00	0,76	0,00	0,75	0,76	0,00
		3	0,54	0,00	0,00	0,00	0,75	0,76	0,00
	R00	1	1,43	0,00	0,75	0,00	0,00	0,75	0,00
		2	1,30	0,75	0,00	0,00	0,75	0,75	0,00
		3	1,26	0,00	0,00	0,00	0,75	0,76	0,00
High-Rise	R1A	1	1,27	0,70	0,00	0,00	0,70	0,00	0,00
		2	1,10	0,00	0,71	0,00	0,70	0,71	0,00
		3	0,85	0,00	0,00	0,00	0,70	0,71	0,00
	R00	1	1,97	0,00	0,76	0,00	0,00	0,76	0,00
		2	1,75	0,76	0,00	0,00	0,76	0,76	0,00
		3	1,69	0,00	0,00	0,00	0,76	0,76	0,00

At the fourth step of comparison the outputs of m-factor program has been presented in table below which shows flexural DCR of columns. The framing system of the buildings is composed of beam and column and no shear wall or other lateral resisting element was valid in the assumption of existing buildings. The performance level of the buildings at existing phase should be evaluated based on these framing elements. However at this stage the columns which are the primary component have been considered to reflect the performance level of all structure. Therefore, knowing the range of DCR at all columns is necessary for determining the performance level.

At table below, the columns are presented in four arrangement, the intervals are the DCR less than 1, the DCR between 1 and 2, the DCR between 2 and 4 and the DCR higher than 4. No m-factor or other seismic reduction factor has been included in these values so the nominal values are presented.

The number of columns at all stories is 20 and the failure has been showed as number and also the percentage. When the column does not furnish the requirements of life safety performance level it would be considered as failed.

At low-rise building it can be easily seen that neither R00 nor R1A furnish the life safety performance level because there are failed column at both models. However, as it has been emphasizing is that EBF system at R1A configuration has provided some contribution for the building. Likewise, the columns with DCR higher than 4 are almost disappeared from R1A model as shown in table below, however there are still the columns with DCR higher than 1 and even 2 in the model. It should be highlighted that these DCR values are purely obtained from elastic analysis and o seismic reduction factor is included. So it is early for deciding that R1A and even R00 do not fulfill life safety requirements.

In mid-rise R00, at first story 100% of the columns have DCR higher than 2 and for meeting the life safety level they need a m-factor of 2 and even higher. However at the same story in R1A only 5% of the columns have DCR higher than 2 and the rest are between other ranges, 40% higher than 4 and 55% between 1 and 2. As it could be guest, knowing and estimating the performance level is impossible from these data and m-factored values are required. For each element there could be taken a different m-factor from table of m-factors. The toughness and the volume of calculation which should be done is obvious at this stage and doing all these calculation is time-taking and mistakable. M-factor program developed in visual-basic in the database of Excel is of considerable advantage for doing all calculation automatically.

Therefore, the DCR table has shown below is for only knowing the range of DCR in hand and is a prerequisite for acceptance criteria table. The DCR table helps to estimate the required m-factor from elastic analysis into inelastic analysis.

Table 7.4. Flexural demand-to-capacity ratios for columns of R00 and R1A/ expected capacities, under PMM interaction and prior to brittle failure mode control.

Structure			*Failure Mode*			*Failed Columns*	*Total Columns*	*% Failed*
			Axial	*Shear*	*Flexure*			
Low-Rise	*R1A*	*1*	8	0	0	8	20	40
		2	1	0	0	1	20	5
		3	0	0	0	0	20	0
		4	0	0	0	0	20	0
	R00	*1*	0	20	0	20	20	100
		2	0	16	4	20	20	100
		3	0	10	10	20	20	100
		4	0	0	20	20	20	100
Mid-Rise	*R1A*	*1*	8	0	0	8	20	40
		2	8	0	0	8	20	40
		3	8	0	0	8	20	40
		4	8	0	0	8	20	40
		5	0	0	0	0	20	0
		6	0	0	0	0	20	0
		7	0	0	0	0	20	0
	R00	*1*	0	0	20	20	20	100
		2	0	0	19	19	20	95
		3	0	2	17	19	20	95
		4	0	1	15	16	20	80
		5	0	13	7	20	20	100
		6	0	4	16	20	20	100
		7	0	0	19	19	20	95
High-Rise	*R1A*	*1*	8	0	0	8	20	40
		2	8	0	0	8	20	40
		3	8	0	0	8	20	40
		4	8	0	0	8	20	40
		5	8	0	0	8	20	40
		6	8	1	0	9	20	45
		7	0	0	1	1	20	5
		8	0	0	0	0	20	0
		9	0	0	0	0	20	0
		10	0	0	0	0	20	0
	R00	*1*	0	0	17	17	20	85
		2	0	0	17	17	20	85
		3	0	2	14	16	20	80
		4	0	1	12	13	20	65
		5	0	0	11	11	20	55
		6	0	13	0	13	20	65
		7	0	0	13	13	20	65
		8	0	0	8	8	20	40
		9	0	0	0	0	20	0
		10	0	0	0	0	20	0

Table 7.5. Number of columns failing the acceptance criteria at R00 and R1A / Including m and k factors

Structure			Failure Mode			Failed Columns	Total Columns	% Failed
			Axial	Shear	Flexure			
Low-Rise	R1A	1	8	0	0	8	20	40
		2	1	0	0	1	20	5
		3	0	0	0	0	20	0
		4	0	0	0	0	20	0
	R00	1	0	20	0	20	20	100
		2	0	16	4	20	20	100
		3	0	10	10	20	20	100
		4	0	0	20	20	20	100
Mid-Rise	R1A	1	8	0	0	8	20	40
		2	8	0	0	8	20	40
		3	8	0	0	8	20	40
		4	8	0	0	8	20	40
		5	0	0	0	0	20	0
		6	0	0	0	0	20	0
		7	0	0	0	0	20	0
	R00	1	0	0	20	20	20	100
		2	0	0	19	19	20	95
		3	0	2	17	19	20	95
		4	0	1	15	16	20	80
		5	0	13	7	20	20	100
		6	0	4	16	20	20	100
		7	0	0	19	19	20	95
High-Rise	R1A	1	8	0	0	8	20	40
		2	8	0	0	8	20	40
		3	8	0	0	8	20	40
		4	8	0	0	8	20	40
		5	8	0	0	8	20	40
		6	8	1	0	9	20	45
		7	0	0	1	1	20	5
		8	0	0	0	0	20	0
		9	0	0	0	0	20	0
		10	0	0	0	0	20	0
	R00	1	0	0	17	17	20	85
		2	0	0	17	17	20	85
		3	0	2	14	16	20	80
		4	0	1	12	13	20	65
		5	0	0	11	11	20	55
		6	0	13	0	13	20	65
		7	0	0	13	13	20	65
		8	0	0	8	8	20	40
		9	0	0	0	0	20	0
		10	0	0	0	0	20	0

The actual assessment of R1A and comparing it to R00 is done in the table of acceptance criteria. The values of m-factor and k (knowledge factor) are included at the results of this table. The appropriate m-factor for every element has been set from m-factor table DCR of each element has been divided into obtained m-factor separately. At acceptance criteria, the failure modes have been arranged as axial, shear and flexural, so each element’s cause of failure would be observed.

At low-rise 100% of columns fail at R00 but after retrofitting only 45% at base story, 5% at first story and no failure at upper stories is observed. At mid-rise and high-rise buildings 100% failure is fallen into 40% and 85% failure is fallen into 40% respectively. R00 and R1A do not fulfill life safety performance level; however R1A shows a behavior close to this performance level in comparable to R00 and is not so far from this level.

7.2. R2B and R2C structures

Second type of comparison would be between buildings retrofitted by concentrically braced frames and the buildings retrofitted by eccentrically braced frames. The effect of concentrically braced frames upon the rigidity of the floor diaphragms has been inspected in many studies. This system of bracing could be implemented as X, K, Chevron and inverted brace systems. Concentrically braced frames mechanism of inelastic behavior is focused on braces. Braces will undergo tension and compression stresses and will transfer only axial forces in this way. The only anxiety in concentrically braced frames is buckling of compression brace, as a matter of fact the buckling load control the behavior and design of such braced systems. In particular, chevron and inverted braces design is of higher difficulty in comparable to others, because the vertical unbalanced force produced by difference of compression and tension braces, causes higher section of intersected beam and at most cases are uneconomical.

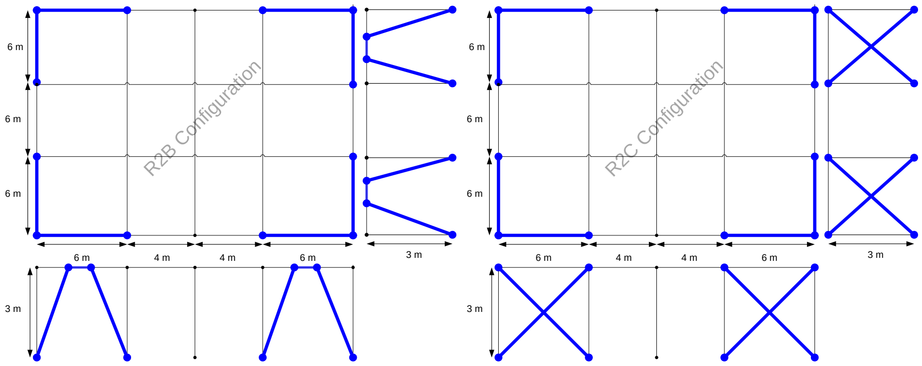


Figure 7.2. Story forces of R2B and R2C

Table 7.6. Story forces of R2B and R2C

Structure		*Stroy No*	*Story Height (m)*	*Area (m2)*	*Dead Load (kN)*	*Live Load (kN)*	*Stroy Shear Forces in X Direction (kN)*	*Stroy Shear Forces in Y Direction (kN)*
Low-Rise	R2B	*1st*	3	360	2480	1003	1441	1324
		2nd	3	360	2452	996	614	432
		3rd	3	360	2185	868	627	498
		4th	3	360	967	193	209	158
		Total	12	1440	8085	3059		
	R2C	*1st*	3	360	2458	989	787	732
		2nd	3	360	2422	979	542	446
		3rd	3	360	2441	991	644	554
		4th	3	360	1096	227	376	337
		Total	12	1440	8416	3186		
Mid-Rise	R2B	*1st*	3	360	2770	1002	4950	4448
		2nd	3	360	2768	1002	1893	1140
		3rd	3	360	2592	996	1614	1067
		4th	3	360	2593	997	1485	992
		5th	3	360	2440	991	880	628
		6th	3	360	2175	864	798	621
		7th	3	360	923	186	289	228
		Total	21	2520	16262	6037		
	R2C	*1st*	3	360	2813	1044	2771	2543
		2nd	3	360	2844	1055	1319	900
		3rd	3	360	2642	1041	1442	1146
		4th	3	360	2671	1051	1516	1164
		5th	3	360	2495	1036	951	777
		6th	3	360	2501	1038	886	724
		7th	3	360	1088	233	448	367
		Total	21	2520	17054	6497		
High-Rise	R2B	*1st*	3	360	3188	1005	14318	13796
		2nd	3	360	2969	1003	13954	13436
		3rd	3	360	2777	1000	13222	12715
		4th	3	360	2764	997	12229	11751
		5th	3	360	2760	996	11022	10596
		6th	3	360	2591	995	9673	9316
		7th	3	360	2584	994	8135	7861
		8th	3	360	2461	996	6344	6157
		9th	3	360	2436	991	4213	4113
		10th	3	360	1216	213	1387	1367
		Total	30	3600	25746	9189		
	R2C	*1st*	3	360	3311	1072	5197	4744
		2nd	3	360	3115	1077	1583	1068
		3rd	3	360	2779	1029	1566	1084
		4th	3	360	2815	1041	2091	1505
		5th	3	360	2820	1046	2135	1459
		6th	3	360	2612	1032	1883	1400
		7th	3	360	2692	1059	2266	1731
		8th	3	360	2440	1023	1106	842
		9th	3	360	2498	1046	1532	1291
		10th	3	360	1277	238	1560	1193
		Total	30	3600	26359	9662		

Concentrically cross braces were selected to be compared to EBF system at this study. R2C configuration in figure above shows typically location of braces in end bays. At the same configuration also eccentric braces were located in the model. During the design of these two systems it has been tried to keep roughly close DCR at braces.

The unbalanced length ratio of all braces have been input as 1 as major and minor axis, and the effective length ratio of the braces were selected as 0.7 for all braces due to the fact that the connection of both ends of the braces is enough rigid and this manner were reflected to the models in this way.

The table below is to compare the base shear and story shear of R2B and R2C structures and compare these outputs. As it can be seen from the view of force control, R2B is dominating over R2C; the story shears at R2B are much higher than R2C.

At R2B high-rise building the base shear is about 3 times the base shear obtained by R2C. so far it was obviously observed that EBF system could provide much higher stiffness for the buildings in comparable to CBF system.

One of the scale to determine the performance level of the buildings is to check the drift ratio of each story and limiting it under some restriction presented in FEMA 365. At the table below the story displacement of each building has been shown. Comparing D_x oat mid-rise shows that displacement of base story at R2C is about 2 times less than it in R2B.

After a quick look it can be easily observed that story displacement of R2C which is retrofitted by CBF is less than R2B. By taking into mind that story shear of R2B was higher than R2C in previous table; it may cause a confused point here, when a building has higher story shear to other buildings it is not supposed to have less story displacement. In FEMA 356 there force controlled and displacement controlled combination to perform this confusing point. R2B which had high story shear would have obviously higher story displacement, so for stating certain comparison it is so early and commenting is not possible from only base shear and story displacements.

Table 7.7. Story displacements of R2B and R2C

Structure			*Displacement*	
			D_x [mm]	*D_y [mm]*
Low-Rise	*R2B*	*1st*	17	18
		2nd	17	17
		3rd	13	14
		4th	7	7
	R2C	*1st*	8	8
		2nd	8	8
		3rd	8	9
		4th	6	6
Mid-Rise	*R2B*	*1st*	19	19
		2nd	25	26
		3rd	26	27
		4th	23	24
		5th	21	21
		6th	16	16
		7th	9	9
	R2C	*1st*	9	9
		2nd	12	12
		3rd	14	15
		4th	14	15
		5th	15	15
		6th	12	13
		7th	9	10
High-Rise	*R2B*	*1st*	13	14
		2nd	23	23
		3rd	24	25
		4th	24	25
		5th	23	24
		6th	23	24
		7th	21	22
		8th	19	20
		9th	17	18
		10th	8	10
	R2C	*1st*	8	8
		2nd	12	12
		3rd	14	14
		4th	16	17
		5th	17	18
		6th	20	20
		7th	20	21
		8th	19	20
		9th	20	21
		10th	13	15

The natural period of R2B at low-rise is 0.41s and at R2C is 0.3; this period is in Y-direction at both models. The table below shows that the first three periods of R2B are longer to R2C. up to now, it was observed that R2B provides high story shear, high story displacement and longer periods in comparable to R2C.

Table 7.8. Participation mass ratios of R2B and R2C

Structure		Mode	Period (s)	Participation Mass Ratio			Sum of Participation Mass Ratio		
				UX	UY	UZ	SumUX	SumUY	SumUZ
Low-Rise	R2B	1	0,41	0,00	0,87	0,00	0,00	0,87	0,00
		2	0,40	0,87	0,00	0,00	0,87	0,87	0,00
		3	0,29	0,00	0,00	0,00	0,87	0,87	0,00
	R2C	1	0,30	0,00	0,82	0,00	0,00	0,82	0,00
		2	0,29	0,83	0,00	0,00	0,83	0,82	0,00
		3	0,20	0,00	0,00	0,00	0,83	0,82	0,00
Mid-Rise	R2B	1	0,69	0,00	0,80	0,00	0,00	0,80	0,00
		2	0,67	0,80	0,00	0,00	0,80	0,80	0,00
		3	0,48	0,00	0,00	0,00	0,80	0,80	0,00
	R2C	1	0,49	0,00	0,75	0,00	0,00	0,75	0,00
		2	0,48	0,75	0,00	0,00	0,75	0,75	0,00
		3	0,32	0,00	0,00	0,00	0,75	0,75	0,00
High-Rise	R2B	1	0,97	0,00	0,75	0,00	0,00	0,75	0,00
		2	0,94	0,75	0,00	0,00	0,75	0,75	0,00
		3	0,67	0,00	0,00	0,00	0,75	0,75	0,00
	R2C	1	0,72	0,00	0,68	0,00	0,00	0,68	0,00
		2	0,70	0,69	0,00	0,00	0,69	0,68	0,00
		3	0,46	0,00	0,00	0,00	0,69	0,68	0,00

Flexural demand capacity ratios of columns in R2B and R2C buildings have been shown in the table below. Demand capacity ratio is a good scale of comparison, because the percentage and the range of DCR distribution through the building helps to draw the way of comparing and approximated results could be derived at this step.

At low-rise comparison in the table below, 100% of the existing columns in upper stories have DCR less than 1 which is good result. The only comparison could be done from base story and 55% of existing base columns are less than 1 in R2B but in R2C only 45% of them are less than 1 and the rest are have DCR between 1 and 2 range.

At mid-rise buildings, at R2B no columns has DCR higher 4 but in R2C about 35% of columns in base story and 25% in third story have DCR higher than 4. Otherwise about 60% of columns in base story of R2B have DCR between the ranges of 2 and 4. for having an overall and at the same time true observation, it is of benefit to comment only upon DCR less than 1. So, the percentage of columns in R2B, which have DCR less than 1 is considerably high rather than R2C.

At high-rise buildings, at R2B no column has DCR higher than 2 however at R2C in story 1,2 and 3 some columns have passed the over DCR higher than 2 and even higher than 4 in story 2 and 3. At R2B except the base story almost all of the columns in upper stories have DCR less than 1.

The table below shows the range of obtained DCR and distribution of that in stories, and it was observed that DCRs are low at R2B compare to R2C. it should be stated that for this table have been prepared as an pre-request for the acceptance criteria table in the following part and determining that which building has better performance is so early without observing acceptance table.

The table below presents the failing mode of columns at each story. The highlighted character of this table is that it involves m and k factors. K corresponds for knowledge factor of data in the buildings which has been assumed as 0.75 in this study and m-factor corresponds for each column's seismic modification factor. The certain comparison could be done easily from this table and failure mode of each element (Axial, shear and Flexure) could be observed.

In low-rise buildings, there is no failed column at R2B which shows that this building conform proposed performance level (LS). There are 11 columns with flexural failure mode at the base story of R2C which means about 55% of the columns at this story have failed, so this building is not match with the LS performance level.

In the mid-rise buildings, there are 12 columns at base story which do not fulfill the condition of LS performance level, in other word about 60% of base columns at R2B fail from flexural mode and because of this the whole building will undergo the mechanism. At R2C in addition to flexural failures there are also some columns failed because of axial. As a matter of fact, these columns fail under tensile force at end-bays. The CBF at each edge creates rigid and stiff along the building at that span however existing columns can not resist for this additional force and can not transfer it to the foundation and fail. So, neither R2B nor R2C provide required conditions for life safety performance level and both systems fail at mid-rise version.

Table 7.9. Flexural demand-to-capacity ratios for columns of R2Band R2C/ expected capacities, under PMM interaction and prior to brittle failure mode control.

Structure			*Number of Columns with*				*% Columns with*				*Total # Existing*
			DCR<1	*1≤DCR≤ 2*	*2≤DCR≤4*	*DCR>4*	*DCR<1*	*1≤DCR≤ 2*	*2≤DCR≤4*	*DCR>4*	
Low-Rise	*R2B*	*1*	11	9	0	0	55	45	0	0	20
		2	20	0	0	0	100	0	0	0	20
		3	20	0	0	0	100	0	0	0	20
		4	20	0	0	0	100	0	0	0	20
	R2C	*1*	8	11	0	0	40	55	0	0	20
		2	20	0	0	0	100	0	0	0	20
		3	20	0	0	0	100	0	0	0	20
		4	20	0	0	0	100	0	0	0	20
Mid-Rise	*R2B*	*1*	4	4	12	0	20	20	60	0	20
		2	20	0	0	0	100	0	0	0	20
		3	20	0	0	0	100	0	0	0	20
		4	18	2	0	0	90	10	0	0	20
		5	18	2	0	0	90	10	0	0	20
		6	18	2	0	0	90	10	0	0	20
		7	20	0	0	0	100	0	0	0	20
	R2C	*1*	13	0	0	7	65	0	0	35	20
		2	12	7	1	0	60	35	5	0	20
		3	12	0	3	5	60	0	15	25	20
		4	17	3	0	0	85	15	0	0	20
		5	19	1	0	0	95	5	0	0	20
		6	20	0	0	0	100	0	0	0	20
		7	20	0	0	0	100	0	0	0	20
High-Rise	*R2B*	*1*	8	12	0	0	40	60	0	0	20
		2	20	0	0	0	100	0	0	0	20
		3	20	0	0	0	100	0	0	0	20
		4	20	0	0	0	100	0	0	0	20
		5	20	0	0	0	100	0	0	0	20
		6	20	0	0	0	100	0	0	0	20
		7	18	2	0	0	90	10	0	0	20
		8	18	2	0	0	90	10	0	0	20
		9	18	2	0	0	90	10	0	0	20
		10	20	0	0	0	100	0	0	0	20
	R2C	*1*	12	7	1	0	60	35	5	0	20
		2	13	4	2	1	65	20	10	5	20
		3	12	3	3	2	60	15	15	10	20
		4	19	1	0	0	95	5	0	0	20
		5	20	0	0	0	100	0	0	0	20
		6	20	0	0	0	100	0	0	0	20
		7	20	0	0	0	100	0	0	0	20
		8	20	0	0	0	100	0	0	0	20
		9	18	2	0	0	90	10	0	0	20
		10	20	0	0	0	100	0	0	0	20

At high-rise buildings, there are also columns failed of flexural and axial. 40% of columns at R2B fail because of axial failure mode and it is observed that there is no flexural failure, the reason is that the columns fail under tensile action and do not find enough ductility to behave under shear or flexural. The axial failure is of the most undesirable failure in retrofitted buildings. So as gravity goes up with the number of stories the failure mode could pass from flexural to axial failure in EBF and CBF buildings.

Table 7.10. Number of columns failing the acceptance criteria at R00 and R1A / Including m and k factors

Structure			*Failure Mode*			*Failed Columns*	*Total Columns*	*% Failed*
			Axial	*Shear*	*Flexure*			
Low-Rise	*R2B*	*1*	0	0	0	0	20	0
		2	0	0	0	0	20	0
		3	0	0	0	0	20	0
		4	0	0	0	0	20	0
	R2C	*1*	0	0	11	11	20	55
		2	0	0	0	0	20	0
		3	0	0	0	0	20	0
		4	0	0	0	0	20	0
Mid-Rise	*R2B*	*1*	0	0	12	12	20	60
		2	0	0	0	0	20	0
		3	0	0	0	0	20	0
		4	0	0	0	0	20	0
		5	0	0	0	0	20	0
		6	0	0	0	0	20	0
		7	0	0	0	0	20	0
	R2C	*1*	12	0	0	12	20	60
		2	8	0	0	8	20	40
		3	8	0	0	8	20	40
		4	0	0	0	0	20	0
		5	0	0	0	0	20	0
		6	0	0	0	0	20	0
		7	0	0	0	0	20	0
High-Rise	*R2B*	*1*	8	0	0	8	20	40
		2	0	0	0	0	20	0
		3	0	0	0	0	20	0
		4	0	0	0	0	20	0
		5	0	0	0	0	20	0
		6	0	0	0	0	20	0
		7	0	0	0	0	20	0
		8	0	0	0	0	20	0
		9	0	0	0	0	20	0
		10	0	0	0	0	20	0
	R2C	*1*	1	0	6	7	20	35
		2	8	0	0	8	20	40
		3	8	0	0	8	20	40
		4	0	0	0	0	20	0
		5	0	0	0	0	20	0
		6	0	0	0	0	20	0
		7	0	0	0	0	20	0
		8	0	0	0	0	20	0
		9	0	0	0	0	20	0
		10	0	0	0	0	20	0

7.3. Optimized Retrofit strategy_ R3D

In section 8.2 R2B and R2C were compared to each other and it was resulted that except R2B low-rise no other buildings fulfill the conditions of Life Safety performance level. However the main purposes were after the contribution of EBF and CBF to existing buildings from seismic resistance point of view. As a result of comparison in section 8.2, R2B monitored that it behave better against lateral force than R2C. Moreover, R2B appeared fewer failures and had less column DCR, so it was closer to life safety performance level rather than R2C, and EBF showed prevailing dominance upon CBF.

As it was stated, except R2B at low-rise no buildings provide life safety, at this part R2B which monitored better performance, would be optimized and demand performance level would be provided. As a matter of fact, at most of the retrofit buildings more than one single retrofit strategy are used. For instance RC shear walls are placed and at the same time some of columns and beams is wrapped by CFRP. So at this part, as a second scope of the study, R2B buildings would be retrofitted to furnish life safety performance level. However during the selection of second retrofit strategy it has been tried to set it as second plan and primary strategy which is EBF would not overlapped. In previous sections, it was observed that R2B fulfill almost 90% of LS conditions so for fulfilling the remained it has been decided to wrap the adjacent columns to EBF. To wrap these columns systematically, steel jacketing is of the most effective methods, since there was tensile axial failure mode at these columns and by using the high yield strength of steel this deficiency could be overcome.

Jacketing is a technique widely used for repair or strengthening of reinforced concrete columns. Columns cross section is enlarged by forming a jacketed around the existing columns, and additional longitudinal and transverse reinforcement is provided.

Recent earthquake around the world have confirmed the poor seismic behavior of reinforced concrete columns incorporating typical pre-1970 reinforcement details. Major problems that may cause the collapse of these columns are inadequate shear strength and inadequate ductility. Up to now several techniques are available for strengthening of concrete columns. One of the most effective methods is steel jacketing technique. Many researchers have conducted experimental and analytical studies on various factors which affect the efficiency and performance of this method. In this study based on constructional conditions practical methods of strengthening of reinforced concrete column using steel jacket has been presented.

Same as R00, R1A, R2C and R2B buildings, the optimized building has been called as R2D. R2D is modified R2B building and the only optimization is additional steel jacketing around adjacent columns to EBF. The preliminary design of R3D is to jacketing these columns along the height of the building, however it is strongly highlighted that wrapping along the building is not effective from economical point of view and jacketing of lower stories would probably be sufficient.

Table 7.11. Story forces of R3D

Structure		*Stroy No*	*Story Height (m)*	*Area (m^2)*	*Dead Load (kN)*	*Live Load (kN)*	*Stroy Shear Forces in X Direction (kN)*	*Stroy Shear Forces in Y Direction (kN)*
Loe-Rise	R3D	*1st*	3	360	2560	1005	2054	1940
		2nd	3	360	2529	996	587	404
		3rd	3	360	2263	869	675	542
		4th	3	360	1055	194	197	158
		Total	12	1440	8407	3063		
Mid-Rise	R3D	*1st*	3	360	2884	1003	6161	5643
		2nd	3	360	2863	997	2131	1380
		3rd	3	360	2802	998	1901	1290
		4th	3	360	2788	995	1494	987
		5th	3	360	2723	991	1096	796
		6th	3	360	2457	865	936	761
		7th	3	360	1250	191	154	129
		Total	21	2520	17766	6041		
High-Rise	R3D	*1st*	3	360	3421	1003	9406	8746
		2nd	3	360	3231	1001	3567	2745
		3rd	3	360	3143	997	1978	1163
		4th	3	360	3137	995	2012	1217
		5th	3	360	3112	989	1650	908
		6th	3	360	3046	990	1923	1339
		7th	3	360	3050	992	1864	1396
		8th	3	360	2949	976	986	702
		9th	3	360	2732	868	1692	1492
		10th	3	360	1569	182	312	306
		Total	30	3600	29391	8992		

Similar to previous comparison at optimization part also story shear and story displacement of R3D building has been carried out. However no comparison has been done at this part, since these outputs have been taken for plastic design of link element and nonlinear static analysis in following chapter.

Table 7.12. Story displacements of R3D

Structure			*Displacement*	
			D_x [mm]	*D_y [mm]*
Low-Rise	*R3D*	*1*	17	18
		2	17	17
		3	13	14
		4	7	7
Mid-Rise	*R3D*	*1*	18	19
		2	26	27
		3	26	27
		4	24	24
		5	20	21
		6	15	16
		7	10	10
High-Rise	*R3D*	*1*	13	13
		2	24	24
		3	26	27
		4	26	27
		5	24	25
		6	24	24
		7	21	22
		8	18	19
		9	15	15
		10	10	11

Table 7.13. Participation mass ratios of R3D

Structure		Mode	Period (s)	Participation Mass Ratio			Sum of Participation Mass Ratio		
				UX	UY	UZ	SumUX	SumUY	SumUZ
Low	R3D	1	0,41	0,00	0,87	0,00	0,00	0,87	0,00
		2	0,40	0,87	0,00	0,00	0,87	0,87	0,00
		3	0,29	0,00	0,00	0,00	0,87	0,87	0,00
Mid	R3D	1	0,71	0,00	0,80	0,00	0,00	0,80	0,00
		2	0,69	0,80	0,00	0,00	0,80	0,80	0,00
		3	0,51	0,00	0,00	0,00	0,80	0,80	0,00
High	R3D	1	1,00	0,00	0,76	0,00	0,00	0,76	0,00
		2	0,97	0,76	0,00	0,00	0,76	0,76	0,00
		3	0,72	0,00	0,00	0,00	0,76	0,76	0,00

At flexural demand capacity of R3D almost all of the columns have DCR less than 1, however at mid-rise and high-rise buildings respectively 5% and 30% of base columns have DCR between the range of 1 and 2. These columns will probably be saved by using their modification factor, m-factor. For monitoring this acceptance table should be observed.

Table 7.14. Flexural Demand-to-Capacity Ratios for Columns of R3D/ expected capacities, under PMM interaction and prior to brittle failure mode control

Structure			*Number of Columns with*				*% Columns with*				*Total # Existing*
			DCR<1	*1≤DCR≤2*	*2≤DCR≤4*	*DCR>4*	*DCR<1*	*1≤DCR≤2*	*2≤DCR≤4*	*DCR>4*	
Low-Rise	*R3D*	*1*	20	0	0	0	100	0	0	0	20
		2	20	0	0	0	100	0	0	0	20
		3	20	0	0	0	100	0	0	0	20
		4	20	0	0	0	100	0	0	0	20
Mid-Rise	*R3D*	*1*	19	1	0	0	95	5	0	0	20
		2	20	0	0	0	100	0	0	0	20
		3	20	0	0	0	100	0	0	0	20
		4	20	0	0	0	100	0	0	0	20
		5	20	0	0	0	100	0	0	0	20
		6	20	0	0	0	100	0	0	0	20
		7	20	0	0	0	100	0	0	0	20
High-Rise	*R3D*	*1*	14	6	0	0	70	30	0	0	20
		2	20	0	0	0	100	0	0	0	20
		3	20	0	0	0	100	0	0	0	20
		4	20	0	0	0	100	0	0	0	20
		5	20	0	0	0	100	0	0	0	20
		6	20	0	0	0	100	0	0	0	20
		7	20	0	0	0	100	0	0	0	20
		8	20	0	0	0	100	0	0	0	20
		9	20	0	0	0	100	0	0	0	20
		10	20	0	0	0	100	0	0	0	20

Table 7.15. Number of Columns Failing the Acceptance Criteria at R3D / including m and k factors

Structure			*Failure Mode*			*Failed Columns*	*Total Columns*	*% Failed*
			Axial	*Shear*	*Flexure*			
Low-Rise	*R3D*	*1*	0	0	0	0	20	0
		2	0	0	0	0	20	0
		3	0	0	0	0	20	0
		4	0	0	0	0	20	0
Mid-Rise	*R3D*	*1*	0	0	0	0	20	0
		2	0	0	0	0	20	0
		3	0	0	0	0	20	0
		4	0	0	0	0	20	0
		5	0	0	0	0	20	0
		6	0	0	0	0	20	0
		7	0	0	0	0	20	0
High-Rise	*R3D*	*1*	0	0	0	0	20	0
		2	0	0	0	0	20	0
		3	0	0	0	0	20	0
		4	0	0	0	0	20	0
		5	0	0	0	0	20	0
		6	0	0	0	0	20	0
		7	0	0	0	0	20	0
		8	0	0	0	0	20	0
		9	0	0	0	0	20	0
		10	0	0	0	0	20	0

The following acceptance criteria table shows that no columns fail. Axial, shear and flexural failure modes at all cells are zero. The following table is only for proof this fact that failed columns with DCR between 1 and 2 in previous table have used their m-factor right and coming lateral forces have been reduced to reasonable amount. In consequence of following table, R3D optimized building at low, mid and high-rise types fulfill the required conditions of life safety performance level.

8. NONLINEAR ANALYSIS METHOD

8.1. Introduction

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 member and building.

8.1.1. 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 section 2.6.
- 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 coordinates 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 refers 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 reinforcements 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 ductile manner as sectional level for structural members. Also the obtained shear force demands will be compared with the shear capacity of sections to make a consideration in brittle manner (Aydinoglu, 2003).

8.1.2. Adding Confinement

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.

8.1.3. 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 much large overall structural drifts (Bertero, 1997).

The beam column joints must be strengthened to allow development of larger moments, in addition to column strengthening. Joints must be strengthened for both action of internal forces and joint forces and must 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.

8.1.4. Coefficient Method

The static push over analysis has been done on R00 and R3D buildings. R00 is the existing building and R3D is the building rehabilitated by eccentric steel braces as R2B configuration and optimized by column steel jacketing. In previous section using DCR and acceptance criteria it was observed that at mid-rise and high-rise modes of R2B and R2C buildings the life safety performance level is not furnished, so by optimizing R2B, R3D was obtained and selected for nonlinear analysis. The static push over analysis is to control the performance level and to see the exact location of plastic hinges.

The desired plastic hinge at R3D building is at link elements which would act as fuse against lateral forces. At section 6.2.2 it was stated that the desired plasticity is the one evaluated by shear, which mostly occurs at short links. So, at two ends of the link elements shear and flexural hinges have been assigned according to the default hinges in FEMA 356 presented at table 3.4. At the mid point of the braces there is axial hinge to control the behavior of brace which could be of tension or compression. As a matter of fact, the reason that braces have been located as working out-of-plane is to prevent their in-plane buckling and enforce them to resist against buckling. The desired plastic in braces is the plasticity evaluated by tension, because at tension the brace would yield and use all allowed capacity to dissipate the energy. However, in EBF systems except link element all other framing components are designed elastically and supposed to remain elastic and not yield. The only element which is allowed to yield is link element. In the table below the coefficients required for adjusting the push over parameters have been obtained for low, mid and high-rise buildings separately.

Table 8.1. Coefficients obtained from SAP 2000 for adjusting target displacement of existing building

Structure	C0	C1	C2	C3	Cm	Sa	R	Weight
Low	0,87	1,00	1,00	1,00	1,00	0,97	4,38	1295
Mid	0,74	1,00	1,00	1,00	1,00	0,67	5,56	2624
High	0,70	1,00	1,00	1,00	1,00	0,49	6,44	4162

Using by C-factors the acceleration required for applying the lateral force to the buildings has been obtained.

Table 8.2. The estimated displacement of the top level in existing building

Structure	Ki (KN/m)	Ke (KN/m)	Te	Ti	Vy (KN)	α	δ_t	δ_{tahmin}	V_y/W
Low	7811	7811	0,66	0,66	286	0,16	0,139	0,06	0,22
Mid	6166	6166	0,95	0,95	317	0,13	0,12	0,1	0,12
High	5033	5033	1,30	1,30	318	0,16	0,11	0,15	0,08

By normalizing the lateral force of the building the target displacement is obtained. Achieving K_i and K_e factors is a trial-error method which needs many analyzing in SAP2000 program. The C coefficient which is the coefficient of lateral forces at each step should be corrected after each step and the final C coefficient obtained after many analyzing is shown in the table above (Celep , 2007).

As it was stated before, there are two checks for controlling a retrofit strategy in terms of system, one is performance level of each single element and the other one is to check the story drift at each story. According to FEMA 356 the employable m-factor of each element should be calculated and DCR should be divided to this useable m-factor, which results in acceptance criteria of the element. For providing the proposed performance level all elements should be within permitted limit and also the system its own should obey the restrictions.

Table 8.1 shows the drift ratio of low, mid and high-rise existing buildings. The drift ratios are supposed to be less than 0.02. It can be easily seen that almost neither of them obey the life safety restriction and their mode in check cell is “NO”.

At table 8.3 and 8.4 coefficient factors of R3D building has been monitored. Because of the stiffness provided by the EBF system at R3D at low, mid and high-rise structures S_a acceleration under seismic effects is lower than existing buildings which shows that natural periods of these buildings is longer.

Table 8.3. Story drift limitations required for Life Safety level in existing building

				Interstory Drift Ratio	Mode	Life Safety Restriction	Check
	Story No.	δi(max)	hi	δi(max)/hi			
Low - Rise	1	0,037	3	0,012	<	0,02	OK
	2	0,075	3	0,025	>	0,02	NO
	3	0,095	3	0,032	>	0,02	NO
	4	0,100	3	0,033	>	0,02	NO
Mid - Rise	1	0,009	3	0,003	<	0,02	OK
	2	0,028	3	0,009	<	0,02	OK
	3	0,052	3	0,017	<	0,02	OK
	4	0,074	3	0,025	>	0,02	NO
	5	0,089	3	0,030	>	0,02	NO
	6	0,097	3	0,032	>	0,02	NO
	7	0,100	3	0,033	>	0,02	NO
High - Rise	1	0,005	3	0,002	<	0,02	OK
	2	0,018	3	0,006	<	0,02	OK
	3	0,042	3	0,014	>	0,02	NO
	4	0,070	3	0,023	>	0,02	NO
	5	0,097	3	0,032	>	0,02	NO
	6	0,120	3	0,040	>	0,02	NO
	7	0,133	3	0,044	>	0,02	NO
	8	0,143	3	0,048	>	0,02	NO
	9	0,148	3	0,049	>	0,02	NO
	10	0,150	3	0,050	>	0,02	NO

Table 8.4. Coefficients obtained from SAP 2000 for adjusting target displacement of retrofitted building

Structure	C0	C1	C2	C3	Cm	Sa	R	Weight
Low	0,85	1,37	1,00	1,00	1,00	1,10	2,56	1417
Mid	0,76	1,19	1,00	1,00	1,00	1,50	6,34	3170
High	0,80	1,00	1,00	1,00	1,00	1,50	6,57	5125

The estimated target displacement of the top level story on a selected joint is as table 8.5 for rehabilitated buildings. These values have been put into SAP 2000 for the initial step of push over after analyzing and viewing base shear-displacement curve a better estimation could be done and repeating the analysis. At coefficient method using the assumed stiffness and periods repeating the analysis is fallen and reaching the results is speeded up.

Table 8.5. The estimated displacement of the top level in retrofitted building

Structure	Ki (KN/m)	Ke (KN/m)	Te	Ti	Vy (KN)	α	δ_t	δ_{tahmin}	V_y/W
Low	71504	71504	0,22	0,22	608	0,09	0,071	0,06	0,43
Mid	43804	43804	0,41	0,41	750	0,05	0,14	0,1	0,24
High	31503	31503	0,60	0,60	1170	0,12	0,18	0,15	0,23

At table 8.6 the story drift ratios of each story at low, mid and high-rise structures have been computed and compared to limit value. it is observed that all calculated drift ratios are within limitation and it shows that R3D building fulfills Life Safety conditions. When all inter-story ratios are less and within rational portion, it could be understood that building has absorbed sufficient strength and stiffness for remaining rigid against lateral forces.

Table 8.6. Story drift limitations required for Life Safety level in retrofitted building

				Interstory Drift Ratio	Mode	Life Safety Restriction	Check
	Story No.	δi(max)	hi	δi(max)/hi			
Low - Rise	1	0,011	3	0,004	<	0,02	OK
	2	0,018	3	0,006	<	0,02	OK
	3	0,017	3	0,006	<	0,02	OK
	4	0,014	3	0,005	<	0,02	OK
Mid - Rise	1	0,007	3	0,002	<	0,02	OK
	2	0,016	3	0,005	<	0,02	OK
	3	0,020	3	0,007	<	0,02	OK
	4	0,019	3	0,006	<	0,02	OK
	5	0,015	3	0,005	<	0,02	OK
	6	0,012	3	0,004	<	0,02	OK
	7	0,010	3	0,003	<	0,02	OK
High - Rise	1	0,011	3	0,004	<	0,02	OK
	2	0,025	3	0,008	<	0,02	OK
	3	0,032	3	0,011	<	0,02	OK
	4	0,030	3	0,010	<	0,02	OK
	5	0,023	3	0,008	<	0,02	OK
	6	0,014	3	0,005	<	0,02	OK
	7	0,007	3	0,002	<	0,02	OK
	8	0,004	3	0,001	<	0,02	OK
	9	0,003	3	0,001	<	0,02	OK
	10	0,002	3	0,001	<	0,02	OK

8.1.5. Capacity Curves of Existing Buildings – R00

This part of the nonlinear study is based on the monitoring the outputs of push over analysis. The aim of push over was to observe the order of plastic hinges and the sequence of occurring. At existing buildings the collapse happens before reaching the target displacement which shows that the structures are brittle and can not resist against upcoming forces.

At low-rise building the mechanisms occurs when the base joint of base columns yield and collapse at that point, it is when displacement reaches 0.1 m and can not continue to yielding after this point. In figure 9.2 the location of yielded points can be seen. The upper story does not yield neither in column nor in beams which is a sign of weakness in framing system, because every separate building have a rate of modification seismic factor or m-factor as FEMA 356 and this building can not use this modification because of collapsing at lower story.

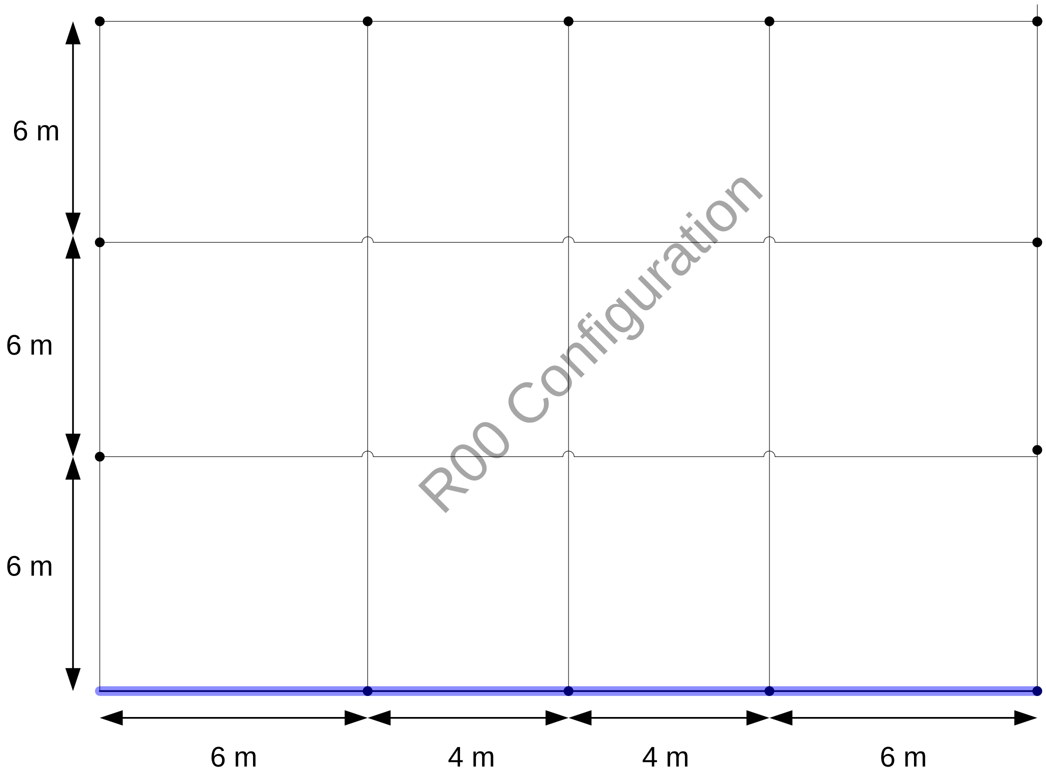


Figure 8.1. Selected 2D end-bay frame to be analyzed in existing building – low-rise

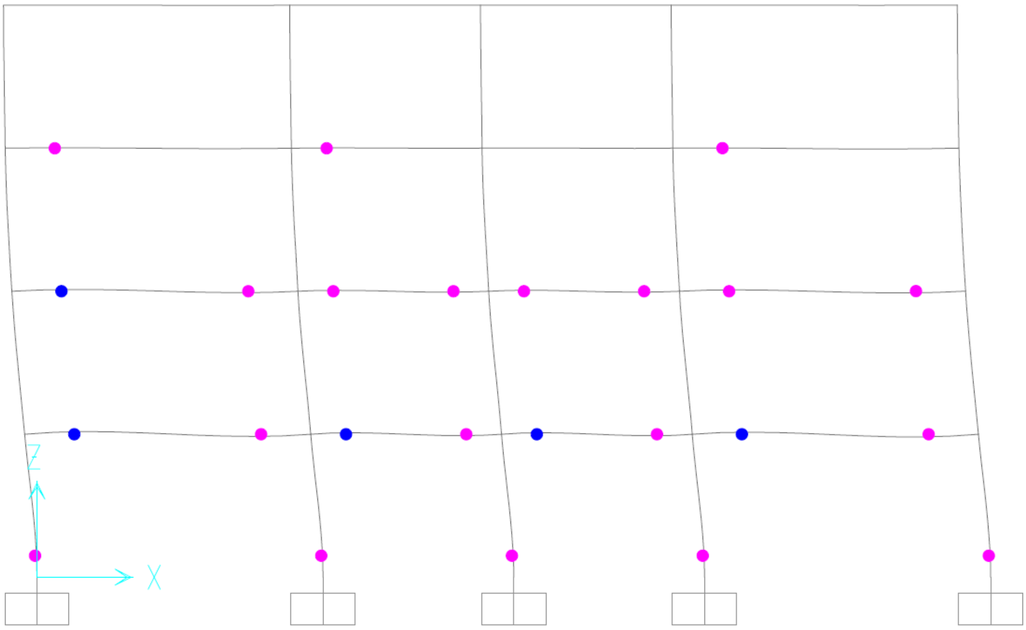


Figure 8.2. Mechanisms mode of exiting building – low-rise

At this study, the push-over analysis has been done on selected 2D frames which are shown in figure 9.1 and 9.2. at existing R00 building the failure mechanisms is shown in figure 9.2, the beam mechanisms is governing the system and systems can occupy up only 120 mm at top level story according to figure 9.3. The capacity curve of existing low, mid is shown in figure 9.3.

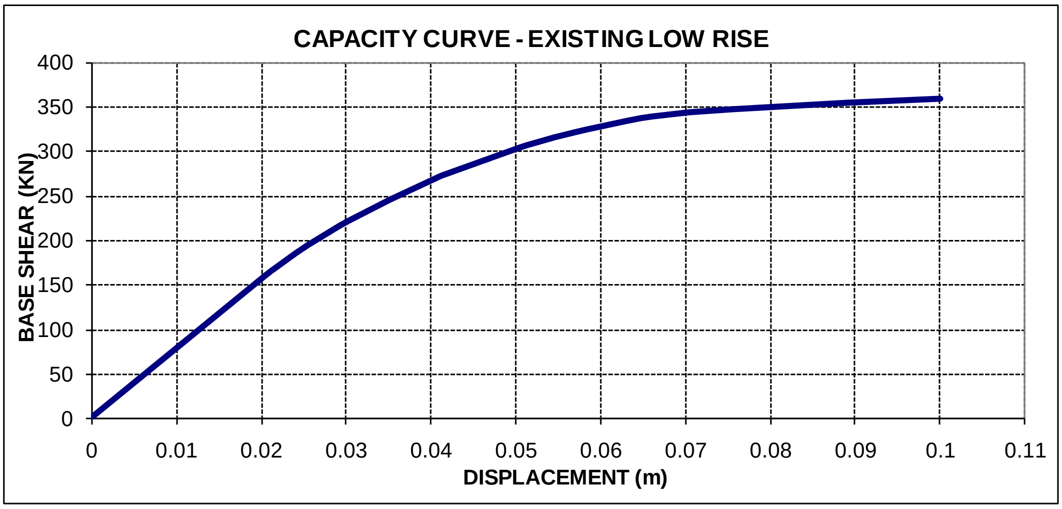


Figure 8.3. Capacity curve of existing low-rise building

The mid-rise building has been assumed to go up the displacement of 0.1 m same as low-rise building. The aim of this was to see only the change of capacity curve up to this point. It was observed that beams start yielding and according to figure mid-rise capacity curve, when gravity increases some strength could be obtained for the building. For instance at the capacity curve of low-rise building the ultimate base shear is about 350 KN but at mid-rise building the ultimate base shear has not been reached and the curve is able to its was after displacement of 0.1 m. it shows that mid-rise building is more ductile in comparable to low-rise building. It should be emphasized that this result is only carried out from comparison of low-rise and mid-rise capacity curves and should not be generalized to other systems. At all buildings plastic hinges have been located at two ends of columns as P-M2-M3 and at the middle of column there is plastic hinge because of axial force. In beams plastic hinges are placed at two ends and middle as M3. All hinge properties in columns have been taken from PCA-column program and in beams default hinge properties from FEMA 356 have been taken into account.

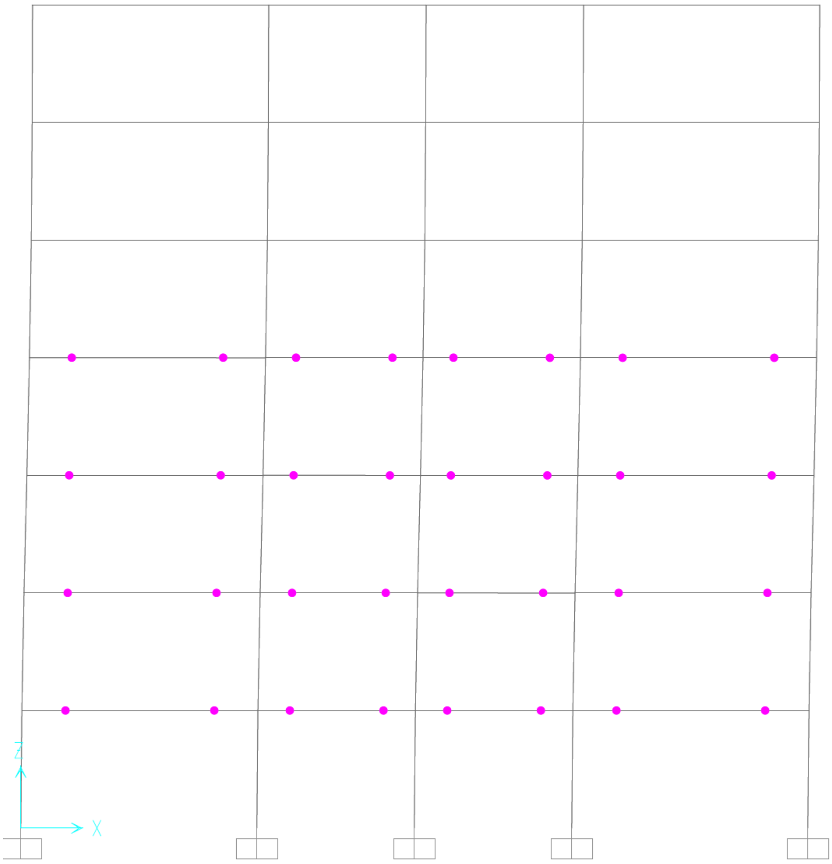


Figure 8.4. The mechanism stage of R00 mid-rise building

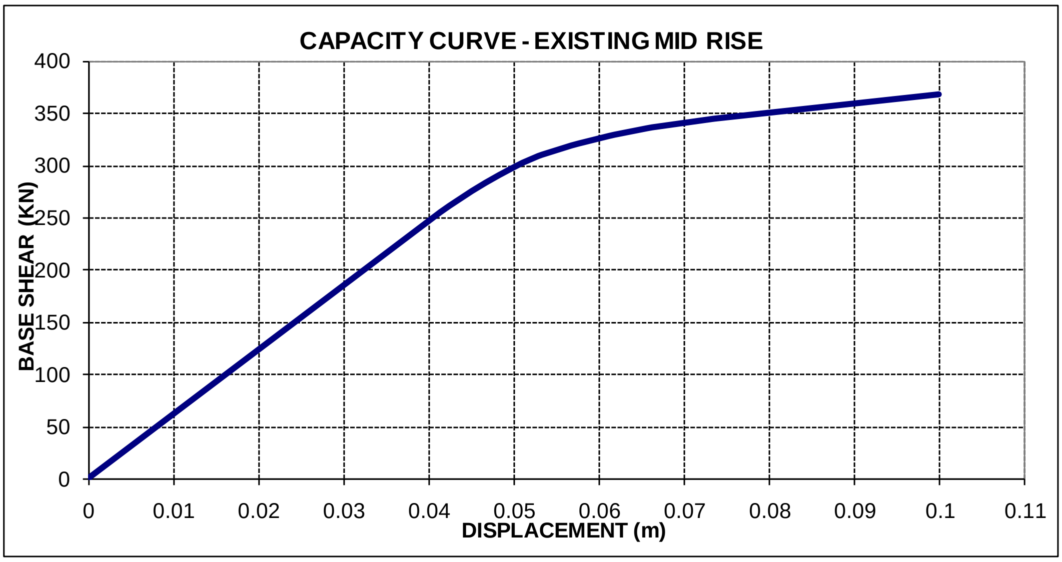


Figure 8.5. Capacity curve of existing mid-rise building

The estimated target displacement of high-rise building has been gain as 0.15 and the buildings have been controlled as monitored displacement up to this point. The same result of mid-rise building is repeated herein, the capacity curve shows that building has more enough strength to go beyond this point however the mechanisms has been obtained as beam mechanisms which is sign of collapse in the building. the ultimate base shear up to estimated target displacement is about 380 KN which is more than low and mid-rise buildings.

All in all, from capacity curves of existing buildings it could be resulted that this buildings reach beam mechanisms which is not desirable for any performance level. Since columns could not take additional forces to act flexural or shear even axial plastic behavior. So due to beam mechanisms at existing buildings investigating column behavior is almost impossible.

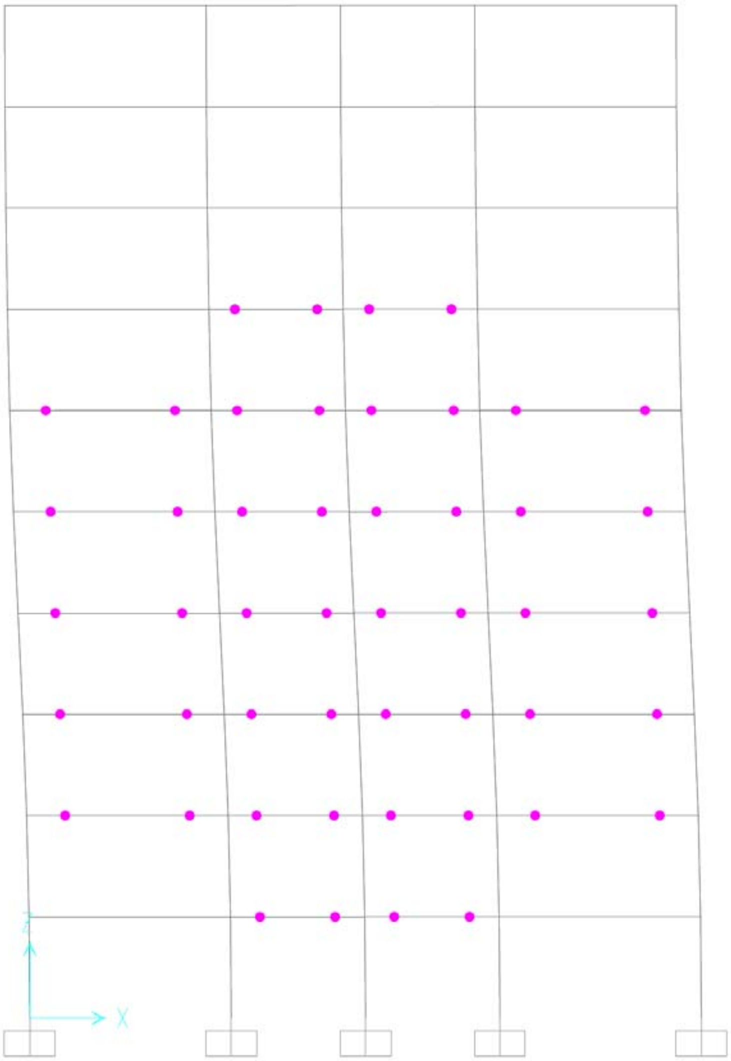


Figure 8.6. The mechanism stage ofR00 high-rise building

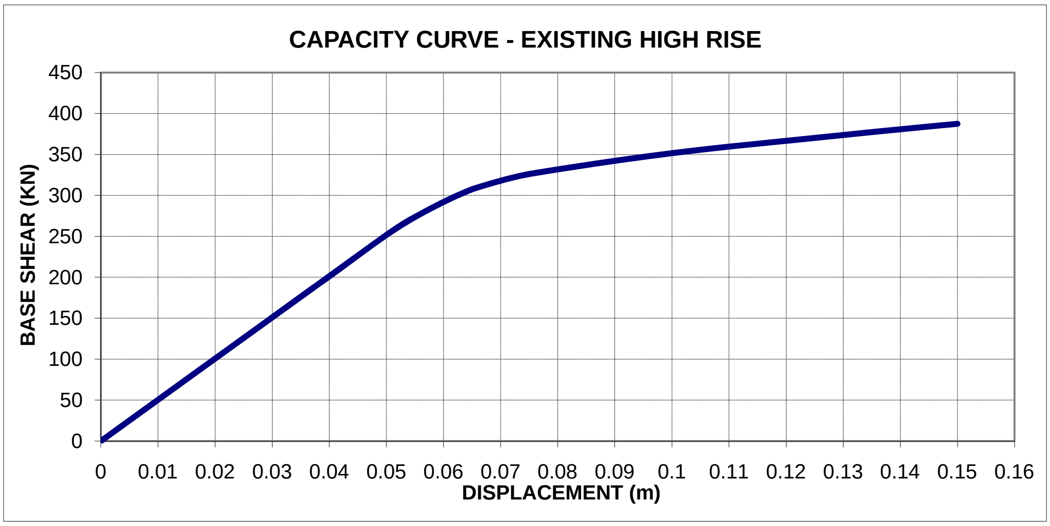


Figure 8.7. Capacity curve of existing high-rise building

8.1.6. Capacity Curves of Retrofitted Buildings – R3D

The desired order of yielding in the retrofitted buildings is to yield at link elements, as a matter of fact no yielding is expected at all other elements up to target displacement. At the selected 2D frame the order of creating yield points is exhibiting in following figures.

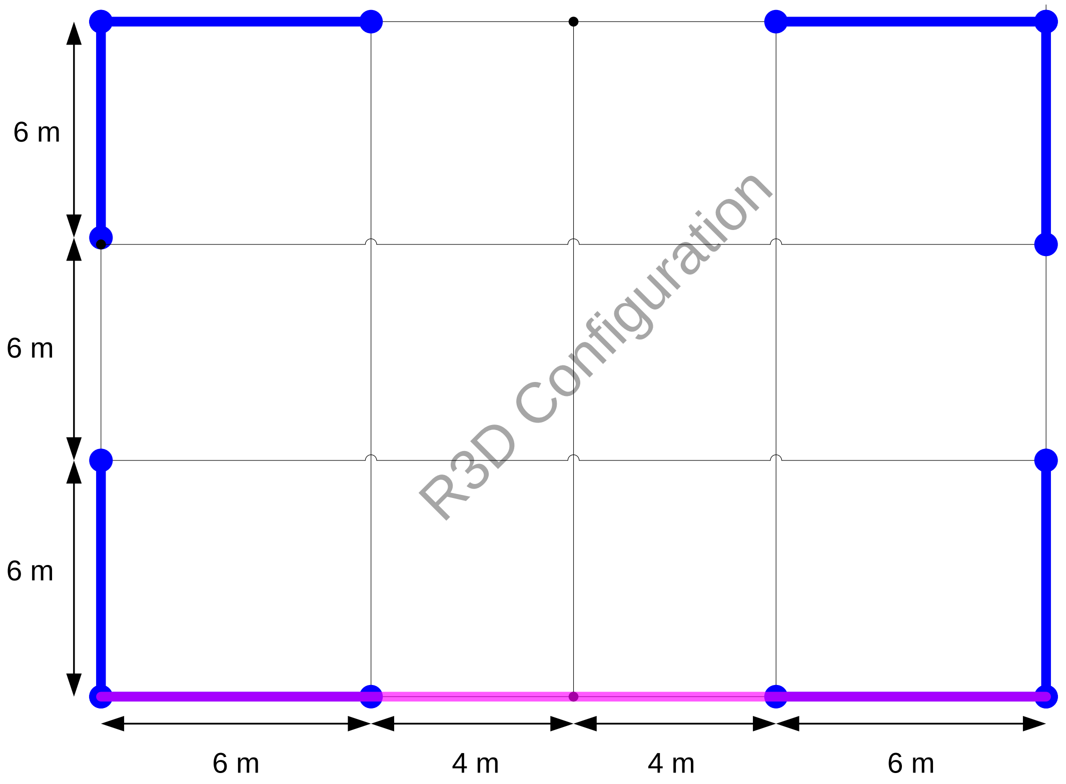


Figure 8.8. Selected 2D end-bay frame to be analyzed in retrofitted building

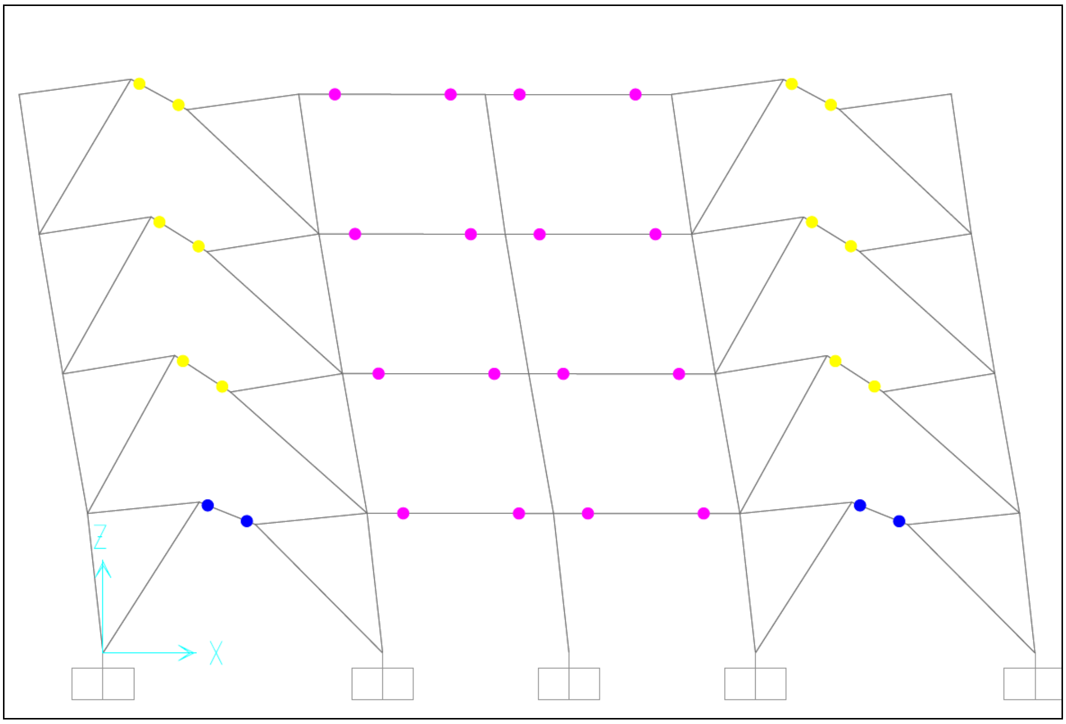


Figure 8.9. The mechanism stage of R00 low-rise building

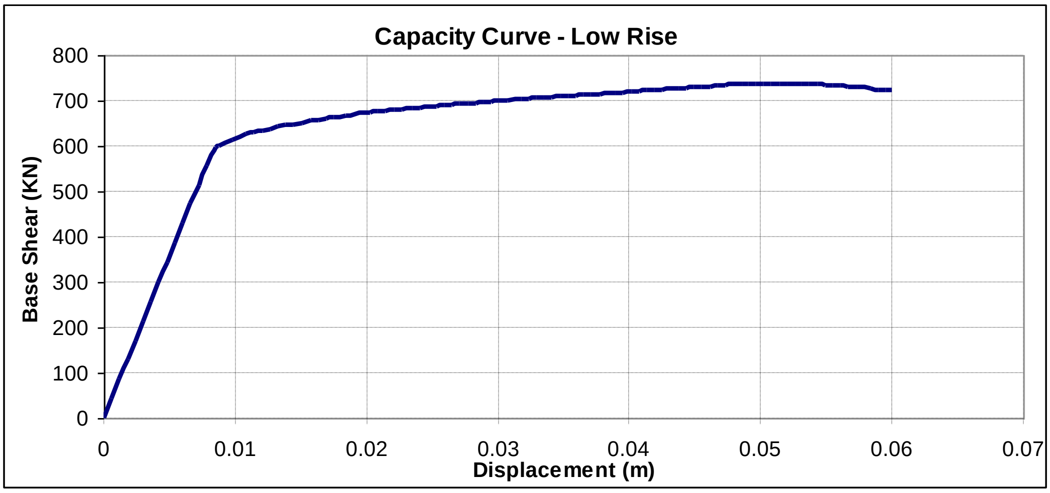


Figure 8.10. Capacity curve of retrofitted low-rise building

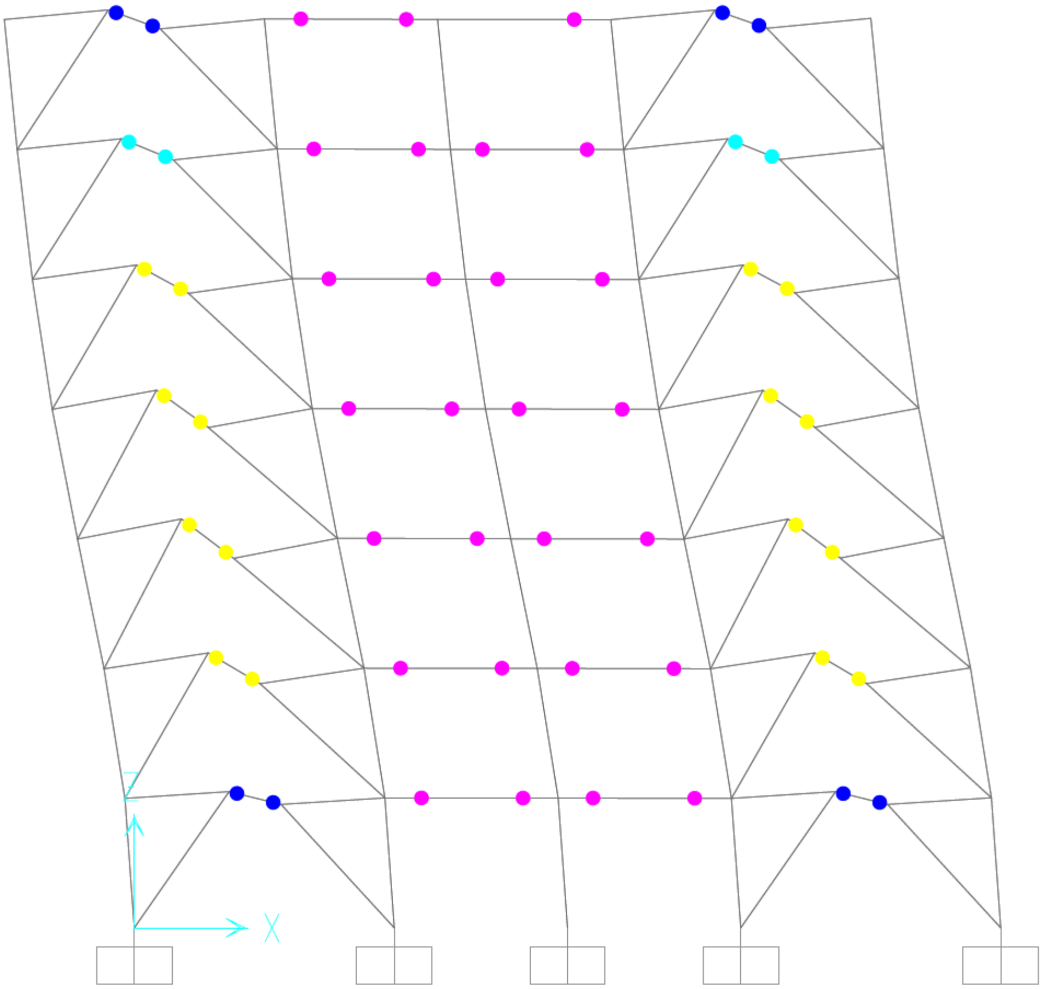


Figure 8.11. The mechanism stage of R3D mid-rise building

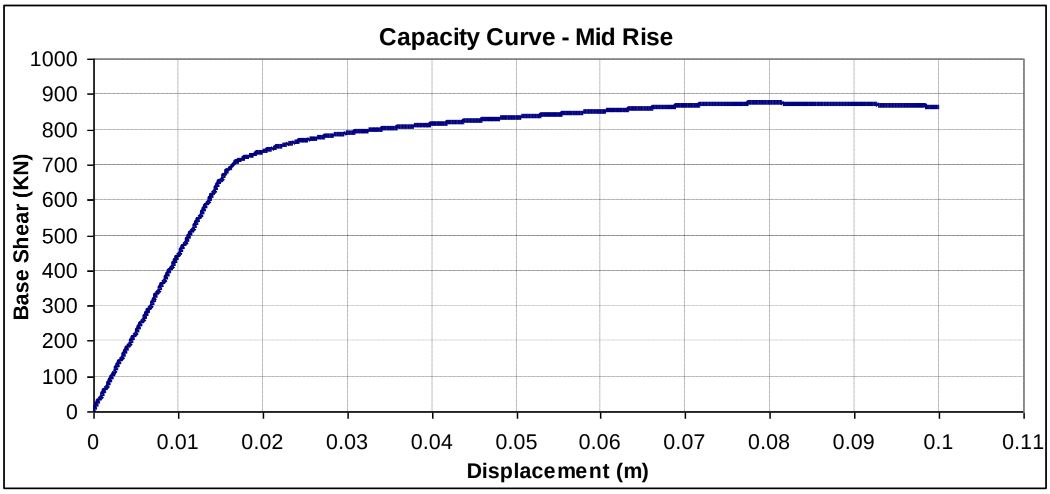


Figure 8.12. Capacity curve of retrofitted mid-rise building

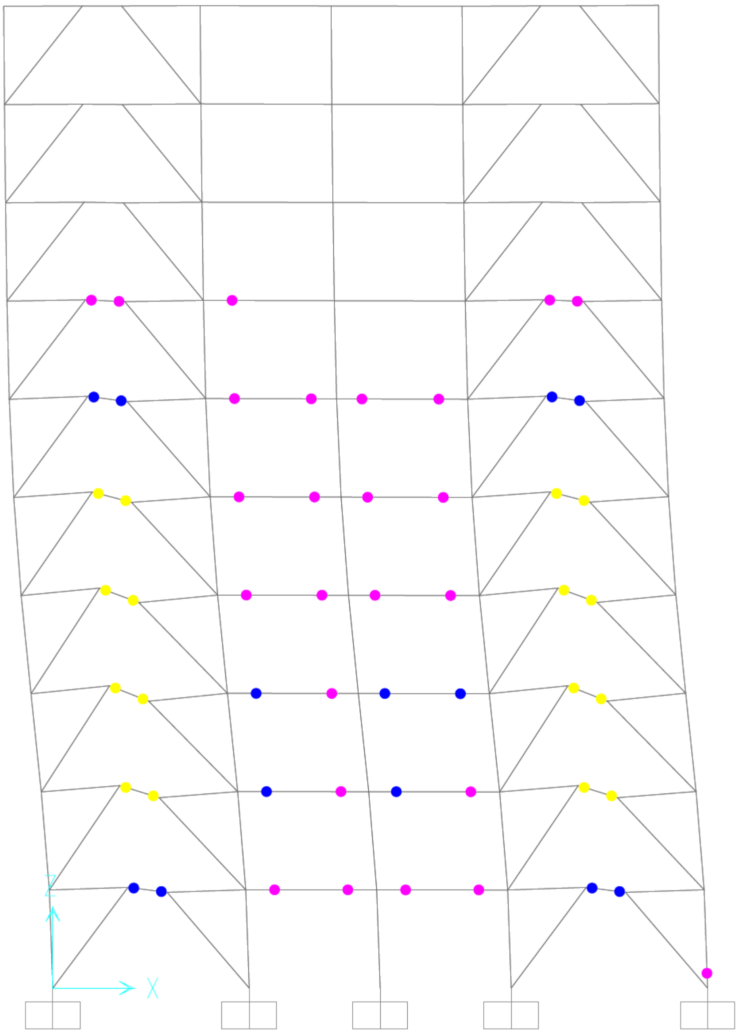


Figure 8.13. The mechanism stage of R3D high-rise building

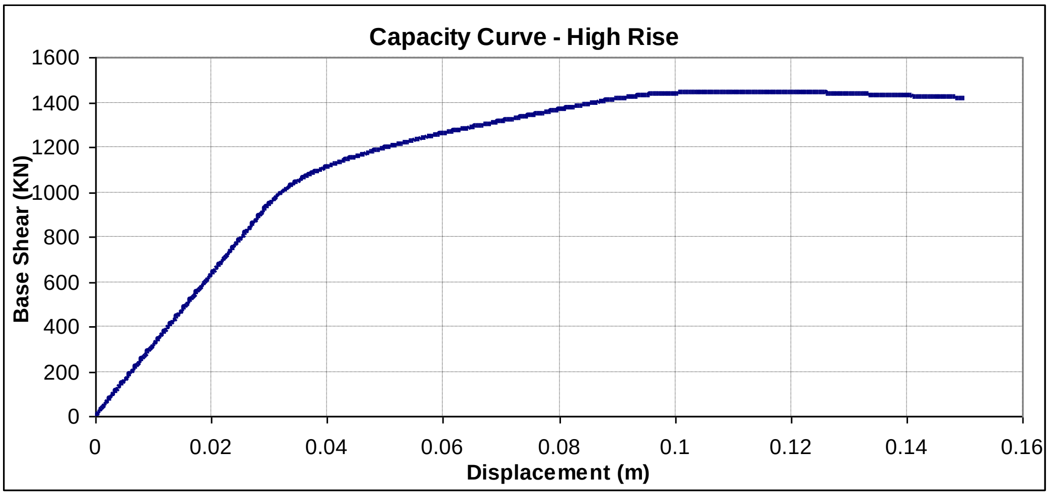


Figure 8.14. Capacity curve of retrofitted high-rise building

9. CONCLUSIONS AND RECOMMENDATIONS

9.1. Summary

In this study 3 types of buildings were analyzed by linear dynamic and nonlinear static procedures presented in FEMA 356. The initial step was to detailed assessment study of typical existing reinforced concrete buildings in Turkey. Typical buildings were classified into low-rise (4 story), mid-rise (7 story) and high-rise (10 story) designed in accordance to 1975 Turkish Code. After detailed assessment study of the existing buildings, it was observed that neither of typical buildings fulfills the conditions of Life Safety performance level in FEMA 356. It should be highlighted that the assessment study has been assumed to be made on the behavior of columns only. In other words it has been considered that existing columns in the buildings without any shear wall represent the major performance. The second step of study was to rehabilitate the existing buildings by eccentric steel braces. Different configurations were modeled and at one configuration (R2B, R2C) two systems of EBF and CBF were compared. The third step was to optimizing the best performed configuration and executing a systematical retrofit strategy. The final stage of the study was to controlling the selected configurations by nonlinear static analysis, to determine the plastic hinges and the order of occurrence, and to achieve target displacement. The buildings in the case study were evaluated for the seismic performance levels of Life Safety in design earthquake (10% probability to occur in 50 years). The effectiveness of the retrofit strategies addition of eccentric braces was determined according to the improvement of seismic capacities.

9.2. Conclusions

Executing eccentric steel braces in RC frames is of high difficulty but of high efficiency. Using eccentric steel braces in RC buildings is unpractical for most of designers due to the fact that they are not familiar with its manner of behavior and its conductivity. High accuracy and attention is required for detailing of link element connection to RC beam and braces, otherwise the expected behavior may not be appeared in EBF system. It is recommended to perform stress analysis for ensuring that link element will take story shear and work as fuse. It is also suggested to try vertical link element because detailing horizontal link on RC beam needs to demolish the existing RC beam and constructing new RC beam with high ductility and strength. Moreover, new RC beam could be detailed to work as link element, when link is located on new RC beam there may be no need for locating steel section in that part. In this study the link element has been located on new RC beam as steel plates cover it. It was observed that steel plates covered RC beam are working out and absorbing the shear. The key point of the investigation is, ending in the fact that the implemented link component can execute and operate by absorbing story shear, the remained difficulty is to transfer the story shear to link element. For having an entire EBF system, except link element all other framing component like braces, columns and beams are supposed to act elastically. The investigation shows that EBF system provides high contribution to existing RC buildings as ductility and strength by the link element in the middle of RC beam.

In R00 which symbolizes typical existing buildings designed according to 1975 Turkish Code, it is observed that low, mid and high rise structures do not perform life safety performance level in FEMA 356.

In R1A building the first configuration of EBF is tried and the result shows that EBF accumulate some contribution for the structures but not sufficient for Life Safety performance.

R2B results show that low-rise building fulfills Life Safety performance but mid-rise and high-rise do not. It could be resulted that by raising the gravity EBF system needs more stiffness and strength, however for avoiding gravity loads on the braces, sequential analysis could be of benefit. In comparison between R2B the buildings rehabilitated by EBF and R2C the buildings rehabilitated by CBF, it is observed that R2B has high ductility but mid strength and R2C has mid ductility and mid strength. In low-rise building EBF system can fulfill Life Safety performance but R2C is close to this level. Neither EBF system nor CBF can fulfill the required performance level at mid-rise and high-rise structures but R2B shows closer intention in comparable to R2C.

R3D building which is the combination of R2B configuration with steel jacketing of adjacent columns to EBF is the optimization mode for complying complete retrofit strategy for existing building. R3D results exhibits perfect Life Safety performance level and it is confirmed by nonlinear static analysis results.

Nonlinear push over analysis shows that R00 existing buildings fail and reach the mechanisms before estimated target displacement and the system shows extremely brittle behavior. From R3D capacity curves it can be seen that R3D strategy provides sensible ductility and stiffness, moreover the building does not fail before reaching target displacement and even inelastic behavior is observed at R3D because of strain hardening of steel plates as link elements. It is advised to not using steel plates with high strength because at such cases the level of ductility is reduced and desirable behavior in steel link which is ductile one is not observed.

9.3. Recommendations for Further Studies

In this study the framing system of selected typical buildings was composed of column and beam, and no shear wall was taken into account. It is recommended to locate typical reinforced concrete shear walls and investigate the combine effect of shear walls with EBF system.

The selected configurations like as R2B and R1A are the ones designed only within this study and different configurations and also different building dimensions could be taken into account. It is suggested to analysis the link element as short, mid and long lengths. An alternative discussion could be done for different kinds of links in different types of framing systems, for instance taken the length of link as e the height of story as H and the span length as L, for combination of various dimensions the behavior of EBF in RC buildings could be investigated.

The code evaluated in visual-basic can only compute for columns. In other words, the linear dynamic procedure is based on the behavior of columns only and it was assumed that the manner of columns reflects overall mode of the building against lateral forces. It is necessary to investigate beams under seismic effects in addition to columns.

In this study, the nonlinear static push over analysis was performed for end-bay 2D frames, it is recommended to do this analysis for 3D model and carry out pushover curve of overall building instead of 2D frame.

One further procedure could be done for analysis of buildings which needs sophisticated software, which is nonlinear dynamic procedure according to FEMA 356, it is recommended to use time history instead of response spectrum in code.

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