Seismic Performance Assessment and Strengthening of a Multi-Story RC Building through a Case Study of "Seaside Hotel"

Mezgeen Abdulrahman Rasol

Submitted to the
Institute of Graduate Studies and Research
in Partial Fulfillment of the Requirements for the Degree of

Master of Science
in
Civil Engineering

Eastern Mediterranean University
February 2014
Gazimağusa, North Cyprus

Approval of the Institute of Graduate Studies and Research

Prof. Dr. Elvan Yılmaz
Director

I certify that this thesis satisfies the requirements as a thesis for the degree of Master of Science in Civil Engineering.

Prof. Dr. Özgür Eren
Chair, Department of Civil Engineering

We certify that we have read this thesis and that in our opinion it is fully adequate in scope and quality as a thesis for the degree of Master of Science in Civil Engineering.

Prof. Dr. Özgür Eren
Co-supervisor

Asst. Prof. Dr. Serhan Şensoy
Supervisor

Examining Committee

1. Prof. Dr. Özgür Eren
2. Assoc. Prof. Dr. Khaled Marar
3. Asst. Prof. Dr. Erdinç Soyer
4. Asst. Prof. Dr. Giray Özay
5. Asst. Prof. Dr. Serhan Şensoy

ABSTRACT

In recent years great developments have been made in the assessment of existing buildings and their performance in resistance to earthquake loading, potential seismic risk, vulnerability and lateral loads. Existing buildings can be repaired and strengthened to include new developments and methods to resist earthquake and seismic loads, which is the most economical way to safeguard against the economic and social catastrophe affected by severe seismic activity in urban environments. Traditional buildings in the 20th century were mostly constructed without sufficient protection, considering only the gravity loads of the structure. On the other hand, steel bars in the concrete may also corrode depending on construction age and environmental factors and effect structure performance against earthquakes.

This thesis presents a case study of a RC Building constructed in 1970 namely the Seaside Hotel, a 10 storey building in Famagusta, North Cyprus. This study consists of three stages: data collection (building plans, material properties, structural condition and reinforcement details) using destructive and non-destructive tests; software modeling of the structure using SAP2000, non-linear static pushover analysis and non-linear dynamic time history analysis for seismic performance assessment based on FEMA 440 and TEC2007 codes. An appropriate method of strengthening and rehabilitation techniques with adequate cost of repairing has been also identified.

Keywords: RC Multistory building, Seismic performance assessment, Earthquake resistance, Pushover Analysis, Nonlinear time-history analysis, and Strengthening.

ÖZ

Son yıllarda mevcut binaların yapısal olarak deprem yüklerine, potansiyel deprem riskine ve dış yüklere karşı değerlendirilmesinde büyük ilerlemeler olmuştur. Mevcut binalar yeni metodlar kullanılarak deprem yüklerine karşı tamir ve güçlendirme ile ayakta tutulabilmektedirler. Yirminci yüzyılda yapılmış olan binaların çoğunda depremlerde olabilecek etkilere karşı herhangi bir koruma düşünülmemiş Sadece düşey yüklere göre tasarım yapılmıştır. Buna ilaveten donatı paslanması nedeniyle yapılarda deprem dayanımında azalma olabilmektedir.

Bu çalışmada yapılan yüksek lisans tez konusu Gazimağusa KKTC'de bulunan ve şu anda kullanılmayan Seaside Otelidir. Otel 10 katlı olup 1970 yılında yapımı tamamlanan betonarme bir yapıdır. Tez üç ana bölümden oluşmaktadır. Bunlar sırası ile tahribatlı ve tahribatsız deney metodları kullanarak veri toplama (bina planları, yapı mevcut durumu ve donatı detayları); SAP2000, statik itme analizi ve doğrusal olmayan dinamik anliz yazılımı kullanılarak modelleme ile birlikte FEMA440 ve TEC2007 yönetmelikleri kullanılarak itme ve doğrusal olmayan dinamik analiz yapılması; binanın uygun metodlar kullanılarak tamir ve güçlendirmesinin maliyet hesabinin yapılması.

Anahtar Kelimeler: B/A çok katlı bina, Sismik performans değerlendirmesi, Deprem dayanımı, Statik itme analizi, Doğrusal olmayan dinamik anliz , ve Güçlendirme.

Dedicated

To my Lovely Father and Mother

To my dearest Brothers and Sisters

For their Love, Endless Support and

Encouragement

ACKNOWLEDGMENTS

First of all, I thank God Almighty for his blessing, good health and special opportunity to obtain higher education. His guidance, wisdom and peace enhanced starting and completing this study.

I would like to express my profound gratitude to my supervisors, Asst. Prof. Dr. Serhan Şensoy and Prof. Dr. Özgür Eren for their support, guidance, and many hours of supervision required to complete this study. Thanks are due to all the members of the examining committee, technical support staff Mr. Ogün Kılıç, Mr. Orkan Lord, and all other members of the Department of Civil Engineering at Eastern Mediterranean University (EMU).

Last but not least, I would like to thank my parents, Mr. Abdulrahaman Rasol and Mrs. Qomri Khaled. My brothers and sisters and all friends, I thank you for your constant prayers and encouragement. I appreciate all of the times you visited, called, chatted and e-mailed with words of faith, encouragement and wisdom that carried me through every day of this process. Thanks to all that encouraged me to complete my study.

Finally, I would like to express my appreciation to everyone who made these years in Cyprus a wonderful experience. Thanking in advance

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF ABBREVIATIONS

Accelerograms	Strong Motion Seismograph or Earthquake Accelerometer
ACI	American Concrete Institute
ANSI	American National Standard Institute
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
ATC	Applied Technology Council
BSI	British Standard Institute
C	Collapse
CP	Collapse Prevention
CSM	Capacity Spectrum Method
DCM	Displacement Coefficient Method
DG	Damage Grade
EMU	Eastern Mediterranean University
EN	European Norms
FEMA	Federal Emergency Management Agency
GM	Ground Motion
IO	Immediate Occupancy
LS	Life Safety
MDOF	Multi Degree of Freedom
MSE	Mean Square Error
NDTHA	Nonlinear Dynamic Time History Analysis
NEHRP	National Earthquake Hazards Reduction Program
NGA	Strong Motion Database
NSP	Nonlinear Statics Procedure
NSPA	Nonlinear Statics Pushover Analysis
O	Operational
P	Probability
PBD	Performance Based-Design
PEER	Pacific Earthquake Engineering Research
PGA	Peak Ground Acceleration
PGD	Peak Ground Displacement

PGV	Peak Ground Velocity
PSHA	Probabilistic Seismic Hazard Analysis
RC	Reinforced Concrete
RSA	Response Spectrum Analysis
SDOF	Single Degree of Freedom
SEAOC	Structural Engineering Association of California
SEI	Structural Engineer Institute
SRSS	Square Root Sum of Square
TEC	Turkish Earthquake Standard Code
TH	Time History
TRNC	Turkish Republic of Northern Cyprus

LIST OF SYMBOLS

Ao	Effective Ground Acceleration Coefficient
a	Ratio of post-yield stiffness to effective elastic stiffness
C0	Modification factor which relates the maximum deformation of the SDOF system to the maximum global deformation
C1	Modification factor for estimating the maximum inelastic deformation of the SDOF from its maximum elastic deformation
C2	Modification factor accounting for effects of degrading nonlinear behavior.
C3	Modification factor accounting for geometric nonlinearity (P-Δ effects)
C_m	Effective mass factor
ε	Strain
f_c'	Concrete Compressive strength
f_y	Steel tensile strength
g	Acceleration of Gravity
I	Building Importance Factor
K_e	Effective Lateral Stiffness
K_i	Elastic Lateral Stiffness
M_1	Effective modal mass for the fundamental vibration mode
m_j	Lumped mass at the jth-floor level
N	Number of floors
P	Lateral Load

R	Ratio of elastic strength demand
R_a	Specific seismic load reduction factor
R_y	Yield strength reduction factor.
$S(T)$	Spectrum Coefficient
S_a	Response spectrum acceleration at the effective fundamental period
S_d	Response spectrum displacement at the effective fundamental period
SD	Standard Deviations
T_1	Fundamental mode period of structure
T_0	Characteristic period of the response spectrum
T_e	Effective fundamental period
T_i	Elastic fundamental period
T_s	Characteristic period of the response spectrum
V_y	Yield Strength
W	Effective seismic load
Δ	Displacement
δ_t	Target displacement
Δ_{top}	Displacement demand
λ_c	Mean Annual Frequency of Collapse
σ	Yield stress
ϕ_1	Normalized fundamental mode shape displacement of each storey
μ_D	Ductility demand
Γ	Modal participation factor

Chapter 1

INTRODUCTION

1.1 General

1.1.1 General Concept of Earthquake

Earthquake is one of the greatest natural disasters in the world, causing immense damage to human lives and property. In addition, economic losses occur due to moderate and severe ground motions. Thousands of people die due to earthquakes every year throughout the world [1]. This is a long standing phenomenon in high seismic active zones such as US, Turkey, Japan, Italy, Indonesia, China and Iran. The inhabitants of these areas must always consider their own specifications in order to design earthquake resistant structures, such as hospitals, fire stations, schools, and ordinary buildings. Maximum safety of these structures must be ensured during and after an earthquake.

The main problems of existing buildings are lateral stability and internal forces which depend on the mass of the building and ground acceleration. Buildings must be designed in such a way that ground motion acceleration for the design earthquake must be resisted by the structure at the level of its limit state. Earthquakes are uncontrollable, but the seismic force affect can be reduced by increasing the required stiffness of buildings, but this is by no means easy or economical.

Corrosion is one of the factors that directly affect building strength and which presents a challenge from the structural safety and economic perspectives (e.g.

concrete deterioration over time). Corrosion is attributable to space between reinforcement bars, concrete high permeability, climate factors and speed of corrosion [6]. Assessment of existing buildings is necessary due to the following reasons;

- To improve the building capacity in other to resist demand of seismic forces.
- To increase strength and stiffness of the structure to protect from ground acceleration.
- Using codes to let engineers consider the ductility factor for reducing earthquake forces.

Ductility is the capability of a structure to deform at an almost-constant load, crossing the elastic case and squandering the energy transmitted by the seismic waves through exhaustion and hysteresis phenomena (Figure 1).Once the elastic limit is increasing, several changes happen in material properties and characterization [36]. On other hand, the Structural Engineers Association of California (SEAOC) specified several requirement of seismic design in its recommendations [78]; structures should be able to resist the following;

- No damages during minor level earthquake.
- No structural damages, but may cause some non- structural damages during moderate level earthquake.
- Collapse prevention of structures, but may possibly cause some structures as well as non-structural damages during major level earthquake.

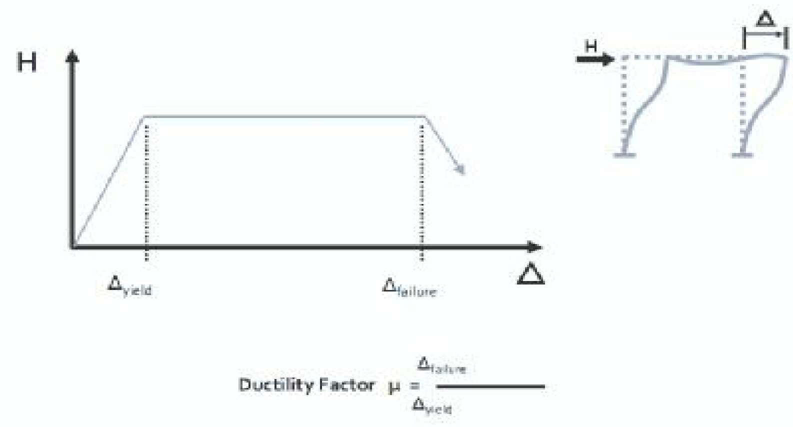


Figure 1. Force deflection curve of structure [36].

1.1.2 Importance of Seismic Performance Assessment

In recent years, performance based-seismic design has become an important tool for earthquake resistant design. In addition, performance and assessment are required for existing buildings whereby only gravity load was considered during design, neglecting lateral and seismic load as a whole. Performance assessment of existing buildings should be carried out by considering several steps. First, data from the existing building such as (concrete properties, element connection, and corrosion problem) should be collected and its present condition should be determined. Performance level is commonly identified by considering four levels [5]:

- Operational
- Immediate occupancy
- Life-safety
- Near collapse

Basically four analysis methods are available for performance and assessment of the existing buildings:-

- Linear static.
- Linear dynamic (Response spectrum or Time-history).
- Non-linear static (Pushover analysis)

- Non-linear dynamic (Modal pushover/Incremental Response spectrum or nonlinear time-history).

Pushover analysis is a method of static non-linear analysis procedure and used for performance-based design and seismic performance assessment of existing buildings [3]. In this study, the non-linear static pushover analysis and non-linear dynamic time history analysis are used for seismic performance assessment of building, identified as FEMA440.

1.1.3 Importance of Strengthening Technique

When all the required properties of the building and its level of damage are determined, the performance assessment analysis can be undertaken. Subsequently, the suitable strengthening technique method must be chosen for a particular building and repair methods can be chosen contemporaneously with the rehabilitation of the structure. Earthquake and seismic specifications must be considered as part of this process. Strengthening is a process of increasing capacity of the member of structure and its behaviour, such as enlargement of members, member jacketing and adding shear wall. Emmons [7] mentioned several primary performance measures based on the structure that should be considered as part of a detailed plan to strengthen existing buildings;

- Increasing protection of reinforced embedded materials.
- Aesthetically plastering.
- Not loose from substrates.
- Carry a portion of compressive load.
- Protecting reinforcement bars from corrosion.

1.2 The Objective and Scopes

The purpose of this study is to obtain the following requirements;

1. To assess the performance of a traditionally constructed building using new developments and considering structural specifications.
2. To use strengthening techniques of rehabilitation and repairing of the building.
3. To improve the structure's capability, strength and stiffness in order to limit the loss due to earthquake.
4. To reduce human and economic loss.
5. Finally, to discuss engineering judgment, suggestions and recommendations.

1.3 Thesis Organization and Outline

This thesis consists of six chapters. These describe all the respective steps and the plans according to the following outline;

Chapter 1 – Introduction: In this chapter, we described a brief introduction of seismic performance assessment and strengthening of existing building.

Chapter 2 – Literature Review: This chapter presents a literature review for seismic performance analysis and strengthening technique of other works related to this research.

Chapter 3 – Methodology and Data Collection: Methodology and data collection concerning study of the existing building.

Chapter 4 – Modeling and Assessment: In this chapter the modeling of the 2-D frames from two direction of the building is presented. Nonlinear static analysis and nonlinear time history analysis are performed to predict performance level of the existing building.

Chapter 5 – Strengthening and Evaluation: This chapter presents the analytical results of strengthening the case study building.

Chapter 6 - Conclusions and Recommendations: This chapter summarizes the findings of this research, presents its conclusions and makes recommendations for safeguarding buildings against seismic activity and for further academic research.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

Significant investment is underway worldwide for the assessment and strengthening of existing buildings. Earthquake activities result in various types of ground motion as seismic waves, which pass under the structure and subject it primarily to lateral forces (and vertical forces to a lesser degree). In this case, the structure should be able to resist vertical and lateral forces without losing strength and stiffness. It needs to resist deformations without developing high stress concentration [8].

Tectonic movement that causes earthquakes is characterized by irregularity of motion, and these movement frequencies impact on the capacity of the soil base, substructure and superstructure. Motion during seismic events happens through the base of the structure, resulting in dynamic loads. These loads affect the structure elements. Hence, the structure must have sufficient stiffness, stability and strength to transfer these seismic loads through the base of the structure and soil safely. The floor system passes sufficient stiffness and strength to resist the safe transfer of total seismic loads between structure's system and elements [42].

Most of the time there are other important problems in existing buildings such as corrosion of reinforcement bars, age of building, environmental affects, and the projection in plan among others.

2.2 Factors Affecting Seismic Performance Assesment

Many structures have been affected by earthquake occurrences, and the most damaged buildings as a result of these events are those which were constructed without due consideration of earthquake codes. The majority of earthquake-related structural damages occur due to [9]:-

- Design and construction material problems.
- Insufficient or inaccurate reinforcement details.
- Non-earthquake conformity, projection in planning and bearing systems.
- Construction errors.
- No consideration given to earthquake specification code of practice.
- Geotechnical or soil conditions and economy effects.

The deficiencies mentioned above are principally the reasons for the damage done to buildings during an earthquake. Each will be discussed and elaborated up in turn.

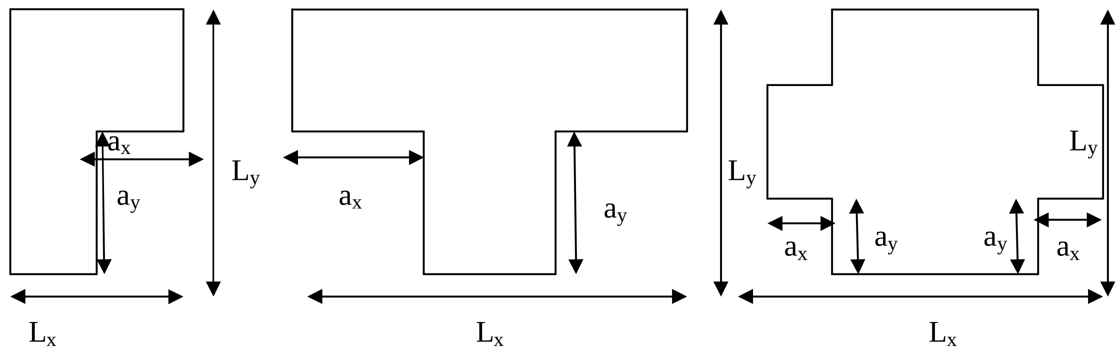
2.2.1 Irregular System of Buildings

The irregularity of buildings can be defined as the absence in the design and construction of the building in the event of undesirable seismic behavior. There are several conditions for the irregularity buildings. They are;

2.2.1.1 Irregularity in Plan

Irregular architectural plans of buildings produce irregularity in building systems. Buildings without symmetry in projection of upper floors have impaired earthquake resistance behavior. There are two irregularities that have a main role in resisting earthquake loads.

The first one is torsion irregularity causes increasing shear forces in shear walls, columns, corners. Second one is floor discontinuities which are near the circumference of building floor discontinuous in each floor. Typical irregularity in plan is shown in (Figure 2).



Type A3 irregularity;

$a_x > 0.2\ L_x$ and at the same time $a_y > 0.2\ L_y$

Figure 2. Irregularity of Projection in Plan [42]

2.2.1.2 Irregularity in Elevation

Irregularity in elevation is one of the major damages that exacerbate the risk posed to the building when earthquakes strike. This type of irregularity affects the first storey then it will be transferred to the other building floors.

The presence of this condition in the first storey of the frame structure system (the “ground floor”) can be affected by the zone being free of walls (open floor plan), having stiff non-structural walls in the upper levels, or shear walls being available in the upper storey, which do not extend to the foundation but which are disconnected at the second floor level [10]. Traditional open floors have some disadvantages: insolubility, inefficiency, and waste, compared to open floor modern design but also has some of it is advantages including economy, hygiene, and pedestrian circulation separation from vehicular traffic [10].

Soft and weak storey irregularity are very common causes of damage in buildings and it is one of the most popular forms of architectural design because of the modern style of architectural configuration being based on five points [11]:-

- Pilotis (open first floor).
- The free floor.
- The free façade.
- Strip windows.
- Roof terrace, roof gardens.

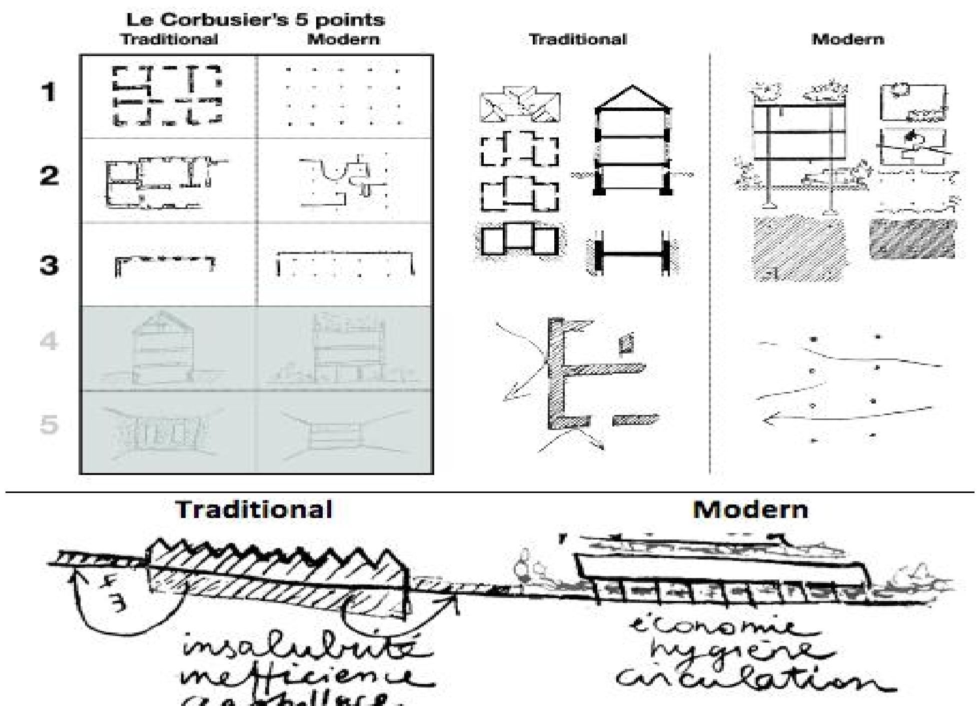


Figure 3. Le Corbusier's five points for modern Architectural design [11]

According to ASCE 7-05[12], a stiffness-related soft storey is one in which the lateral stiffness less than 70% of the storey above or less than 80% of the average stiffness of three storeys above. A stiffness-extreme that has target values of the conditions mentioned above or responds to 60% or 70%, numerous failures and possible collapses can be attributed to the increased deformation demands caused by soft storey (Figure 4).

As mentioned above, according to the Turkish Earthquake Standard [42], in reinforced concrete buildings, the case where in each of the orthogonal earthquake direction, stiffness irregularity factor is , $\boldsymbol{\eta_{ci}}$, which is defined as the ratio of the effective shear area of any storey to the effective shear area of the storey immediately above, is less than 0.80. This relation is shown below by Equation 2.1;

$$\eta_{ci} = (\Sigma A_e)_i \,/\, (\Sigma A_e)_{i+1} < 0.80 \qquad (2.1)$$

On the other hand, soft storey is the case where in each of the two orthogonal earthquake directions, stiffness irregularity factor, $\boldsymbol{\eta_{ki}}$, which is defined as the ratio of the average storey drift at any storey to the average storey drift of the storey immediately above, is greater than 2. This relation is shown below by Equation 2.2;

$$\eta_{ki} = (\Delta_i/h_i)_{ave} / (\Delta_{i-1}/h_{i-1})_{ave} > 2.0 \qquad (2.2)$$

"Soft Storey"
Figure 4. Soft storey.

2.2.1.3 Short Column

Columns that have a shorter height compared to the normal designed column height in the same storey are called “short columns” [13]. The short columns are stiffer than long columns and magnetized larger earthquake forces. The stiffness of the column means its resistance to deformation and seismic demand. If it is not designed to accommodate large force demand, it can be a damaged during earthquake. Damages of the short column are in the form of X-Shape cracking shear failure [14].

The formation of short columns due to the presence of an intermediate beam, difference in ground level “sloped ground”, and partial infill walls can during earthquakes incur damages of much greater magnitude than long columns, causing failure or collapse of the side of the structure containing the foundations of the short columns (Figure 5).

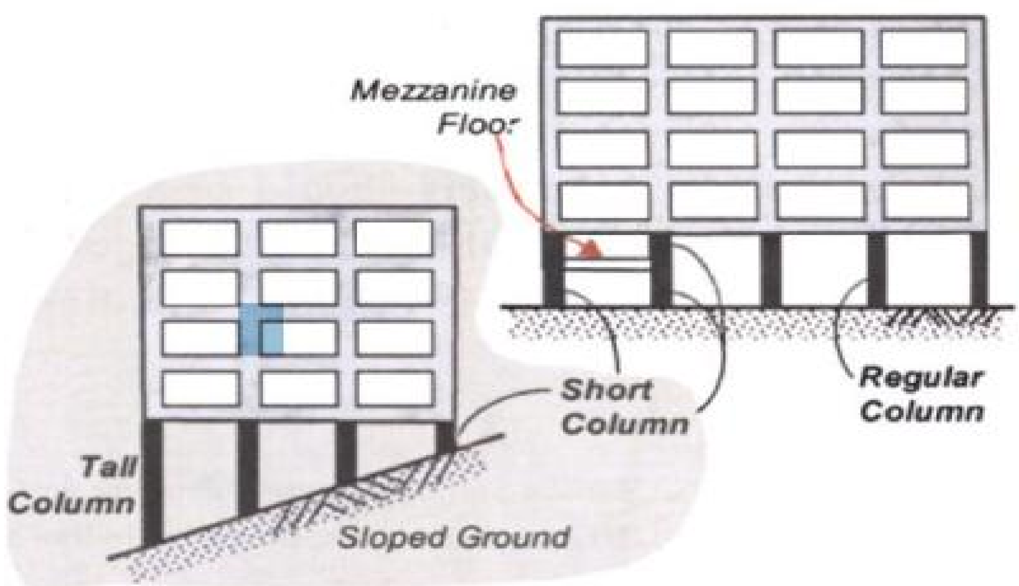


Figure 5. Difference of ground level of formation short column.

2.2.2 Reinforcement Details

Reinforcement details must be endorsed for intermediate moment resisting frames [AS 3600, 1994]; any lack of reinforcement always causes structural damage or collapse. It is briefly described as follows;

Beam Details; The bottom area of the beam at the support face should be able to resist one-third of the hogging moment design when the effected in an inverted direction due to earthquake loads. If the reinforcement of the anchorage length of beam is shorter relative than the development length it may carry tensile stresses and connection is not compacted very well at a junction point (e.g. beam-column junction). To improve this section of structural design you need to add additional bars in the top of the beam, as shown in (Figure 6) and sheet detail no. 1 [17].

Column Details; During earthquake strikes, plastic hinges occur at the end of column and top of the beam at the place that connects them (the beam-column junction). In that location distinguished reversal moment due to the cyclic loading occurs because of high strains by reinforcement. To make it safe during earthquake events columns must be improved by increasing additional shear links and longitudinal reinforcement to get enough durability, structural safety due to energy absorption, as shown in (Figures 7) and sheet detail no.2 [18].

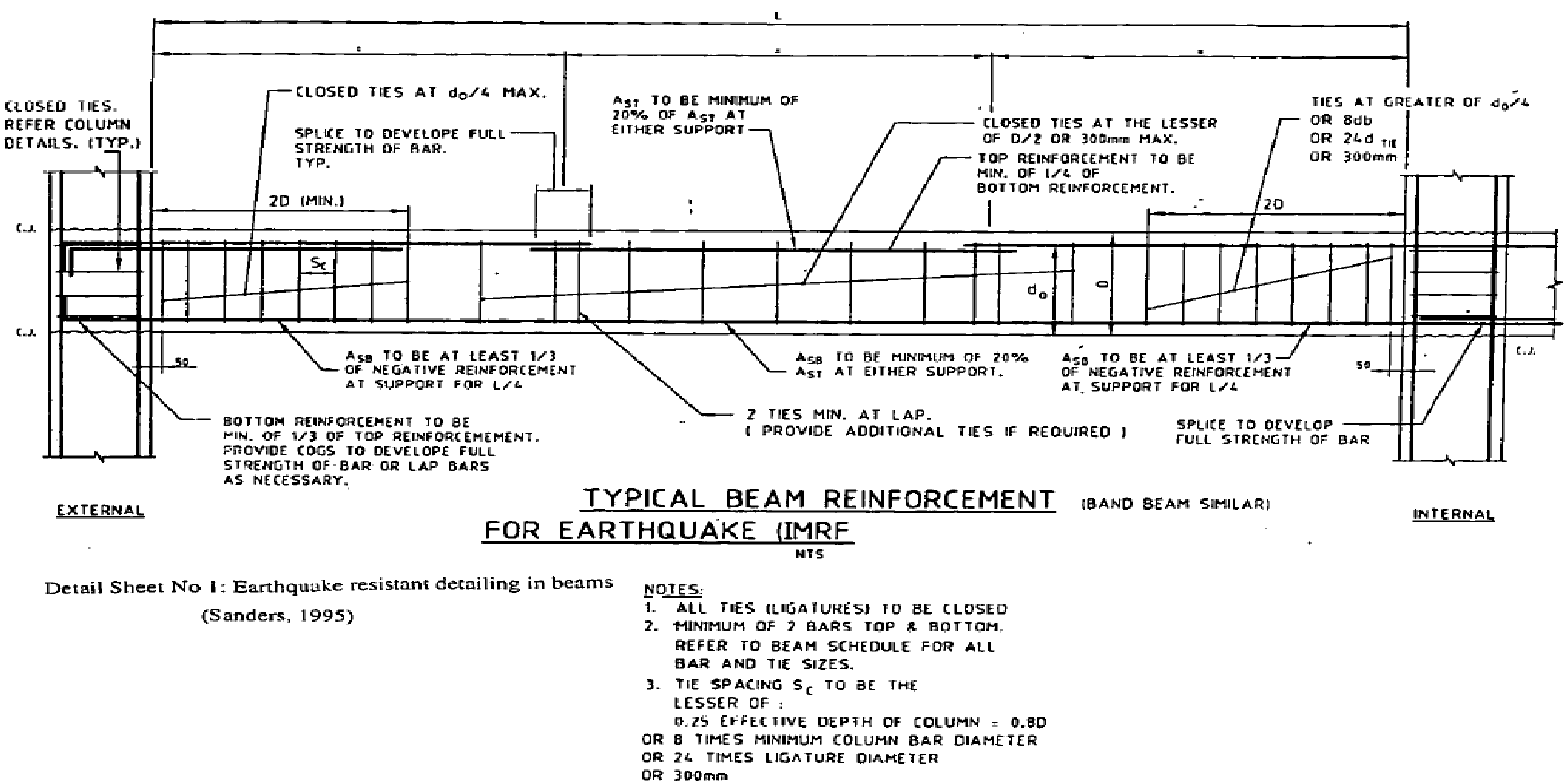
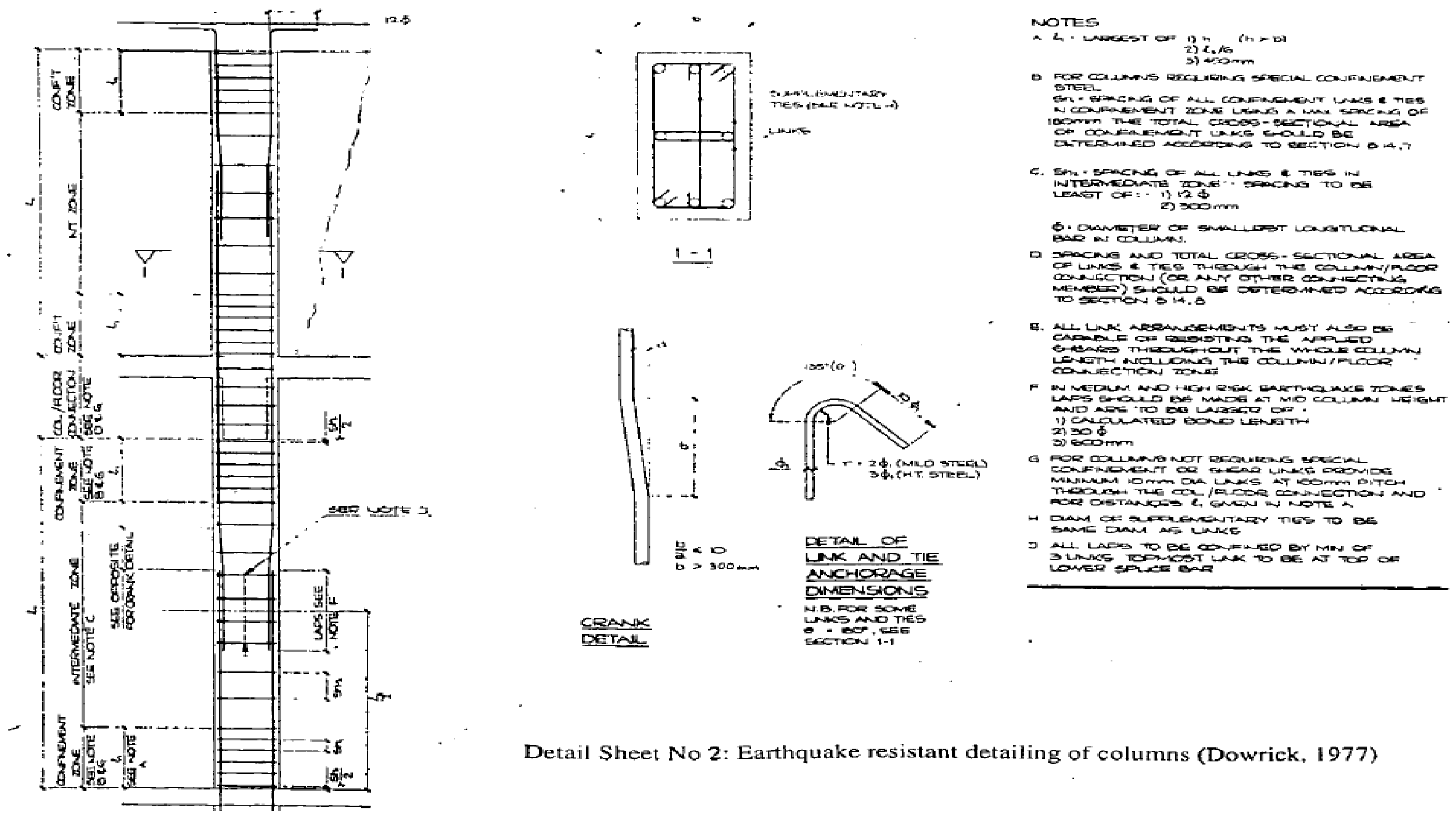


Figure 6. Typical Beam Reinforcement for Earthquake resistant [17]



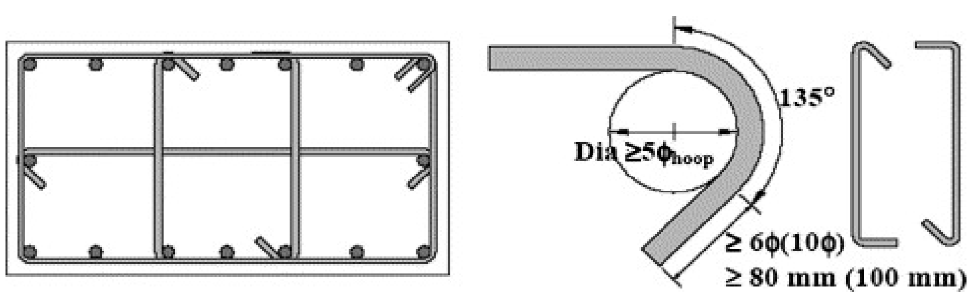
Detail Sheet No 2: Earthquake resistant detailing of columns (Dowrick, 1977)

Figure 7. Typical Column Reinforcement for Earthquake resistance [18].

Figure 8. Beam-column junction damages.

To improve the absorption of energy to maintain structural safety during severe earthquake depends on ductility and is concerned with [16];

- Overlapping length joint 50% at storey level.
- Reinforcing bend or hooks and cross ties at the edge of the elements (special hook and special cross ties) used in columns, wall end zones, beam confinement, beam – column joints, single and two pieces of hoops, as shown in (Figure 9).
- Earthquake resistant stirrups used in columns and beams.



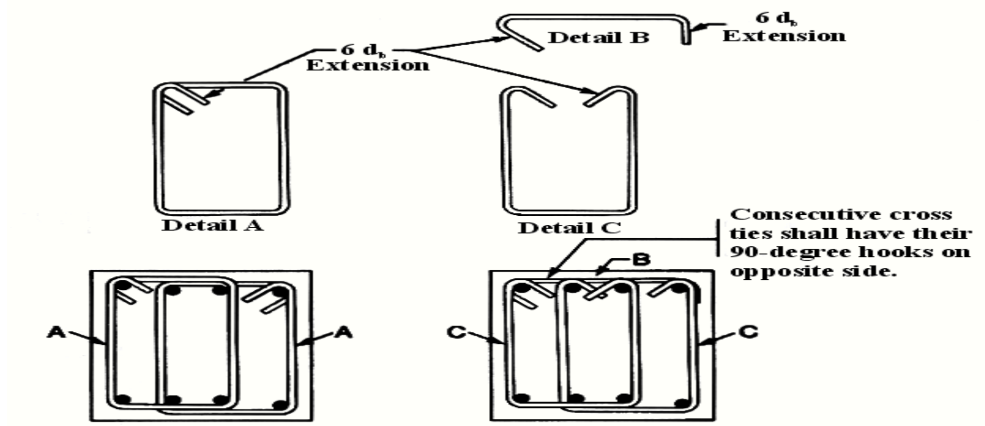


Figure 9. Earthquake resistant hooks or bends and cross ties [33].

2.2.3 Corrosion Effect

Corrosion means degradation of a metal by an electrochemical reaction with its environment [77]. It has surrounding reinforcement bars until the critical corrosion rate at a specific time then bars cause gap due to corrosion effect. Economic impacts and structural safety concerning the strength and stiffness of RC building are issues arising from corrosion effect. When an earthquake strikes, corrosion causes the reduction of concrete compressive strength, resulting in cracks and less ductility of structural elements, as shown in (Figure 10) [21].


Figure 10.Reinforcement Corrosion of Column (Seaside Building).

2.2.4 Geotechnical Conditions.

The influence of local soil conditions is one of the most dangerous factors effecting the foundation types and whole structure system but it has very less controllable aspects of the real state of soil conditions, depending on seismic wave propagation, epicenter, ground acceleration, amplitude, frequency content, duration, stiffness characteristics. Many types of ground failures directly affect the structures, such as liquefaction, lateral spreading and landslides. Some mitigation of soil is needed before strengthening the building, damage as shown in (Figure 11) [20].

Figure 11.Structural damage due to liquefaction (Mexico City, Sep 19, 1985).

2.2.5 Economy and Cost Conditions

Economic resources must be available for appropriate performance and strengthening enhancement. Sometimes the cost of repair and maintenance of the structure is more than the cost of constructing a new building. In such circumstances it is likely that repair will be neglected, but the following simple balancing equation must be considered before engineering judgment [20];

Cost of strengthening and repair < Cost of constructing a new building

2.3 Sesimic Performance Assesment of Existing Building

The main purpose of seismic performance assessment of an existing building is to evaluate and decide upon a suitable strengthening method that protects the building during an earthquake strike. Strengthening techniques need good quality performance analysis to repair the existing building. This section discusses various approaches for performance analysis of buildings in general. Earthquake hazard calculation provides an implementation statement of the motion of the ground on which a building is located in a specific geographical region. Determination of the effects formed by this motion on the building and the amount of probable damage caused requires the implementation of a separate assessment [Yakut, 2008].

Many countries have special seismic performance and strengthening standard codes which they always use during analysis calculation methodologies as a source of specification related to the building location, including FEMA 356, FEMA440, ATC-40, TEC 2007 and Euro Code 8. Their analysis approaches and strengthening techniques are essentially similar. Analysis is performed in order to calculate internal forces and deformations, and to determine structural systems design, assessment and capacities. Sizing or capacity control is done depending on these determined forces and deformations. Two main categories of seismic analysis are used: Linear Analysis System (including Static Analysis/Linear Performance Analysis and Dynamic Analysis) and Non-Linear Analysis System (Static Analysis/Pushover Analysis, Dynamic Analysis/Time-History Analysis).

The most employed methods are linear statics and dynamics methodologies, such as equivalent static load methodology, mode superposition using response spectrum and time history analysis, non-linear pushover analysis and non-linear time history analysis [9]. Some software can determine the behavior of plastic hinges by pushover analysis, such as (Sap2000 and IDE-CAD 5.511), as proposed in many codes.

2.3.1 Seismic Performance Assessment Methods

Seismic performance assessment analysis methods are explained as follows;

2.3.1.1 Linear Static Analysis

Linear static analysis method is widely used for force-based assessment methodology. Equivalent static lateral force analysis can be utilized for performance assessment. Equivalent lateral force analysis is limited to eight-storey buildings with a total height not exceeding 25 m, and it has no tension irregularity. 85% of total mass in base shear force calculation is considered in buildings possessing more than two storeys. The nodes of internal member forces and capacities under an earthquake provocation direction were taken as the signs consistent with the dominant mode shape in this direction [55].

Linear static analysis is used for existing buildings. The code assumes a specific seismic load reduction factor $\mathbf{R_a}$ by requiring provisions for gaining a structural system of high ductility. In existing buildings the demand and capacity ratio of cross sections are evaluated and compared with their own limit value in code specification. The main reason is variation in member ductility for this point we may take ductility level in the existing building. Mostly, three knowledge levels have been obtained and a coefficient is suggested for each knowledge level (Table 1). The determined knowledge level influences the determination of the calculation methodology to be applied.

Table 1. Knowledge level coefficient.

Information Level	FEMA 356	TEC 2007
Limited	0.75	0.75
Moderate	0.75 or 1.00	0.90
Comprehensive	1.00	1.00

- **Limited Knowledge level**

In this knowledge level, several data should be collected such as structural plans by field studies, members and walls location. Foundation system is identified by excavating inspection, topographical information and reinforcement details with visually inspected 10% of columns and 5% of beams in each storey by removing concrete cover [42].

- **Moderate knowledge level**

Basically, this is the same as the limited knowledge level, while reinforcement is visually inspected for 20% of columns and 10% of beams in each story by removing concrete cover. Furthermore, a minimum of three concrete core samples are taken from the columns and beams, where the minimum total number is nine [42].

- **Comprehensive knowledge level.**

Comprehensive knowledge should include the details identified such as structural plans, foundation system identified by excavating inspection, topographical information, material properties, reinforcement details and existing steel strength should be verified by taking three specimens from the building which obtains the stress-strain behavior of reinforcement steel bars [42].

Structural model should prepared in detail to determine earthquake load effect on the structure, Internal and deformation forces for each element of the structure are obtained by using load combination specified for the above calculation of spectral acceleration coefficient used, as shown in (Figures 12 and 13) the directions of earthquake loading effect on actual mass center and shifted mass center according to section 2.4 in Turkish earthquake regulation.

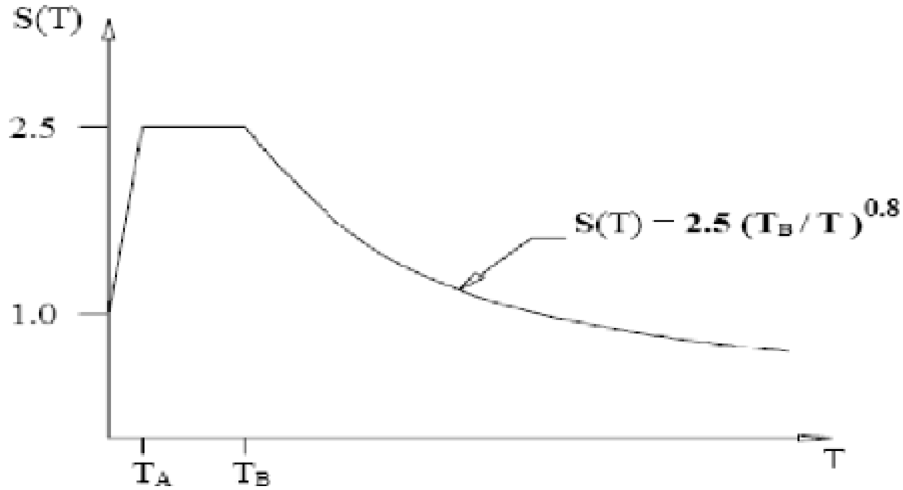


Figure 12. Elastic acceleration spectrums [42].

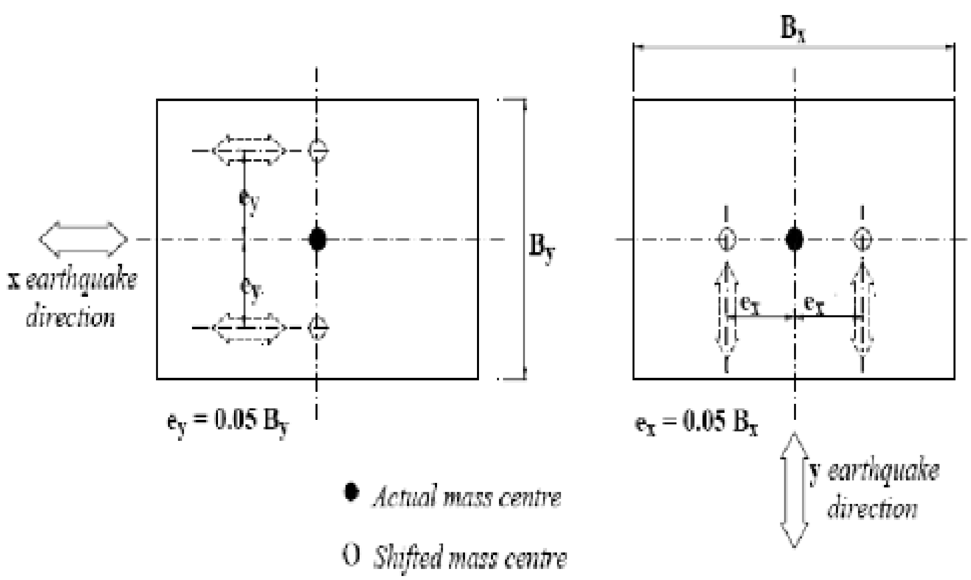


Figure 13. X and Y direction earthquake loading [42].

2.3.1.2 Non-Linear Static Analysis

2.3.1.2.1 Non-Linear Static Pushover Analysis

The NSPA method is the most commonly used in the seismic performance and the assessment of existing buildings and is aimed to estimate seismic demand. In the procedure (e.g FEMA-273/356, FEMA440, ATC-40), the seismic demand determined by NSPA represents the inertial forces experienced by the structure when subjected to ground shaking in a monotonically increasing pattern of lateral loads with a fixed height-wise distribution until a predetermined target displacement is reached [69].

According to this methodology, a sample relation of systems base shearing force-top point displacement is as shown in (Figure 14). This curve represents the building's behavior of the structure under the increasing base shearing force. In other words, vertical axis of the curve reflects different earthquake effects while the horizontal axis shows the deformations corresponding to these effects. Consequently, the curve remains straight under low earthquake effects and represents the structure's elastic behavior [58].

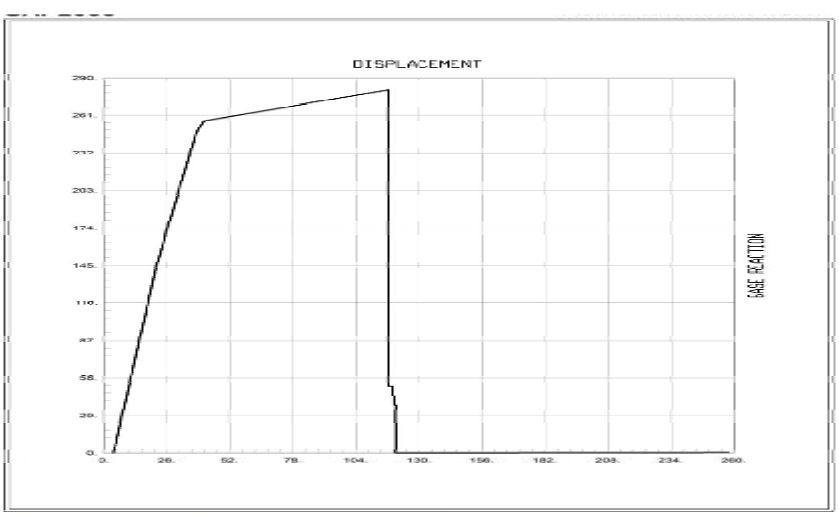


Figure 14. Pushover analysis curve.

Moreover, under the increasing earthquake effects, load deformation relation changes because same elements will reach their elastic capacity and yield. Thus, the structure will go beyond the elastic limit. The last point of the curve coincides to the status of the structure just before the collapse. This curve is actually a curve representing the capacity of the structure rather than the effect of certain earthquake, so to obtain accrued result of analysis, including the use of constant lateral force profiles (uniform or triangle), adaptive multi-modal approaches are needed [57]. Non-Linear Pushover static analysis consists of the following steps;

1. Configured structural model established by computer.
2. The element loads are defined.
3. Then vertical loads are applied on the structure by being them in compliance with the effect of an earthquake.
4. Linear analysis is then carried out by applying incremental horizontal loads on the structure.
5. Finally, the calculated base shear force, top point displacement, element's internal forces and nodal point displacements are recorded

In chapter 4, Non-Linear pushover static analysis is discussed in detail.

2.3.1.2.1.1 Idealizing force-displacement curve

To calculate the effective lateral stiffness K_e and effective yield strength V_y, the obtained force-displacement relationship curve from analysis shall be idealized to a bilinear curve. The ideal line should be constructed in a way that areas above and below the curve remain balanced [61].

In FEMA440 where K_i is the elastic lateral stiffness of the building, K_e is the effective lateral stiffness of the building, α is the yield slop, V_y is the effective yield strength and target displacement is shown as δt the performance point of the structure shall be determined and checked on the cap city curve to find performance level of building. (Figure 15) shows a typical idealized bilinear force-displacement curve.

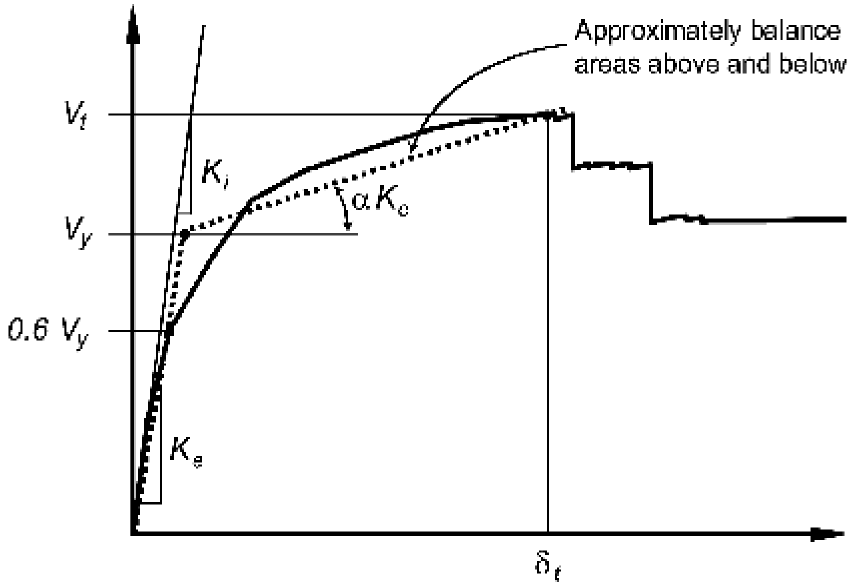


Figure 15. Idealized Force-Displacement curve [61].

2.3.1.2.1.2 Structural Performance limit states

Building's earthquake performance level is determined after evaluating the damage states, the rules for determining building performance are given below for each performance level.

- **Immediate occupancy (IO)**

After the ground motion of the earthquake the structure remains sufficiently safe to be occupied, slight damage to the structure that can be easily repaired is observed, with 10% significant damage to elements.

- **Life Safety (LS)**

After the earthquake loading where the structure resists major damage but has large cracks or falling concrete cover or waste material from members there is risk of injury to life, structural damage can be repaired but might be less economical when compared to complete reconstruction.

- **Collapse Prevention (CP)**

After the occurrence of the earthquake that has brought about complete or partial collapse to one part of the structure, where a large displacement of structure is observed and frames have lost preloading strength and stiffness but still need to carry self-weight, there is the possibility of injury to life. Repair is not seen a good solution for this level because it is accepted that the smallest shock will bring about a complete collapse. In CSI SAP2000 the plastic hinges behavior can be observed and each step changes as shown in (Figure 16).

- **Collapse (C)**

If the building fails to satisfy any criteria of the above levels it means that the building is at the collapse level, and occupancy of the building should not accepted.

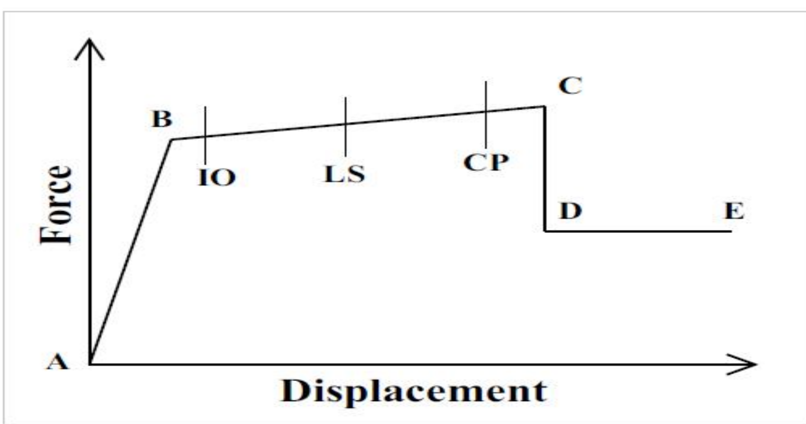


Figure 16. Acceptance criteria of performance level.

2.3.1.2.1.3 Damage Prediction

Lang (2002) devised a method obtain the damage grades of structures and evaluate existing structures’ performance. Lang studied the vulnerability of existing structures in Basel (Switzerland), and proposed a simple method to evaluate reinforced concrete buildings based on their engineering models (Lang, 2002), since there was no major destructive earthquake records available at the time. Cyprus has the same situation regarding historical records of destructive earthquakes, although many occurred during its history. In order to observe the behavior of buildings selected in this study, Lang’s procedure was chosen as well as FEMA to be to compare results and better investigation and evaluation of buildings. The procedure is considering the pushover curve of structure and tries to construct the vulnerability function from displacement demand of structure and spectral displacement. In response to lateral loading presented earthquake ground acceleration. Equation below shall be used to calculate the top displacement of structure;

$$\Delta_{top} = \frac{\Gamma \cdot m_D \cdot T_1^2 \cdot S_a}{R_y \cdot 4p^2} \tag{2.3}$$

Where Γ is the modal participation factor, μ_D presents ductility demand, T_1 is the fundamental mode period of structure, S_a is spectral acceleration, and R_y presents yield strength reduction factor.

$$\Gamma = \frac{\sum_{j=1}^{n} m_j \, . \, \Phi_j^1}{\sum_{j=1}^{n} m_j \, . \, \left(\Phi_j^1\right)^2} \tag{2.4}$$

Where m is represents the mass of each storey of the structure and ϕ_1 is the normalized fundamental mode shape displacement of each storey. Calculating the fundamental frequency of the structure will find the spectral displacement.

$$w = \frac{2p}{T} \quad (2.5)$$

$$\Delta \mathbf{0} = m_D \,.\, \Delta_y \quad (2.6)$$

$$\Delta_o = \Gamma \,.\, S_d\left(T_1\right) \quad (2.7)$$

$$S_d = S_a / \left(2\pi/T \right)^2 \quad (2.8)$$

$$V_o = K_E \,.\, \Delta_o \quad (2.9)$$

$$R_y = \frac{V_o}{V_y} \quad (2.10)$$

Where, V_a is the required base shear by the system to remain elastic, V_y is the force at yield (for bi-liner systems) or the shear capacity of the building (for elastic-plastic systems), Δ_y is the top displacement at the first yield (for bi-liner systems), Δ_o is the required top displacement by the system to remain elastic shown in (Figure 17). Top displacement is shown on a general vulnerability function of an RC building with specified damage grades in (Figure 18). Observing the change in damage stages of building allows engineer to interpret the capacity of structure to resist damage grades with corresponding shear force causing it, hence can estimate the loss of building in case of earthquake. Classification of damage grades proposed by Lang (2002) is shown in Table 2.

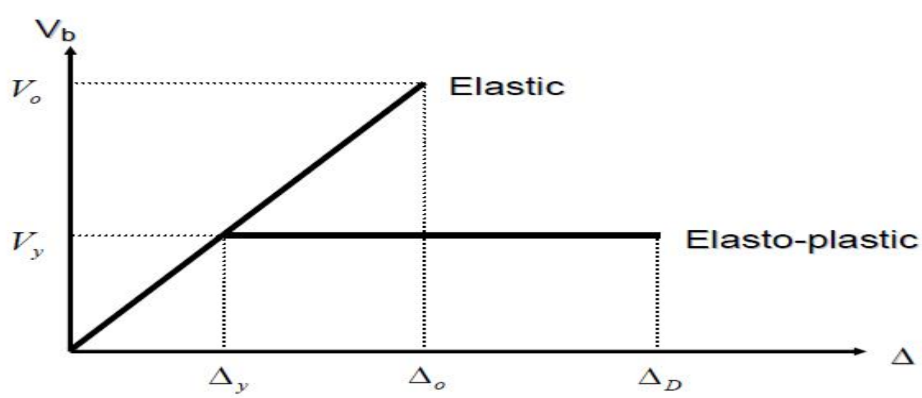


Figure 17. Force-displacement for linear elastic and elastic-plastic behavior.

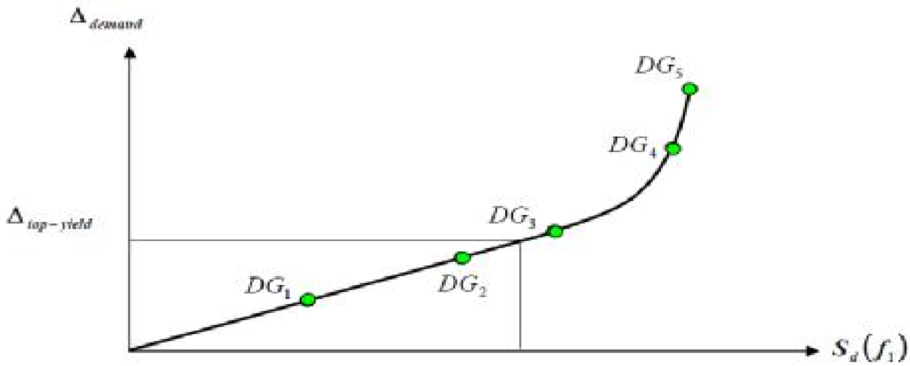
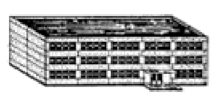
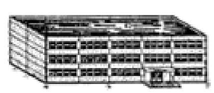
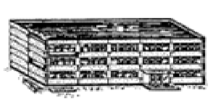
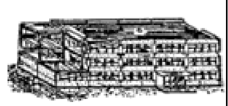
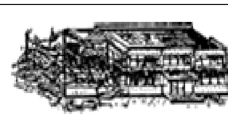


Figure 18. Typical Vulnerability function for R/C building (Lang, 2000).

Table 2. Classification of damage to reinforce concrete buildings[42].

Damage Grade	**Simulation**	**EMS 98**	**Identification (Start of Damage Grade)**
Grade 1		***Negligible to slight damage (no structural damage, slight non-structural damage)*** Fine cracks in plaster over frame elements or in walls at base. Fine cracks in partitions and infills.	After cracking. Onset of tensile strength of members.
Grade 2		***Moderate damage (slight structural damage, moderate non-structural damage)*** Cracks in columns and beams of frames and in structural walls. Cracks in partition and infill walls; fall of brittle cladding and plaster. Falling mortar from the joints of wall panels.	First plastic section. Reduction starts in structural stiffness.
Grade 3		***Substantial to heavy damage (moderate structural damage, heavy non-structural damage)*** Cracks in columns and beam column joints of frames at the base and at joints of coupled walls. Spalling of concrete cover, buckling of reinforced rods. Large cracks in partition and infill walls, failure of wall-panels.	Final plastic section before individual section failure. Building stiffness tends to zero.
Grade 4		***Very heavy damage (heavy structural damage, very heavy non-structural damage)*** Large cracks in structural elements with compression failure of concrete and fracture of rebars; bond failure of beam-reinforced bars; tilting of columns. Collapse of a few columns or a single upper floor.	First individual section failure. Start of reduction in base shear.
Grade 5		***Destruction (very heavy structural damage)*** Collapse of ground floor or parts of the building.	Final individual section failure. Loss of lateral stability. Buckling of some columns.

2.3.1.2.2 Displacement Coefficient Analysis Method

Displacement coefficient analysis method (DCM) was introduced in 1996 by FEMA (Federal Emergency Management Agency) to evaluate the seismic vulnerability of an existing structure, which needs to be rehabilitated due to lack of deformation capacity for a given level of ground motion demand [57].

DCM is an approximate static nonlinear analysis, the maximum inelastic deformation of the equivalent single –degree-of-freedom (SDOF) system is estimated from its elastic deformation by using statically derived modification factors. In the case of pushover curve applied to;

- Clarify the lateral load resistance capacity of the building, and
- Represent the (MDOF) system as an equivalent of SDOF.

Generally, in FEMA-273/356 at first a control node must be selected then at least two lateral load distributions are considered, the relation between base shear and control node will be created, and the nonlinear relation will be idealized to a bilinear relation to get the effective lateral stiffness K_e and effective yield strength V_y of the structure, as described in (Figure 19).

The modification factor depends on the effect of a) lateral strength, b) hysteretic behavior and c) geometric nonlinearity (P-Δ) of the structure. The DCM method is a simple, straight and non-iterative procedure to be used since lateral strength capacity of an existing building is known [57].

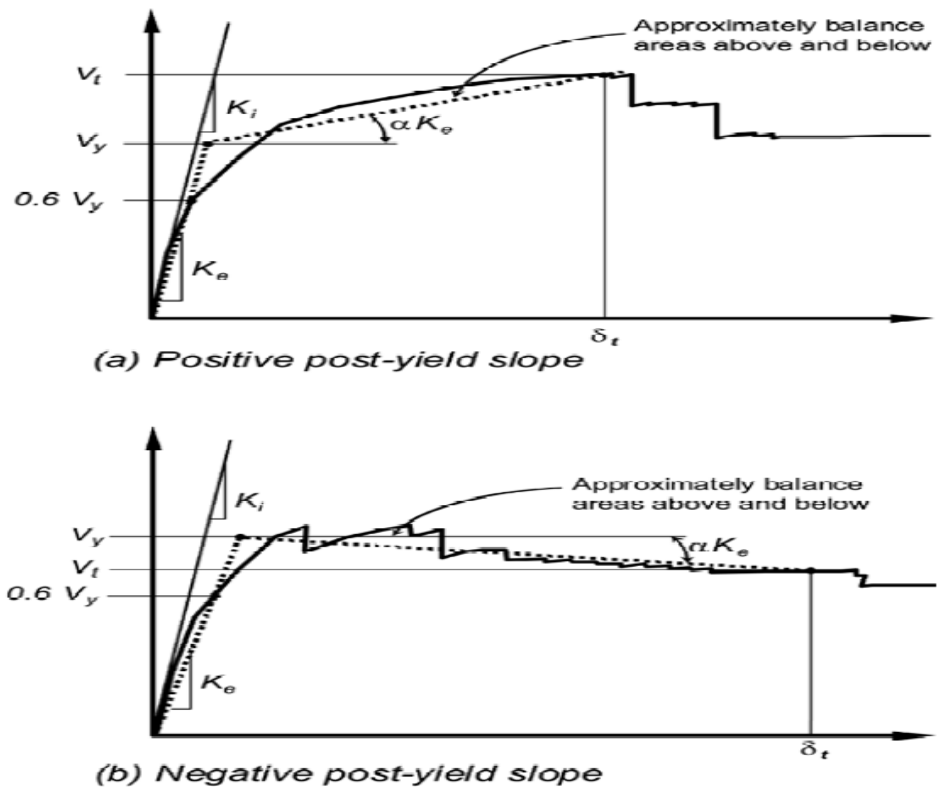


Figure 19. Idealized force-displacement curves [44].

This procedure is proposed to calculate a performance point by using elastic spectrum with capacity curve. Building's top point displacement (**δ_t**) corresponding to performance point is calculated using the relation given below.

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{Te^2}{4\pi^2} \tag{2.11}$$

Here,

C_o = the coefficient correlating the displacement calculated for equivalent single degree system with structure's top point displacement. This coefficient can be obtained using the result of modal analysis and the values taking place used depend on number of floors of the building (Table 3).

Table 3. Values for modification factor Co[1]

	Shear Buildings[2]		Other building
Number of stories	Triangular load pattern[3]	Uniform load pattern[3]	Any load pattern
1	1.0	1.0	1.0
2	1.2	1.15	1.2
3	1.2	1.2	1.3
5	1/3	1.2	1.4
+10	1.3	1.2	1.5

[1] Linear interpolation shall be used to calculate intermediate values.

[2] Buildings in which for all stories, inter story drift decreases with increasing height.

[3] Possible load patterns are defined in section 2.3.1.3 triangular load pattern can be each of patterns defined in first category and uniform load pattern is part (a) of second category.

C_1= Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

$$C_1 = \begin{cases} 1 & Te \geq Ts \\ 1 + \dfrac{(R-1)Te}{Ts} & Te < Ts \\ 1.5 & Te < 1.0\ Sec \end{cases} \qquad (2.12)$$

T_e= Effective fundamental period of the building in the direction under consideration, in seconds.

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.

R= Ratio of elastic strength demand to calculated yield strength coefficient which in calculated by the following Equation 2.13.

$$R=[Sa/(vy/W)]\ \ Cm \qquad (2.13)$$

Where,

Vy = Yield strength of structure obtained from capacity curve,

W= weight of structure,

Cm= accounts for effective modal mass factor for fundamental mode of structure.

C_2= Modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response. Alternatively, use of $C_2 = 1.0$ shall be permitted for nonlinear procedures.

$$C_3 = 1.0 + \frac{a\,(R-1)^{3/2}}{T_e} \tag{2.14}$$

Table 4. Values for Modification factor C_2

	T<0.1	**T<0.1**	**T>Ts**	**T>Ts**
Structural performance level	**Framing Type 1**	**Framing Type 2**	**Framing Type 1**	**Framing Type 2**
Immediate occupancy	1.0	1.0	1.0	1.0
Life safety	1.3	1.0	1.1	1.0
Collapse prevention	1.5	1.0	1.2	1.0

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

g = acceleration of gravity.

a = ratio of post-yield stiffness to effective elastic stiffness, where the nonlinear force displacement relation shall be characterized by a nonlinear relation as described in previous section.

The effective fundamental period in the direction under consideration shall be based on the idealized force displacement curve. The effective fundamental period shall be calculated in accordance with Equation 2.15:

$$T_e = T_i\sqrt{\frac{K_i}{K_e}} \tag{2.15}$$

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.

2.3.1.2.3 Non-linear Time History Dynamic Analysis Method

Nonlinear Time-History analysis, also known as Nonlinear Dynamic analysis, is a powerful method to identify the response of structure to ground motion acceleration. The book of "Advanced Earthquake Engineering Analysis" states that "It is widely recognized that nonlinear time-history analysis constitutes the most accurate way for simulating response of structures subjected to strong levels of seismic excitation" (Pinho, 2007).

This method of analysis shows the displacement of structural members subjected to selected acceleration series applied to structure. In this study critical frame of each structure has been selected and ground motion acceleration were applied to study the behavior of each frame in terms of displacement to find out the possibility of each frame to be in different performance limit states. For the purpose of this study structure models were created using computer program "CSI SAP2000 14.0 Advanced". In order to select the Earthquake records to apply for analysis three steps were taken; the first step is to specify the design response spectrum, second step is to search for earthquake records according to Earthquake design spectrum and site characteristics and final step is to upload and create a load case to apply the selected time series to the structure and investigate the response of it subjected to applied acceleration load. In chapter 4, represents detailed time history dynamic analysis and its fundamental methodology.

2.3.3 Available Codes for Seismic Performance Assessment.

Seismic Performance assessment of existing building or seismic based-design for a new building that always consider the specification of earthquake code related to the location of construction building. Nowadays have many codes related to seismic performance and have a good approach for construction site [8]. In this section, several codes are defined that have relevance in this thesis.

1. **FEMA-356,** the abbreviation of **“The Federal Emergency Management Agency”** Pre-standard and Commentary for the Seismic Rehabilitation of Buildings. It is a code that is used for seismic performance and assessment of an existing building. It is prepared by **ASCE** “American Society of Civil Engineering’’ and **SEI** “ Structural Engineering Institute” and prepared for **FEMA** Federal Emergency Management Agency Washington, D.C November 2000. The NEHRP Guidelines approved the formal code for the Seismic Rehabilitation of Buildings and the American National Standards Institute (ANSI) of the USA and The guideline are also used by other countries around the world. [44]. In this research FEMA 440 Commonly used.

2. EUROCODE-8. Is the abbreviation of **“The European Standard EN 1998-3”** Eurocode 8 has explained for Design of structures for earthquake resistance, Assessment and Retrofitting of buildings. it is prepared by Technical Committee CEN/TC 250 "Structural Eurocodes" with secretariat BSI and CEN/TC 250 is responsible for all Structural Eurocodes. According to the CEN-CENELEC Internal Regulations, the National Standard Organizations of the following countries are bound to implement this European Standard: Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Slovakia, Slovenia, Spain, Sweden, Switzerland and the UK [26].

3. TEC-(2007) is the abbreviation of **“The Turkish Earthquake Code 2007”.** Specification for Buildings to be Built in Seismic Zones (2007) prepared under the direction of M.Nuray AYDI_OGLU, PhD. Professor, Department of Earthquake Engineering, Bogazici University. Ministry of Public Works and Settlement Government of Republic of Turkey and“Kandilli Observatory and Earthquake Research Institute 34684 Cengelkoy, Istanbul, Turkey”. It is used for Turkey and Turkish Republic of Northern Cyprus, Issued on: 6.3.2007, Official Gazette 0.26454, Amended on: 3.5.2007, Official Gazette 0.26511 [42].

4. ATC-40 is abbreviation of **“The Applied Technology Council”** Seismic evaluation and retrofit of concrete buildings Volume 1 Redwood city, California USA. It is prepared by “California Seismic Safety Commission” 1971, to assist the design practitioner structural engineering and related to Wind, Soils, and Earthquakes for existing buildings. This code is used in the US [43].

2.4 Strengthening Technique of Existing Building.

The technique of strengthening and repair has two principles that are used for strengthening of existing building. The first principle is Stabilization, which is the process of strengthening and repairing the cracks that occurs in structural elements by grouting cement. The second one is strengthening which commonly has many definitions. in general, strengthening is the progress of renewal and reconstruction of a structural members, adding capacity by increasing the strength of structure and improving seismic performance , when concrete of structure elements became poor ,have an awkward strength and loss stability, behavior it may cause damages to the building [7].

2.4.1 Strengthening Selection Method

Furthermore, the strengthening should be estimated and designed with respect to minimize repair and maintenance. The knowledge about the structure is often incomplete and wrong because of lack of many details for performance. For that reason, we must consider seismic codes to design of a strengthening with fulfill requirement [22]. The strengthening of the RC Structures is needed due to the following reasons;

- Increasing load due to live load, wheel loads increase, installation of heavy machines and or vibration.
- Structural element damages because of aging of structure construction or fire damages, steel reinforced corrosion and impact of vehicles on the structure.
- Suitability improvement due to deflection limitation, reinforcement steel stress reduction and cracks width reduction.

- Structural system modification because of walls/columns elimination to slab openings.
- Construction and planning errors due to scanty use of reinforcement steel and dimension design.
- To reach required safety with strengthening/repair.
- The structure does not meet new codes regulations.
- Building does not have adequate lateral rigidity.

The selection of the structural approach form should be discussed on the basis of application convenience, environmental conditions, aesthetic aspects, construction duration, and obstruction usage and stoppage time. It is not only financial and structural aspects [22, 23]. In general, the retrofitting and strengthening process can be categorized into three categories as follow and show in (Figure 20) [32];

1. Assessment and Analysis.
2. Design of Retrofitting and strengthening Techniques/Approach.
3. Construction/Implementation of Retrofitting and strengthening.

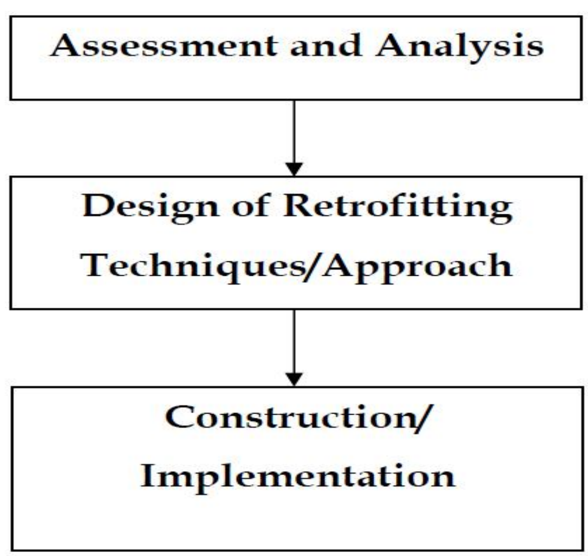


Figure 20. General Process of Retrofitting and strengthening [32].

2.4.2 Repair and Strengthening Techniques

2.4.2.1 Repair of Cracks

Cracks in structures are common occurrence. Concrete structure element cause cracks whenever stresses in the member transcend its strength [24]. Factors affecting cracks, spalling and delimitation [24, 25] are shown in (Figure 26) and listed below:

1. **Structural Cracks Factor.**
   - Incorrect Design Specifications.
   - Overloading (live, dead, seismic, wind).
   - Lack of Construction (faulty construction).
2. **Non-Structural Cracks Factor.**
   - Tensile strength of concrete.
   - Concrete cover over reinforcement.
   - Bond condition (interface between rebar and concrete).
   - Diameter reinforcing bar.
   - Reinforcement corrosion.
   - Foundation settlement (soil problems).
   - Thermal, moisture movement, shrinkage, chemical actions.
   - Joint problems.
   - Elastic deformation.

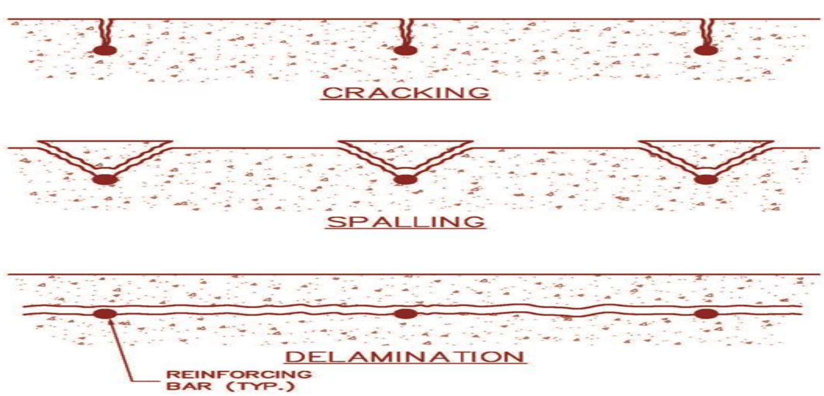


Figure 21.Cracks, spalling, delamination [27].

Cracks are classified and repaired according to type [26]:-

1. If crack width is small (e.g. less than 10 mm);
   - The crack may be repaired using sealed mortar.
   - If thickness of the member is large, cracks may be repaired by using of member is large, cracks may repair used by cement-grout injection, no-shrinkage grout, epoxy grout as shown in (Figure 22).
2. If crack width is small (e.g. less than 10 mm) or there are large diagonal cracks;
   - May be repaired by using elongated (stitching) stones or bricks, dovetailed clamps, metal plates, polymer grids and voids filled by mortar.
   - May be repaired by using bed-joints small standard diameter wire ropes or polymeric grids stirrups.
   - May be repaired by using vertical concrete ribs with closed stirrups and longitudinal bars [28] and [29].

The steps of crack repairing are shown in (Figures 22 and 23).

- First step is to ensure that the area where the crack is located on the concrete structure member must is cleaned of dirty material, corrosion, grease, oil…etc.
- Waste material should be removed.
- The paste or stitches should be removed.
- Crack surface should be sealed against leakage.
- Backfill with a reasonable method of repairing material such as (epoxy grout, stitching…etc).
- The crack area must be smoothed with a trowel and waste grout removed.

Figure 22. Crack repairing used Epoxy grout [27].

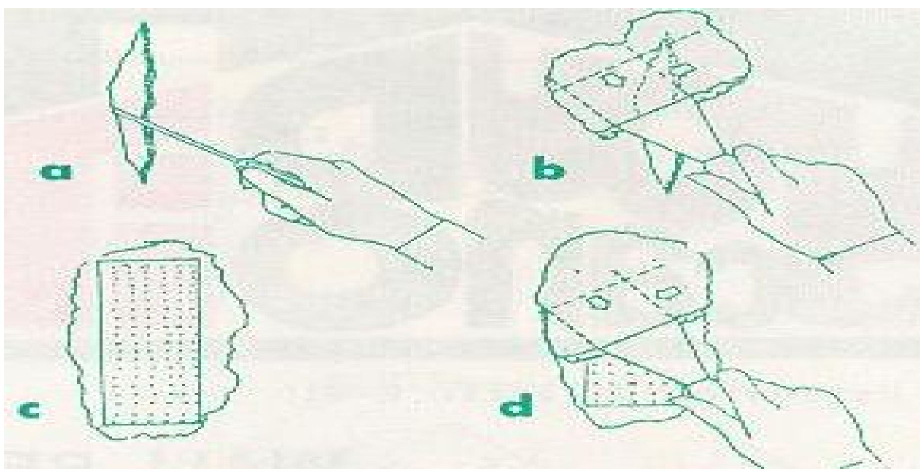


Figure 23. Steps of crack repairing [29].

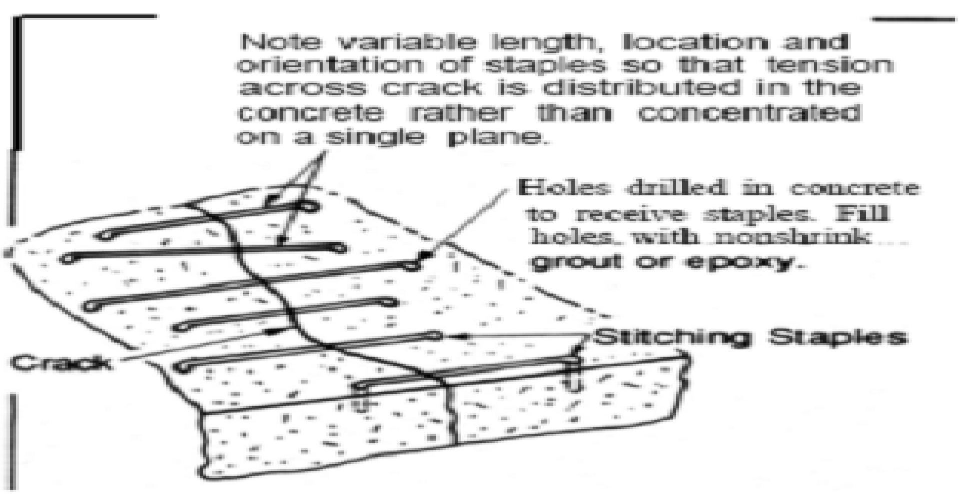


Figure 24. Stitching method of crack repairing [28].

2.4.2.2 Repair and Strengthening of Foundations.

The foundation is an important part of a building because it carries all kinds of loads. It needs strengthening when the applied loads are more than their capacity. Jacketing the foundation anchored to the neck of the column facilitates to transfer of loads properly. Jacket dimensions and depth must be identified from the design by considering codes and standard specifications. During the construction of foundation jacketing, care should be taken so that foundation will not be affected during excavation for jacketing.

Isolating footing strengthening is the process of increasing the dimension and depth of the foundation by adding reinforced steel bras to increase its resistance. The process is described as follows [30];

1. Excavating around the footing.
2. Cleaning and roughening the surface of concrete.
3. Installing dowels at 25-30 cm spacing in a vertical and horizontal direction using appropriate epoxy materials.
4. Using steel wires with fasting new steel bars, the number of steel bars will depend on the design.
5. Coating footing to achieve a good bond between the old and new concrete.
6. Pouring the concrete before drying the materials, using low-shrinkage concrete.

The previous steps are illustrated in (Figures 25 and 26).

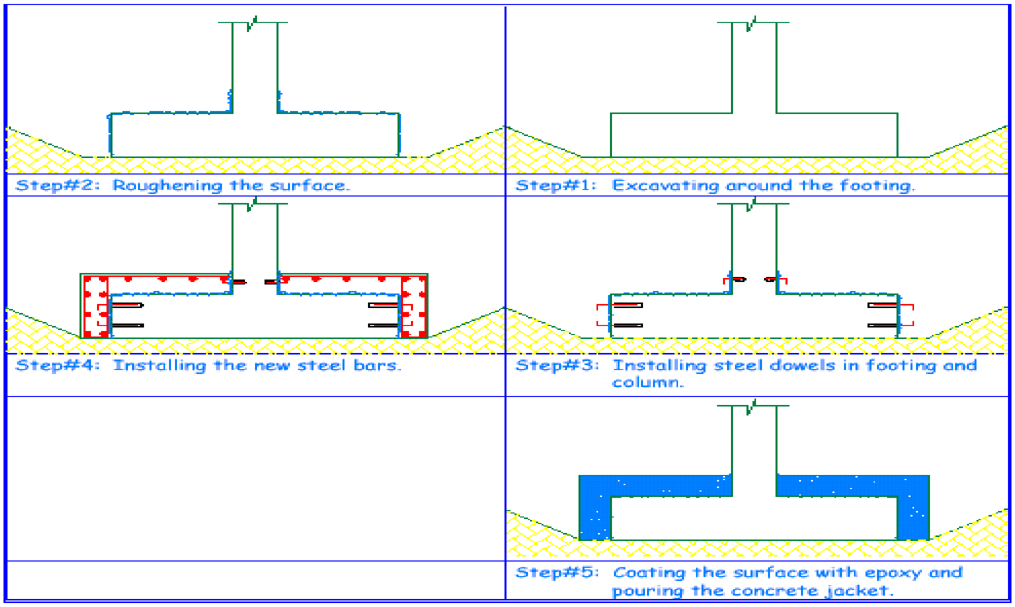


Figure 25. Strengthening jacketing of Footing [30].

Step#1 xcavating around the footing.

Step#2 hardening the surface and installing the dowels.

Step#3 Installing the main steel.

Step#4 Completing the jacket.

Figure 26. Steps of strengthening jacketing of footing [30].

2.4.2.3 Repair and Strengthening of Slabs

Slab strengthening is required if the damage done is because of increased applied loads on slabs or unsafe design or corrosion of reinforcement steel bars or cracks on slabs. For this kind of damage we should make use of one or more of the solutions listed below [30];

1. When the slab is incapable of carrying the negative moment and the lower steel is adequate, an upper steel mesh should be affixed with a new concrete layer.

2. When the slab is incapable of carrying the positive moment or when the dead load (loads on slab) is much less than the live load held by the slab, then a new concrete layer should be added to the bottom of the slab. To put these solutions into effect, the following steps should be made as shown in (Figure 27, 28, 29):

1. Removing the concrete cover.
2. Cleaning of steel bars by sand compressor or wire brush.
3. Preventing corrosion by coating steel reinforcement steel bars with epoxy materials.
4. Adding a new mesh of steel when a high level of corrosion is detected.
5. Installing and fasting the new steel mesh vertically to the slab of the roof and surrounding it horizontally to the beams by using dowels.
6. Coating the surface of the concrete with adequate epoxy materials which will facilitate a good bond between the old and new concrete.
7. Pouring the concrete with a design thickness before epoxy dries, using low shrinkage concrete.

There are some other techniques used for slab strengthening such as [30]:

- By adding steel plates connected with vertical screw bolts to increase shear capacity.

- By post stressed reinforcement.
- By adding steel beams.
- By adding reinforced concrete inside the holes of hollow slabs.

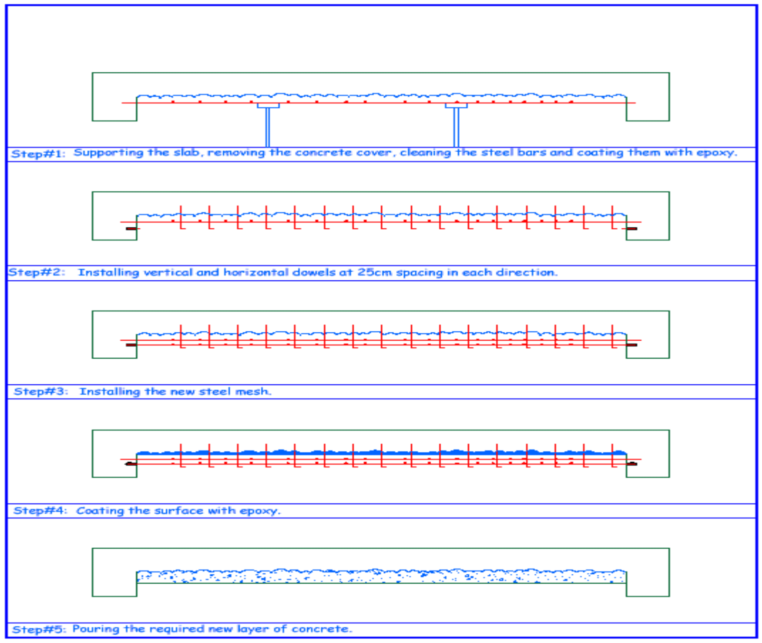


Figure 27. Strengthening a slab by increasing bottom depth [30].

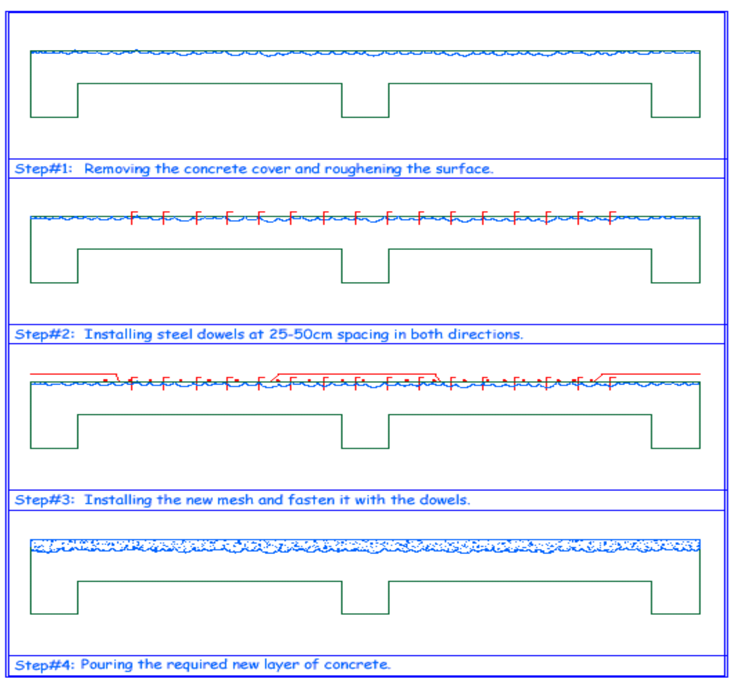


Figure 28. Strengthening a slab by increasing top depth [30].

Figure 29. strengthening of a slab by increasing top and bottom depth.

2.4.2.4 Repair and Strengthening of R.C Columns.

Columns are an integral part of the structure of a building and constitute the most important safety factor in the case of an earthquake. The strengthening of columns is required when the following factors are present;

- There has been an increase in the number of floors to the original building and the design is considered unsafe.
- Concrete compressive strength of the columns, percentage and type of concrete used does not comply with codes and standard specifications.
- Column inclination is more than permissible.
- Foundation settlement is more than permissible.

Improving two important factors are required during the strengthening process. Bending resistance can be improved by increasing the cross sectional area of the column. Longitudinal reinforcement, and shear resistance and ductility can be improved by adding stirrups, steel belts and straps. Two techniques are used for the strengthening of columns:

2.4.2.4.1 Reinforced Concrete Jacketing

The jacket size and the number and diameter of steel bars to be used during the strengthening process depend on the structural analysis that was performed on the columns. Reducing or eliminating loads should be considered in some cases through the following steps [30];

- Setting mechanicals jacks between floors.
- Setting additional props between floors.

Moreover, in some other cases when corrosion of reinforcement bars are detected the following steps can be taken:

- Removing the concrete cover.
- Cleaning of steel bars with a sand compressor or wire brush.
- Preventing corrosion by using epoxy to coat steel bars.

If the previous steps are not needed, jacketing techniques could be used by the following steps [30]:

1. Cleaning and roughing the surface of columns.
2. Installing steel connectors or dowels by making holes 3-4mm or larger than the diameter of used steel and at a depth 10-15cm by fasting new stirrups in both vertical and horizontal directions, the space not more than 50cm.
3. Filling holes with adequate epoxy material and then inserting the connectors.
4. Fasting vertical steel bars by adding vertical steel connectors to the jacket (as step 1 and 2).
5. Installing stirrups and vertical steel bars to the jackets depending on design dimensions and diameters.
6. Coating the R.C Column with an appropriate epoxy material with a view to creating a good bond between the old and new concrete.

7. Pouring the new concrete layer before the epoxy material dries, the concrete should have a low shrinkage with a small aggregate content, sand, and cement and shrinkage preventive materials. The previous steps are clarified in (Figure 30).

2.4.2.4.2 Steel Jacketing.

This technique is chosen when the applied load is increased or there are problems in the stirrup zone and increasing the cross sectional area is not possible. If the previous steps are not needed, steel jacketing techniques can be performed in the following way [30];

1. Cleaning and roughing the surface of the columns.
2. Cleaning of the steel bars with a sand compressor or wire brush.
3. Preventing corrosion by using epoxy to coat steel bars.
4. Installing the steel jacketing with the design size and thickness, by pouring appropriate epoxy material. This allows a good bond between the old and the new concrete.
5. Filling the space between the steel jackets and the concrete columns by appropriate epoxy materials.

The previous steps are illustrated in (Figure 31).

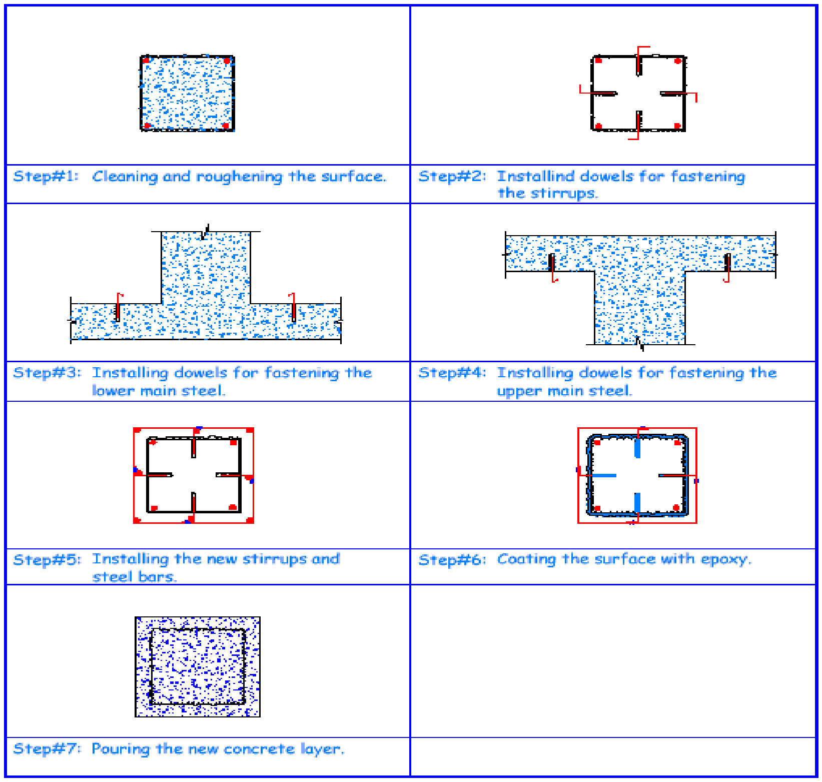


Figure 30. Increasing cross sectional area of columns by RC Jacketing [30].

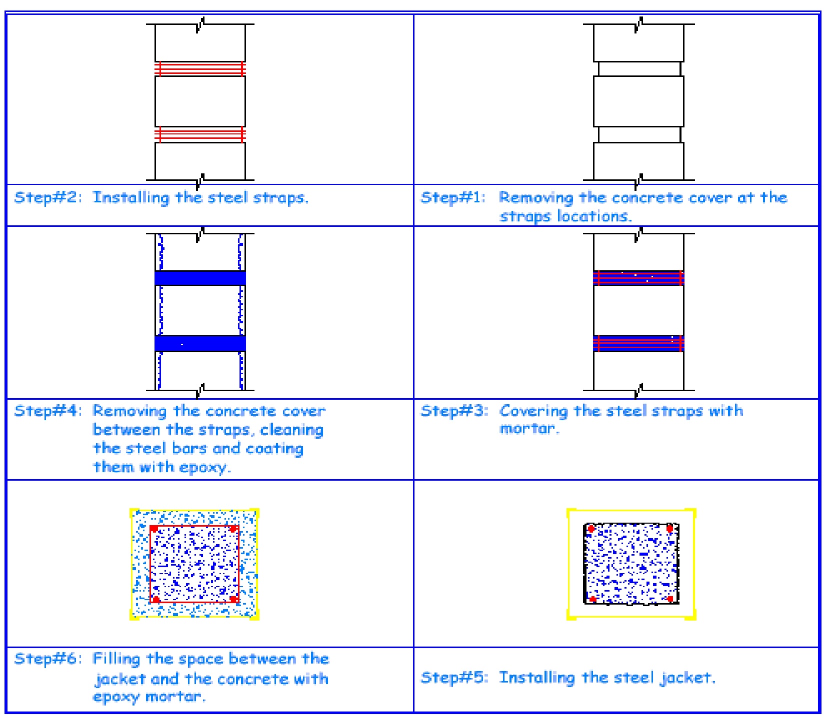


Figure 31. Increasing cross sectional area of columns by Steel Jacketing [30].

In some cases the bending moment should be improved and transfer successfully to other floors. For this reason, we almost using steel collars (steel angles) at the neck of columns connecting by bolts or appropriate epoxy materials. The steel elements should be covered with a shotcrete to protect against fire. This technique will not only increase capacity but laboratory work done by under loading 4 corners of RC Columns. The capacity will increase 92% after reinforcement and 89% after repair and the rigidity increase 32% after reinforcement and 22% after repair. The ductility will improve with no more change in rigidity as shown in (Figure 32) [31].

Figure 32. Increase strengthening by Steel angles.

2.4.2.5 Repair and Strengthening of Beams

Beams have an effective role in the structural system by connecting columns to make a building frame system. It requires strengthening when having an unsafe reinforcement steel bars or cannot resist bending resistance and shearing force in order to avoid the load-bearing system strong beams and weak columns deficiency. There are various solutions of reinforcement concrete beam as follows;

I-Adding Reinforcement Steel Bars to the Main Steel without Increasing the Beam's Cross Sectional Area.

This solution is used when the reinforced steel bars of the beam are not capable of withstanding the applied stresses to the beam. The following steps can be taken [30]:

1. Removing the concrete cover of both upper and lower steel bars.
2. Cleaning the steel bars and coating with appropriate materials to prevent corrosion.
3. Making holes in the whole of the beam span under the slab as shown in (Figure 38), with a width 15-25 cm apart, a diameter of 1.3cm, so extending the beam's total width.
4. Filling holes with epoxy materials that have low viscosity and installing steel connectors to fasten the new stirrups.
5. Installing steel connectors into columns in order to have steel bars added to the beam.
6. Adding closed stirrups using steel wires.
7. Coating surface by bonding epoxy materials.
8. Pouring concrete cover over new steel and new stirrups.

The previous steps are clarified in (Figures 33 and 34).

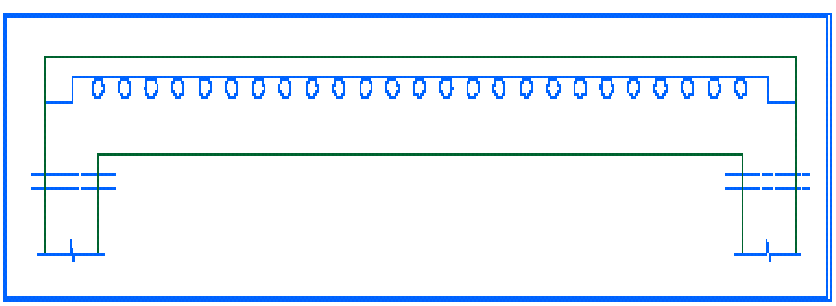


Figure 33. Holes in the span of beam [30].

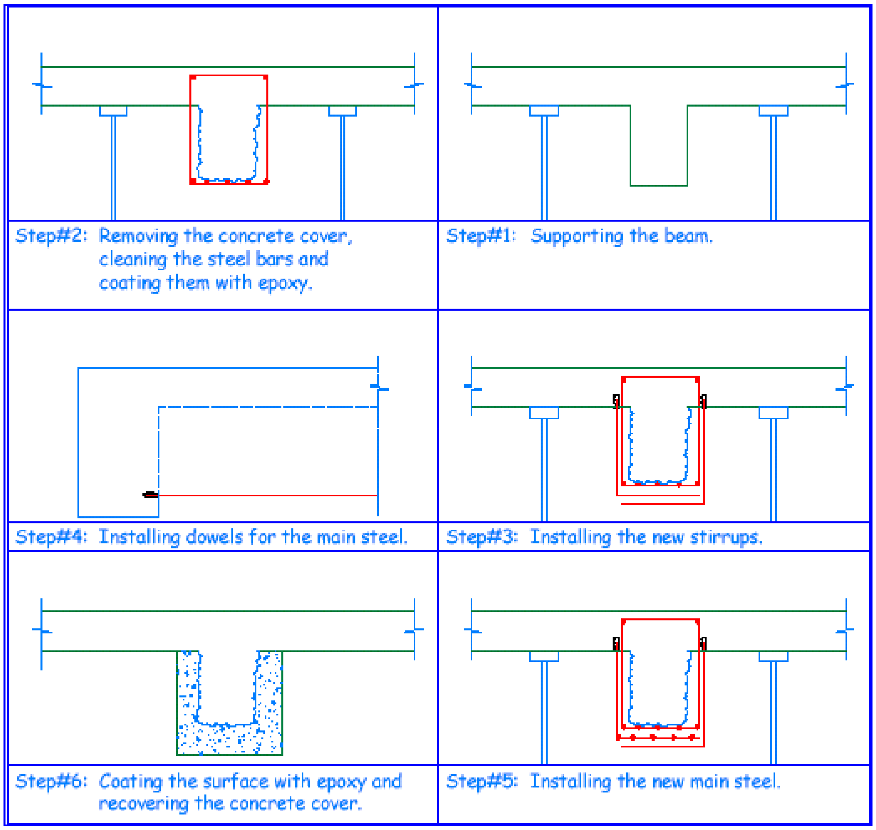


Figure 34. Strengthening a beam without increasing the cross sectional area [30].

II-Increasing Both the Reinforcing Steel Bars and the Cross Sectional Area of Concrete.

This solution is used when the reinforced steel bars and the concrete of beam are not capable to withstand additional applied stresses to the beam. The following steps should be followed [30];

1. Removing concrete cover for both upper and lower steel bars.
2. Cleaning steel bars and coating with appropriate materials to prevent corrosion.
3. Making holes in the whole of beam span under the slab as shown in (Figure 35), with a width of 15-25 cm apart and so enlarging the size and dimensions of the beam.

4. Filling holes with epoxy materials that have a low viscosity and installing steel connectors to fasten the new stirrups.
5. Installing steel connectors into columns in order to have steel bars added to the beam.

Adding closed stirrups using steel wires, coating the surface by bonding epoxy materials, and pouring concrete cover over new steel and new stirrups.

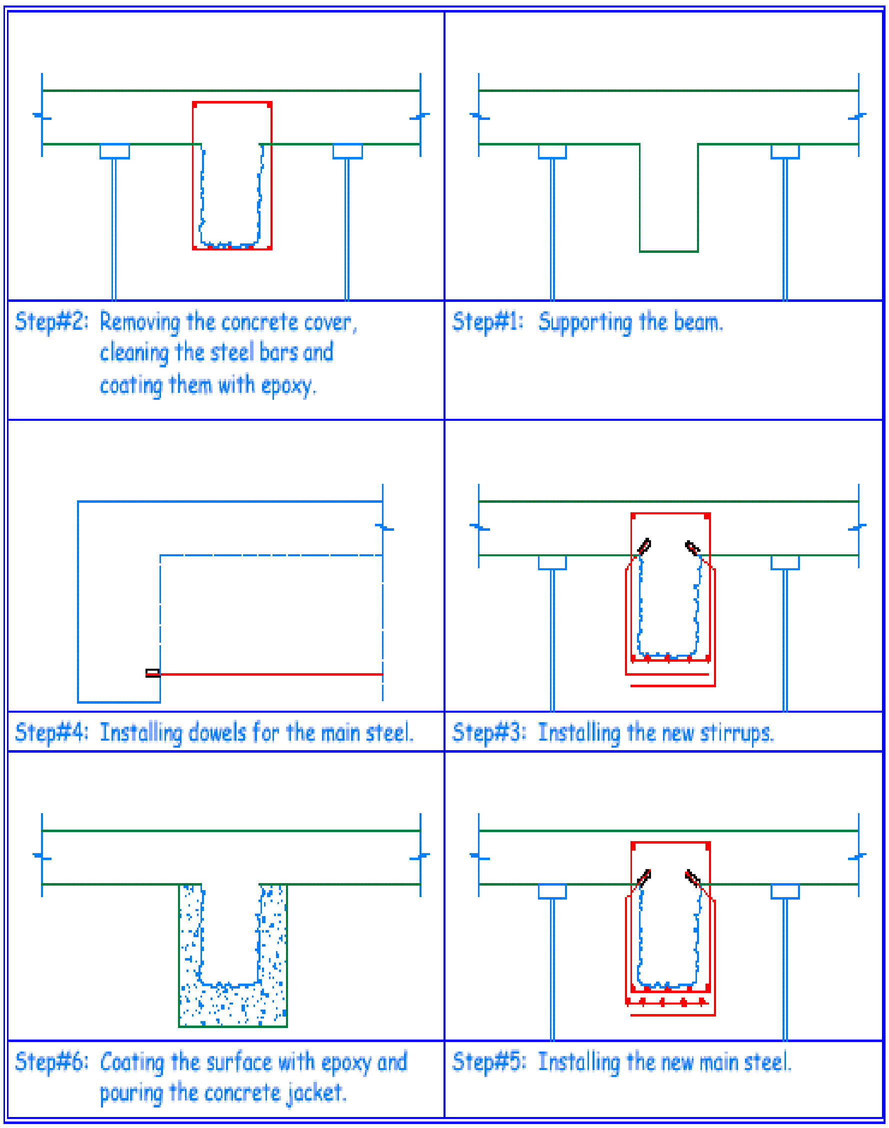


Figure 35. Strengthening a beam by increasing cross sectional area and bars [30].

III-Adding Steel Plates to the beam.

This solution is used when the strengthening required improve the resisting moment and shear stress with the designed steel plates size and thickness. Then those plates are connected to the beam as follows;

1. Cleaning and roughing the surface of the attached plates.

2. Coating the surface of the concrete with epoxy materials.

3. Making holes for a connection into the concrete beam and plates.

4. Putting an epoxy mortar layer on the upper of plates with a 5mm thickness.

5. Connecting and attaching the steel plates to the concrete beam using bolts.

The previous steps are illustrated in (Figure 36).

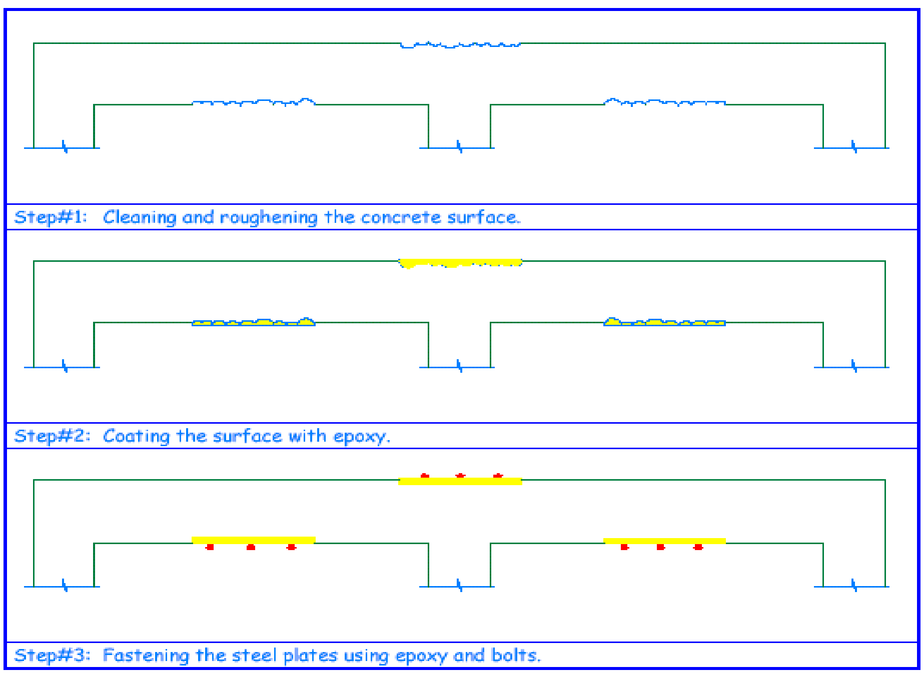


Figure 36. Strengthening a beam by adding steel plates [30].

In some cases, when we implement the strengthening we need to support the beams partially or completely against additional loads, supporting the beam by a bottom steel beam or a top one is also used, as shown in (Figure 37).

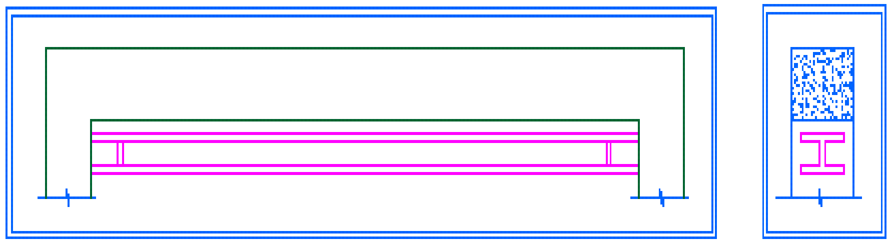


Figure 37. Reducing loads on the beam using steel beam [30].

The following pictures (Figure 38) were taken during the strengthening of an existing building; presenting a practical method of implementing some strengthening techniques.

Strengthening a beam , slab and column.

Strengthening a beam and Slab.

Jacketing a beam by increasing bars and cross section.

Strengthening by steel plates.

Figure 38. Strengthening of Beam [30].

2.4.2.6 Repair and Strengthening of Column-Beam Joints.

Column-Beam joints are generally called nodes in a structural analysis system. Nodes are very important part of building that cause damages during earthquake and suffer shearing, adherence and anchorage damages. When structure cause the seismic loads it immediately make plastic hinges in columns and beams surface. Beams suffer fracture due to internal stress and suffer various stresses due to compressive and tensile stress for this reason they lose capacity in the column-beam joints.

The strengthening of nodes is done by cleaning the concrete surface and sheathing plates of both parts of the columns –beams, connecting to each other, filling with epoxy materials and increasing the longitudinal and lateral reinforcement steel bars with increasing stirrups to resist seismic loads and increasing its rigidity. Shear resistance, may also be improved by using steel sheets with epoxy materials or pre-stressed bolts in order to protect the sheets against corrosion problems as shown in (Figure 39) [9].

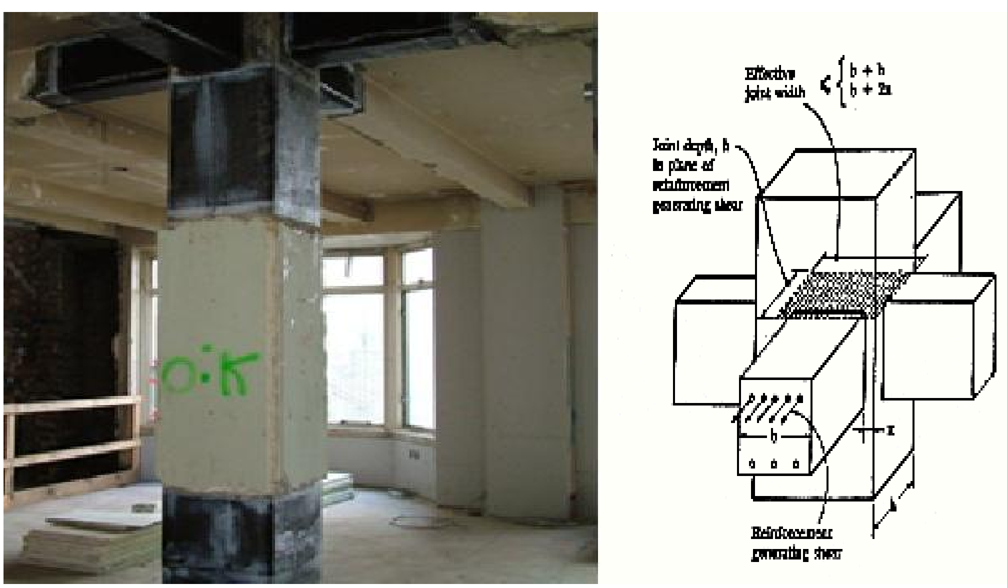


Figure 39. Strengthening of Column-Beam Joint (Nodes) [33].

2.4.2.5 Repair and Strengthening of R.C Shear walls.

The main purpose of repairing and strengthening shear walls is to improve their lateral load bearing capacity or to correct irregularities. Dimensions and reinforcements added to the shell should be at least 5 cm and thickness edge column be at least 10 cm. The procedure to follow is [9], [30];

1. Cleaning and roughing the surface of the concrete.
2. Installing steel connectors with 25-30cm whole spaces in vertical and horizontal directions, the diameter will obtain in the design and the depth is 5-7 times the diameter.
3. Installing the steel connectors into the footing of shear walls by using the number and the diameter of the main vertical steel bars with epoxy materials.
4. Using steel wires to the steel connecters by installing the steel mesh and fasten it.
5. Coating the wall surface with appropriate epoxy materials.
6. Pouring the concrete layer before the epoxy material dries, the concrete should have low shrinkage materials.

The previous steps are illustrated in (Figure 40).

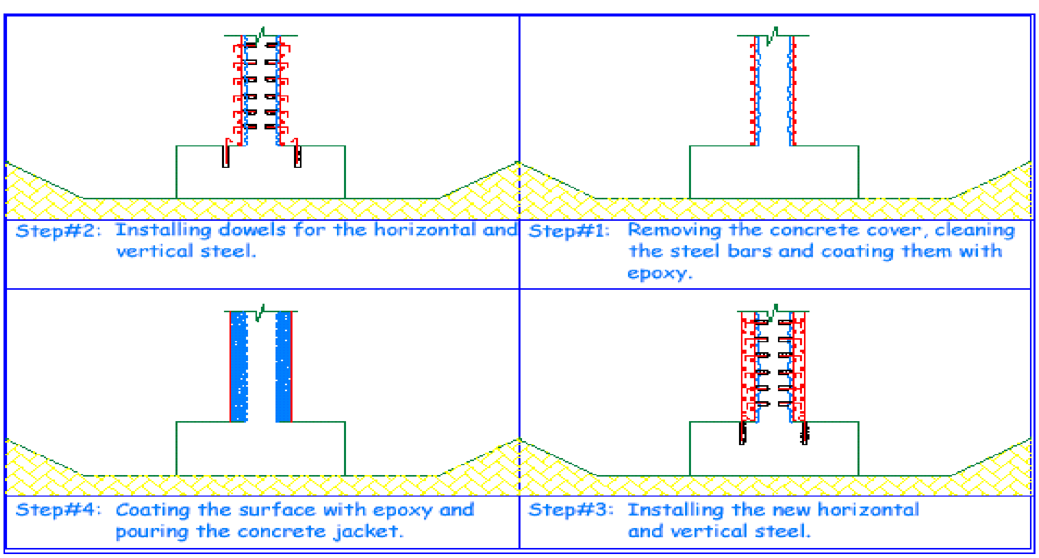


Figure 40. Increase strengthening of RC Shear wall [30].

2.4.3 Improving Structural System

2.4.3.1 Strengthening by Adding Walls or Columns

The strengthening of a building can also be enhanced by adding shear walls or columns to achieve the required building strength. Adding elements that need to be installed must be done under the proper conditions, especially the slab and foundation level of elevation should be carefully done in order to transfer the shear forces from the building system and protect its safety and rigidity. If the building has several slab level with adding shear walls by connecting centre of rigidity and centre of mass to minimized torsion caused shown in (Figure 41) [9].

Figure 41. Adding Shear walls and Columns.

2.4.3.2 Strengthening by Seismic Restraints (Seismic Bracing).

Seismic forces are exerted on to the building during an earthquake strike. This force acts horizontally through the structure as well as the piping, ductwork, cable trays and so on. Sometimes the building has been constructed without accounting for the horizontal loads only considering gravity and vertical loads for design purpose. Seismic restraints (i.e. braces) resist the horizontal forces and keep the structure secure and safe in place, the main purpose of the seismic bearing brace to minimize the loss of life and severe damages during earthquakes as shown in (Figure 42) [41].

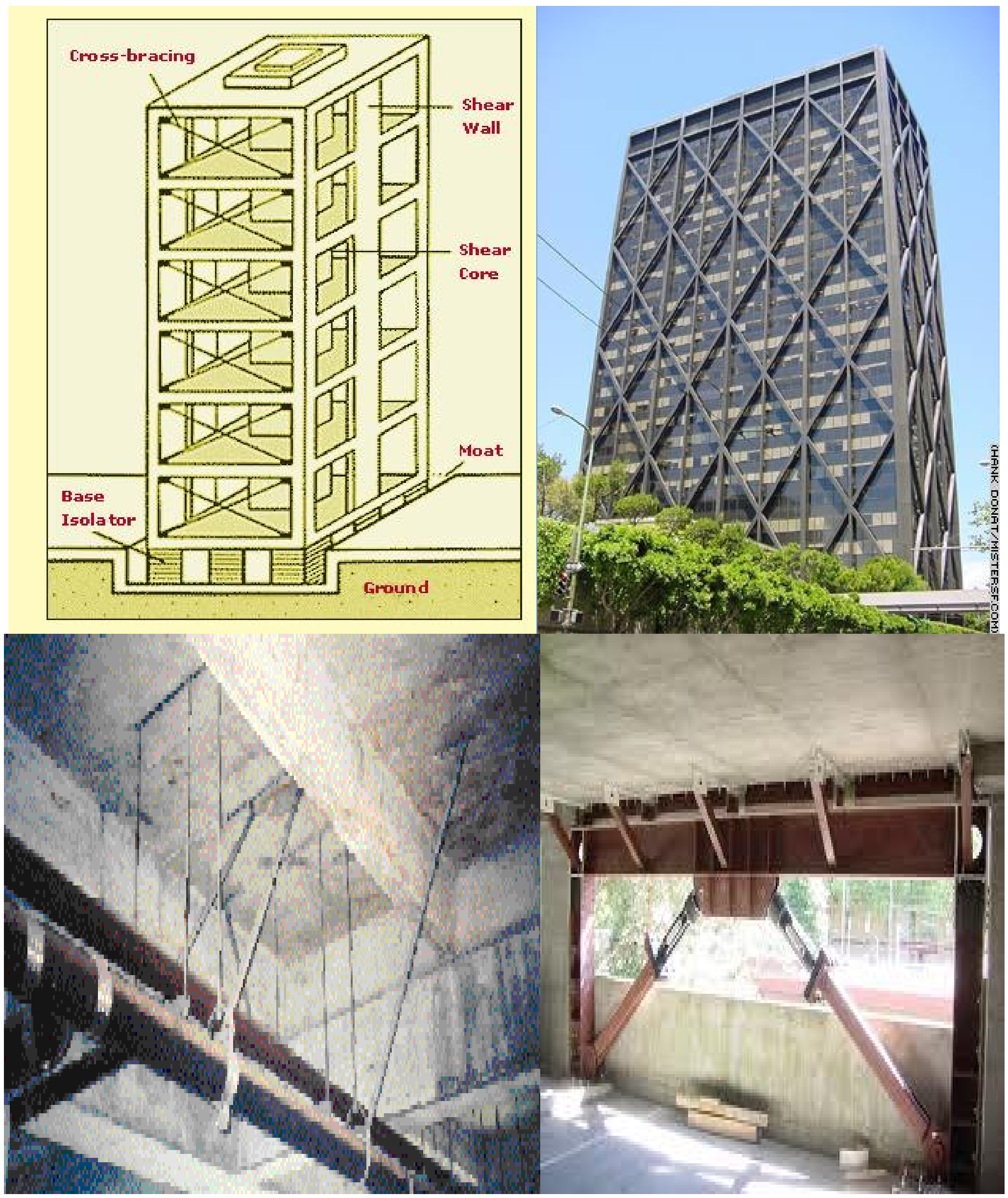


Figure 42. Seismic Restraints of building (i.e. Braces) [41].

2.4.3.3 Strengthening by Seismic Isolation Base

Nowadays, the use of isolation is increasing because of earthquake problems and structural damages during and after an earthquake strike because of seismic loads shows (Figures 43 and 44) main concepts of isolation base are the following;

1. To protect the structure from the ground motion to make it move when an earthquake occurs and so not have much structural damage with an elected shifting period of vibration to avoid severe damage and reducing base shear force [34].
2. To reduce the potential damage caused by earthquakes.
3. To reduce transmitting the base shear of the structure and the displacement to increase damping [35].
4. Keeping the building safer for use after earthquakes.
5. To minimize the drift and acceleration, and so have a flexible structure this reduces the acceleration and drift in the structure [36].
6. Minimize the damage of equipments such as (computers, precision equipment, medical equipments, communication equipments ... etc).
7. To prevent deformation.
8. To provide high frequency of performance against severe earthquakes.
9. To protect the building from collapse.

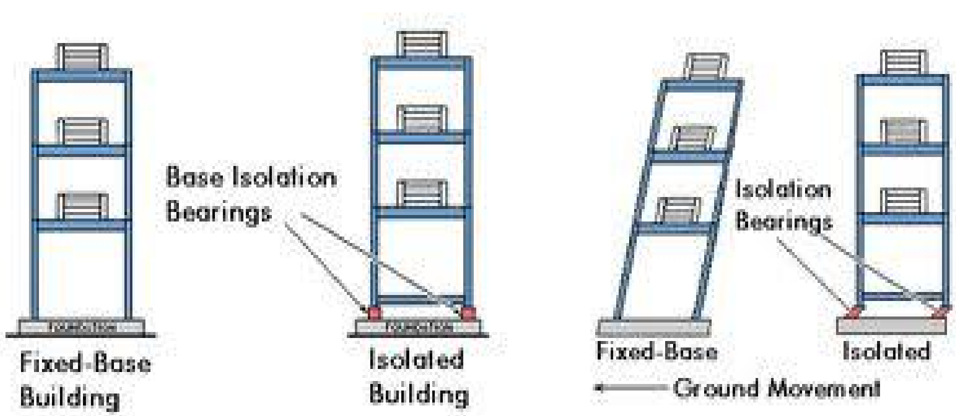


Figure 43. Movement of Building with Base Isolation and No Base Isolation [40].

There are two basic type of seismic base isolation system;

1. The system has been widely used in recent years by the elastomeric bearings; it is natural rubber pads or neoprene installed between the structure and the foundation to reduce the frequency of ground motion than a fixed base structure would normally have [39].
2. The system of sliding isolation such as lead –bronze steel or friction pendulum to limit transmit of base shear across the isolation interface. Up to now it has only been used mostly in China and the USA. It is still going through much research and is generally used in new and retrofit buildings [39].

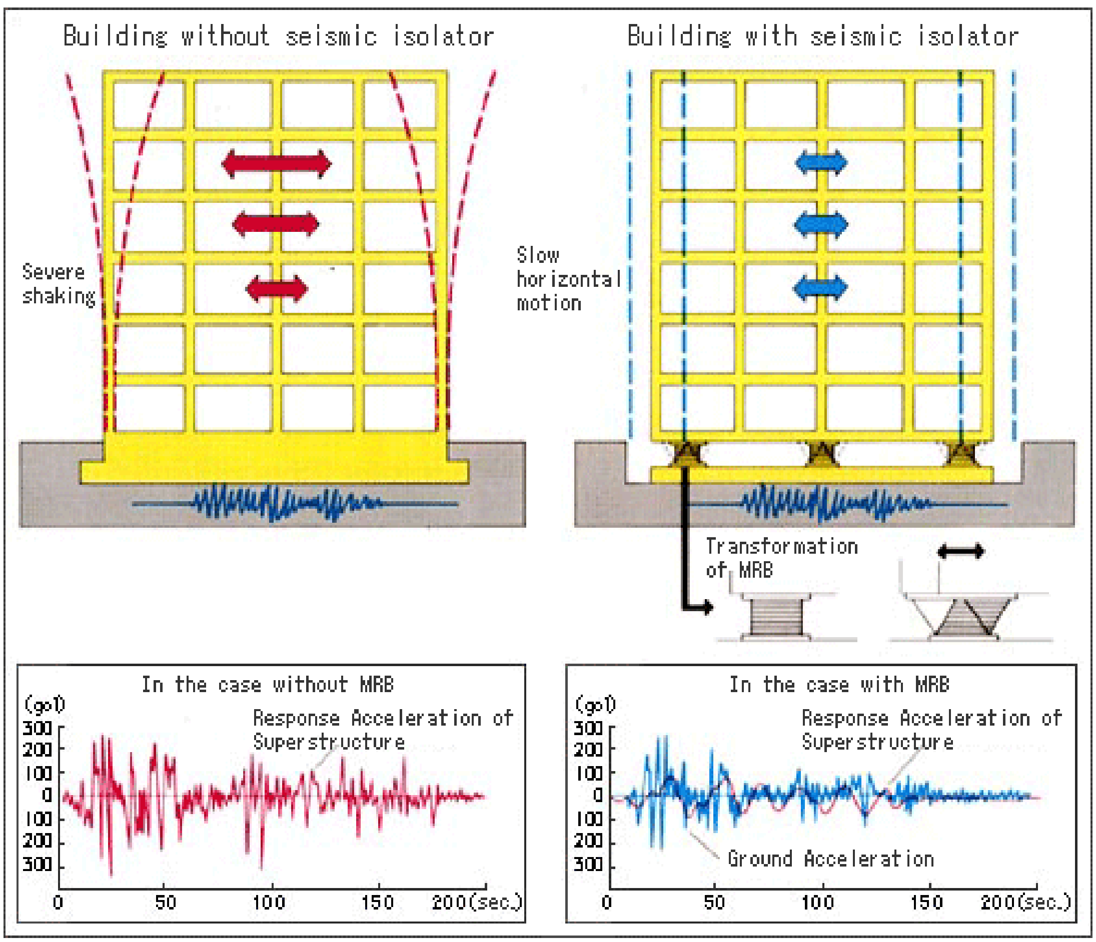


Figure 44. Compare between isolated and fixed base building with affect of ground motion [38].

Chapter 3

METHODOLOGY AND DATA COLLECTION

3.1 Introduction

The main aim of this study is to determine the behavior of a building during an earthquake and its contributing to the building system. The collection of data for assessment and evaluation the performance of the building is essential to the nature of this study. The process must be completed before the earthquake occurs. The information that is collected from the building depends on the building type and a suitable analytical approach that best accommodates the building under inspection. First of all, a list of essential requirements for the assessment of the building is required to allow the building to be properly evaluated.

In TEC 2007 “Turkish Standard Earthquake Code’’ includes several important points before an assessment of an existing building as collection data can proceed [42];

1. Determination of information level (limited, moderate or comprehensive).
2. Determination of concrete properties and reinforcing bars.
3. Calculation of elements critical cross-section strengths (bending and shear).
4. Determination of size, location and number of reinforcing bars in sections.
5. Determination of failure types of reinforced concrete elements.
6. Determination of bending and wrapping reinforcement amounts and their details which are required for the determination of element’s damage limits.

3.2 Methodology

The Compressive strength for the fifty six core sample was made test of cylindrical specimens was carried out according to ASTM C39/C39M – 11.

3.2.1 Building Data

This thesis is a case study of strengthening and assessment performance of an existing building. The building is the Seaside Hotel located alongside the Salamis Bay Continental Hotel and Resort on the main Salamis Road to Famagusta in the Turkish Republic of Northern Cyprus. The building was constructed in the early 1970s.

The location of the building on the map of the city of Famagusta, Cyprus is shown with coordinates 35°12'19.67" N 33°54'06.23" E by Google Earth Map 2003 archives in (Figure 45).

Figure 45. Seaside Hotel Locations in Famagusta, Cyprus [Google Earth].

3.2.1.1 Structural Plans

The Seaside Hotel is a reinforced concrete building and consists of 10 storeys**,** comprised of two parts: the main section of the hotel and the hotel reception area. The building area dimensions are 24.7 m x 8.80 m and the height of each floor is 3 m.

The complete plans of the building, floor plans, and dimensions of columns, beam lengths, stair types and dimensions, foundation and shear wall type are shown in Appendix A at the end of this thesis. For assessment and performance purposes only the main part of the Seaside hotel was used in the analysis for consideration.

Figure 46. Seaside Hotel front and Side View in Famagusta, Cyprus.

3.2.1.2 Building Geometry

The geometry of the building is determined by the following;

1. Designs and plans for the building are not available. The static project was determined after the building survey had been inspected.

2. Determination of locations, span length, heights and reinforcement concrete element dimension and walls dimensions.

3. Obtaining the building geometry and necessary details to calculate the building mass.

4. Taking notes of short columns or other similar issues that have been recorded in the storey drawings and the corresponding cross-sections.

5. Detachment, adjacency or the existences of joints are stated.

3.2.1.3 Foundation Type and Dimension Details (Sub Structure)

A foundation is the lowest and supporting part of a structure. It is the main part of a building that resists any soil problem and lateral loads, including seismic loads. In this thesis, we assumed the type of foundation that was constructed in a case study of the Seaside Hotel by Mat foundation (Figure 47).

Figure 47. Seaside Hotel ground floor.

3.2.2 Determination of Concrete Material Characteristics

The fundamental materials of a reinforced concrete building are the concrete and steel reinforcements. The strength and capacity of the building will be determined by measuring both of these materials. To this end several tests should be done to identify the character of materials these tests are of a destructive and non-destructive experiment methodological approach.

3.2.2.1 Core Test

According to ASTM C42 / C42M - 13 Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete. Fifty six samples were taken from the building as mentioned [TEC 2007] and in order to obtain the variability of concrete strength behavior of structure minimum numbers of core samples to be taken are fifty six by having at least three samples from each floor in high rehabilitation performance. These samples were taken to get a result of concrete material characterization that is concrete compressive strength shown in the figures below.

Core sample taking and core sample tests are the ones mostly used as the destructive methodologies to get the concrete strength. When we select the place of core tests, we must notes several points. These points are;

- The place of the core sample should represent the concrete in good behavior.
- Should not affect the strength of structural elements.
- Prevent from facing high pressure tensions.
- Should not be taken from reinforced concrete zones as much as possible.
- The natural humidity of the sample should be maintained until the test day [45].

Core samples should be examined under axial pressure after capping the voids on the sample and filling the place by high strength repair mortar, there will have very low quality of concrete. A good quality depends on the structural elements behavior, it has many difference between concrete strength by destructive and non-destructive tests.

Figure 48. Core samples.

Procedures of preparing core sampling are according to ASTM C42/C42M - 13 Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete (Figures 49 and 54).

1. Selection place of core sample taken on structural elements.

Figure 49. Core sample selection.

2. Scanning the structural elements for detecting reinforced zones by Ferroscan.

Figure 50. Scanning reinforced zone by Ferroscan.

3. Taking sample from elements and considering a good way.


Figure 51. Taking core sample.

4. Cleaning the sample and cutting out for required dimensions (D=65 mm, length=80 mm).


Figure 52. Cutting and cleaning samples.

5. Capping the upper and bottom of the clean core specimens by very high temperature mortar then and keeping samples in a laboratory room that have high humidity of 90% and 21ºC temperature. Then the samples were kept in water for 48 hours before compressive strength test.

Figure 53. Capping and covering samples by water.

6. Testing samples for compressive strength.

Figure 54. Core samples after compressive strength test.

3.2.2.2 Schmidt hummer Test

Testing the compressive strength of concrete using the Schmidt hammer (also called: rebound hammer, impact hammer and accelerometer), ASTM designation C 805 75T is considered as a non destructive test. The major principle of the Schmidt hammer test is that it measures the rebound of an elastic mass when it collides with the concrete surface under the test [47]. The tested concrete has to be smooth and firmly supported. The hammer is pressed against the concrete, and then the mass inside the hammer is rebounded from the plunger and gives a reading on the scale. This reading is called the Rebound number which is the distance traveled by the mass expressed as a percentage of the initial extension of the spring [47].

Note that the rebound number depends on the hardness of concrete, and the energy stored in the spring of hammer, mass size, energy it absorbs from the collision. There are many factors affecting the obtained results including moisture content, surface smoothness and finish, the presence of carbonation and coarse aggregate type [47] (Figure 55).

Figure 55. Schmidt hammer compressive strength taking.

There are the Schmidt hammer procedures for use [47]:

1. Smoothing and removing mortar and painting from the surface in the test area should be prepared very well note that circle of test area is 15 cm.
2. Placing hammer in touch with the test area and pressing and push the barrel forward until there is a rebound action when you can get the measurement.
3. 10 readings are obtained within 10 minutes. There should not be more than a 25mm distance between any two reading points.

3.2.2.3 Compressive Strength Test

The test of specimens for compressive strength of cylindrical to that concrete having a density in excess of an 800 kg/m^3 [50 lb/ft3] was carried out according to ASTM C39/C39M – 11 (Figures 56 and 57). In this study, the loading test was made for fifty-six cylindrical specimens as was mentioned above. All calculations and corrections of the compressive strength were carried out according to guidelines specified in "Concrete Society Technical Report No.11-Core Testing for Strength" and the three following formulas were used to calculate the actual strengths.

<u>Estimate Compressive strength for Vertical direction:-</u>

$$\text{Actual strength} = \frac{2.3}{(1.5 + \frac{1}{l})} * \text{Core strength} \tag{3.1}$$

$$\text{Potential strength} = \frac{3}{(1.5 + \frac{1}{l})} * \text{Core strength} \tag{3.2}$$

Where:

λ=L/D

L= Core length

D= Core diameter

Estimate Compressive strength for Horizontal direction:-

Actual strength = $\dfrac{2.5}{(1.5 + \dfrac{1}{l})}$ * Core strength (3.3)

Potential strength= $\dfrac{3.25}{(1.5 + \dfrac{1}{l})}$ * Core strength (3.4)

Where:

λ=L/D

L= Core length

D= Core diameter

For Singular Bar:-

Corrected strength = Core strength × [1.0 + 1.5(φr /φc) × (*h*/*l*)] (3.5)

Where:

Φr=Diameter of bar

Φc= Core diameter

h= Axis distance of bar from nearer core edge

L= Core length

For Multiple Bars:-

Corrected strength = core strength × [1.0 + 1.5 Σ(φr × *h*)/(φc × *l*)] (3.6)

Where:

Φr=Diameter of bar

Φc= Core diameter

h= Axis distance of bar from nearer core edge

L= Core length

If the spacing of two bars is less than the diameter of the largest bar, only the bar with the higher value of (φ_r × *h*) should be considered.

Find all compressive strength calculation details for all columns and slabs, and compressive strength for each floor in Appendix B.

The compressive strength of each column's core sample was calculated, and for each floor 3 samples for columns were taken and 3 samples for beams. Each floor's compressive strength elements (Columns, Beams) were found using the following Equation 3.7;

$$f_i = f_{avg.} - SD \tag{3.7}$$

Where,

f_i = Compressive strength of each the $_{i}th$ floor.

f_{avg} = Average compressive strength of the $_{i}th$ floor.

SD = Standard deviation compressive strength of each the $_{i}th$ floor.

Figure 56. Compressive strength tests [Materials of Construction Laboratory].

Figure 57. Compressive strength and bar steel in core sample [Materials of Construction Laboratory].

Table 5. Compressive strength test results for each floor.

No.	Floor Number	Columns Compressive strength(MPa)	Beams Compressive strength(MPa)
1	(Ground)	14	36
2	1^{st} floor	14	36
3	2^{nd} floor	33	7
4	3^{rd} floor	8	34
5	4^{th} floor	11	21
6	5^{th} floor	12	25
7	6^{th} floor	8	12
8	7^{th} floor	12	28
9	8^{th} floor	12	20
10	9^{th} floor	11	33

3.2.2.4 Tensile Strength Test

One material property that is widely used and recognized is the strength of a material. Tensile testing is a destructive test process used to outfit information about the strength and ductility of a material or to face acceptance test requirements. Tensile testing includes applying an ever-increasing load to a test sample up to the point of failure. The process produces a stress/strain curve showing how the material reacts throughout the tensile test and it is known as a tension test. The data generated during the tensile test is used to determine mechanical properties of materials and provides a quantitative measurement of tensile strength, yield strength and ductility or stiffness [52] (Figure 58).

According to ASTM E8 referenced in this study, a tension test was made for 3 specimens of steel bars having (diameter #10). General specifications of steel bars depend on the diameter of the steel bar used for testing and the specimen length should be equal to 20 times the diameter. The length of the is (20 cm) also in the Turkish standard code (TEC 708 BC-1A, March 1996) mentioned specification [53], [54] as shown in (Figure 59).Tensile strength test procedures carried out by ASTM E8:

Figure 58. Tensile strength test machine [EMU Laboratory].

Figure 59. Three steel bar specimen before and after tension test.

Calculate the strength parameters;

a. Yield stress ,σ

$$\sigma = F / A \qquad (3.8)$$

b. Ultimate tensile strength, Ult. [MPa]

$$Ult. = F / A \qquad (3.9)$$

c. Strain, ε [%]

$$\varepsilon = Lf - L_{\circ}/L_{\circ} \qquad (3.10)$$

when ; σ = yield stress

F= Force applied to the steel bars.

Ult.= Ultimate tensile strength [MPa].

ε = Strain, ε [%].

Lf = Length of specimen before tension test [mm].

$L_{\circ}$=Length of specimen after tension test, Elongation [mm].

Table 6. Tensile strength test calculation.

No.	Yield (kN)	Ult.(kN)	Elongation(mm)	Area of bar(10mm)	Yield Strength (MPa)	Ult.Strength (MPa)	Strain (%)
1	17.5	22.9	114	78.5	222.93	291.72	14
2	18.2	24.4	114	78.5	231.85	310.83	14
3	17.5	24.1	113	78.5	222.93	307.01	12

According to Turkish standard (TEC 708 BC-1A, March 1996) three specimens did not meet the requirements. However tensile strength (220 MPa) was used and the following is the TEC 708 specifications;

Yield (MPa) = 220 MPa   Ult.(MPa) =340 MPa   Strain(%) = 18%

Figure 60. Specimens after tension test.

3.2.2.5 Ferroscan Test

The Ferroscan test is a very common test used in scanning and detecting the location of steel reinforcement bars in structural members. Basically it is a device consisting of a display monitor and scanner, with a 600 mm square board. The device is easy to use and gives useful information. This is the procedure for a Ferroscan Test:

1. Attach a 600 mm square board to the member.
2. The scanner has the capability of detecting 15 mm width, so 4 readings in both directions horizontally and vertically are required to cover a 600mm square board.
3. Data was saved to the Ferroscan automatically with identifying depth and distance between the steel reinforcement.

The equipment can read the reinforcement details to a maximum 100mm in depth. At any point deeper the devices loses analysing accuracy this is one disadvantage to the Ferroscan but on the whole it remains on the best ways to detect reinforcement details from an existing building as shown in (Figures 61 and 62).

Figure 61. Ferroscan device and typical sample of ferroscan of reinforcement details.

Figure 62. Reinforcement details scanning by Ferroscan.

3.2.3 Determination of Reinforcement Details Characteristics (Super Structure)

Characteristics with reinforcement details for sections of columns and beams for frame structural members determined via Ferroscan device shown in (Figure 63). Reinforcement details are shown below;

BEAMS; Beams are an important element of the structure. They have two essential functions for the structure's load bearing system [6], [9];

1. Firstly, when loads act vertically, beams can act to transfer vertical dead loads from slabs, if there is an existing load of walls of beams, they can act to transfer the load to columns or shear walls to which they are anchored.
2. Secondly, when loads act horizontally, beams can act to transfer the load resulting from earthquakes or wind to a vertical load bearing with slabs.

Beams are assessed by using destructive and non-destructive (Ferroscan) methods. Characterization of beams as given in the table below:

Table 7. Beams section characterizations.

Sections	**Dim.(cm)**	**Top**	**Bent-up**	**Bottom**	**Add. at Bottom**	**Stirrup**
Beam	40x60	2 Φ16	2Φ16	2 Φ16	2 Φ 16	Φ10/15

(cover = 3 cm, MPa, ***fy*** = 220 MPa ,unit weight of concrete = 25 kN/m3)

More information about the Ferroscan procedures of reading reinforcement details and reinforcement section details are shown in the following (Figures 63).

COLUMNS; Reinforced concrete columns are the most important elements of frame structure systems. The positioning of the columns are the most important elements of a RC structure, Therefore columns have to carry [6], [9];

1. Axial loads. 2. Bending moments. 3. Shear forces. 4. In some situation torsion.

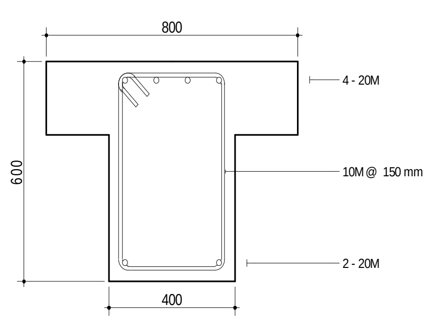
Columns are assessed with using destructive and non-destructive (ferroscan) methods, Characterization of beams is shown in Table 8. More information about the ferroscan procedures of reading reinforcement details and reinforcement are given in (Figure 69).

Table 8. Columns section characterizations

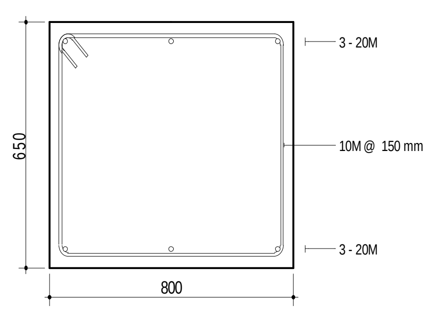
Sections	Dim.(cm)	Major	Minor	Middle layer 1	Middle layer 2	Stirrup
Column C1	65x100	4 Φ 20	3Φ20	2 Φ16	---------	Φ10/15
Column C2	80x65	3 Φ 20	3 Φ20	2 Φ16	---------	Φ10/15
Column C3	65x45	3 Φ 20	3 Φ20	2 Φ16	---------	Φ10/15
Column C4	35x65	2 Φ20	2 Φ20	2 Φ16	---------	Φ10/15
Column C5	45x65	2 Φ 20	2 Φ20	2 Φ16	---------	Φ10/15
Column C6	85x45	3 Φ 20	3 Φ20	2 Φ16	---------	Φ10/15
Column C7	50x35	2 Φ20	2 Φ20	2 Φ16	---------	Φ10/15
Column C8	35x50	2 Φ20	2 Φ20	2 Φ16	---------	Φ10/15

(cover = 3cm , ***fy*** = 220MPa, unit weight of concrete = 25 kN/m3)

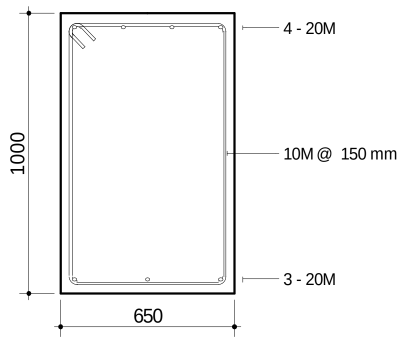
More information about the Ferroscan procedures of reading reinforcement details and reinforcement section details are shown in the following (Figures 63).



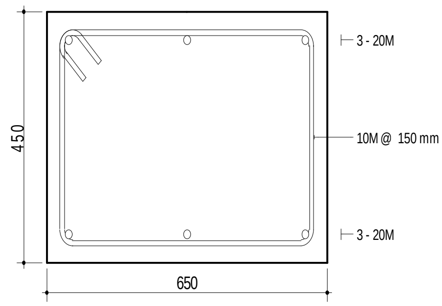
Interior Beam



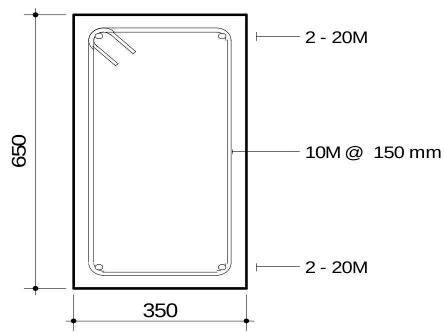
Column (C1)



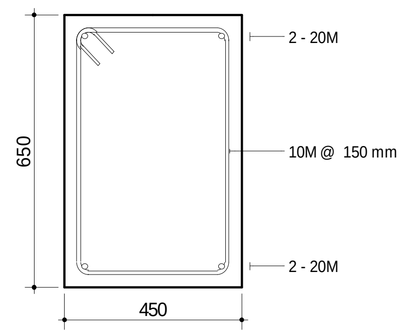
Column (C2)



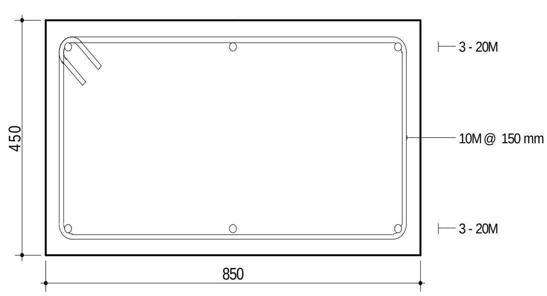
Column (C3)



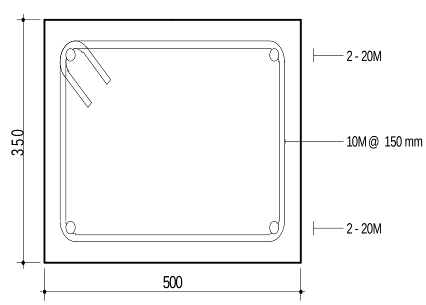
Column (C4)



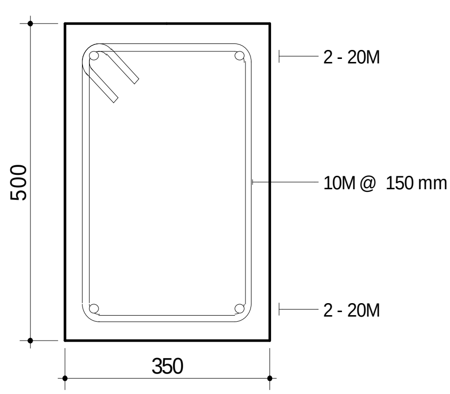
Column (C5)



Column (C6)



Column (C7)



Column (C8)

Figure 63. Reinforcement details for beams and columns section

3.2.4 Problems observed in the building

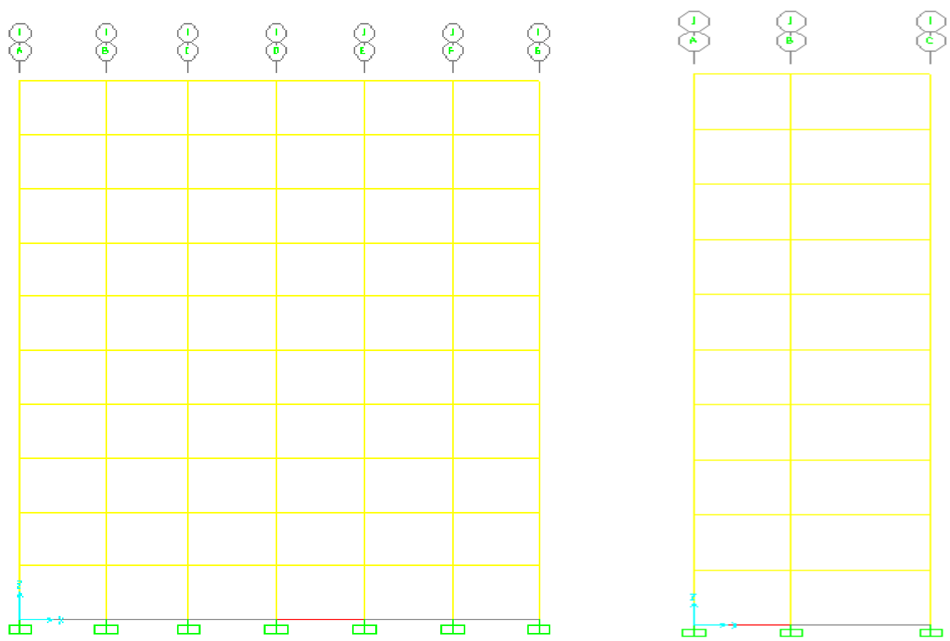
The main problem observed in the Seaside Hotel reinforced concrete building is corrosion. This is very likely to have a direct effect on the performance of the building especially during an earthquake. It will be the single most important reason for causing severe damage to columns, which are the main elements of the structure shown in (Figure 64).

Corrosion has many and varied effects on a structure. Formally, it causes a decrease in the cross-sectional area of steel bars, creates both internal and external cracks, reduces the strength of the concrete and brings about lateral displacement due to slippage [46].

Figure 64. Corrosion problems in a column

3.2.5 Building Modeling

In this study, properties of materials and structural members have been identified. The simplest way to analyze a building and perform a seismic assessment is to study on a frame model. In this research the structure frame consisting of beams and columns are modelled using a structural analysis program called CSI SAP2000 14.0.0 program. By using SAP2000 software, the geometry of the structure can be drawn, then properties and loads to the members of the structure assigned to completely define the model. For the two dimensional sectional analysis of beams and columns "Response-2000" program is used to calculate Moment-Curvature relationships for columns and beams sections in order to define hinge properties in the frame model as shown in (Figure 65).



A. X-direction frame models.

B. Y-direction frame models.

Figure 65. Frame models

Chapter 4

MODELING AND ASSESMENT

4.1 Introduction

In this study, a ten storey RC building named Seaside Hotel is evaluated according to FEMA440 in detail. This building was studied in order to establish the consistency of seismic analysis and design procedures as specified in the code. Each storey is 3.00 m in height with 217.36 m^2 floor area. This building has 437 load-bearing elements (210 columns and 227 beams). Each storey has approximately similar plans. All plans of the Seaside Hotel are given in Appendix A. Seismic assessment procedures were mentioned in Chapter 2.

In this chapter 2D seismic analysis of two frames from a case study will be used; the Seaside hotel RC existing building by each direction which is X-Direction and Y-Direction as shown in (Figure 66 and Figure 67). Linear and non-linear analysis calculation were provided by using the hotel's existing building properties and earthquake parameters, which are shown below in Table 4.1. Earthquake zone has been considered as Zone 2 to give a more realistic result [Can, 1997, Yücemen, 1997]. Existing building information level has been considered as a comprehensive level of data collections. The difference in the compressive strength of concrete on each floor was given in Chapter 3 and also provided in Appendix B. The reinforcement bars are tested and determined as S220 class for all members.

Two dimensional X-direction frame representing seismic assessment analysis for a length side with a total length of 24.7m as measured shown in (Figure 66). Two dimensional Y-direction frame representing seismic assessment analysis for a side with a total length of 8.80 m as measured shown in (Figure 67).

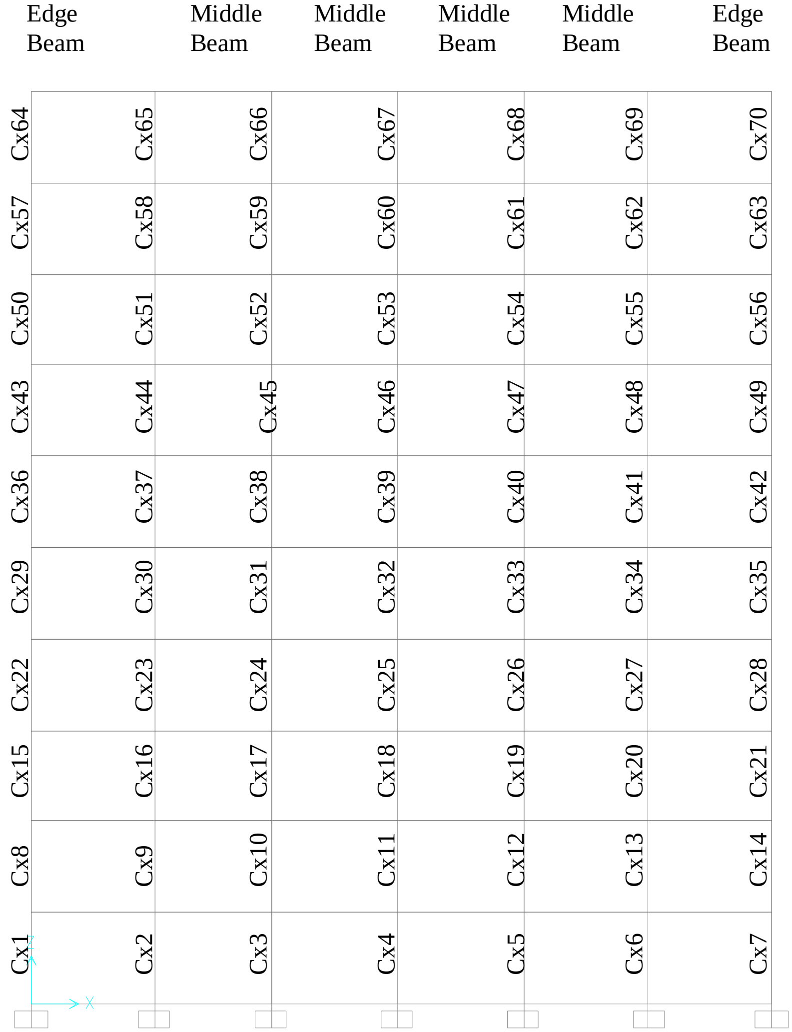


Figure 66. Two dimensional X-Direction frame model (Sea Side Hotel).

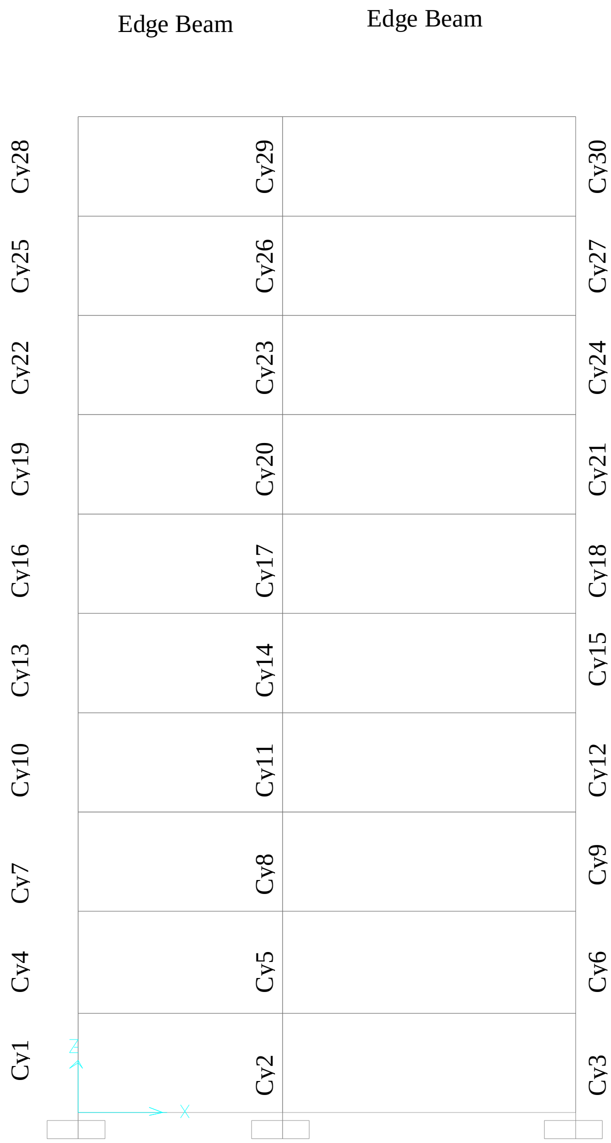


Figure 67. Two dimensional Y-Direction frame model (Sea Side Hotel).

Reinforcement details of the two frames model shown in the tables;

Table 9. Reinforcement details for X-Direction frame.

Section type	Elements Identifying
• C1	None
• C2	None
• C3	Cx2,Cx3,Cx4,Cx5,Cx6,Cx9,Cx10,Cx11,Cx12,Cx13,Cx16,Cx17, Cx18, Cx19, Cx20
• C4	None
• C5	Cx1,Cx7,Cx8,Cx14,Cx15,Cx21
• C6	None
C7	Cx23,Cx24,Cx25,Cx26,Cx27,Cx30,Cx31,Cx32,Cx33,Cx34, Cx37,Cx38, Cx39, Cx40, Cx41,Cx44,Cx45,Cx46,Cx47,Cx8, Cx51,Cx52, Cx53, Cx54, Cx55, Cx58, Cx59, Cx60, Cx61, Cx62, Cx65, Cx66,Cx67, Cx68, Cx69
• C8	Cx22,Cx28,Cx29,Cx35,Cx36,Cx42,Cx43,Cx49,Cx50,Cx56, Cx57,Cx63,Cx64,Cx70
• Beam	All Beams

Table 10. Reinforcement details for Y-Direction frame.

Section type	Elements Identifying
• C1	None
• C2	None
• C3	Cy1,Cy2,Cy3,Cy4,Cy5,Cy6,Cy7,Cy8,Cy9
• C4	None
• C5	None
• C6	None
• C7	Cy11,Cy12,Cy14,Cy15,Cy17,Cy18,Cy20,Cy21,Cy23,Cy24, Cy26, Cy27, Cy29, Cy30
• C8	Cy10,Cy13Cy16,Cy19,Cy22,Cy25,Cy28
• Beam	All Beams

Table 11 . Existing properties and code parameters of the building

Existing Building Properties	
Knowledge level	Comprehensive
Knowledge level Coefficients	1.0
Existing concrete compressive strength	Mentioned in Chapter 3
Existing steel reinforcement tensile strength	220 MPa
Earthquake Code Parameters	
Seismic Zone	2.0
Seismic Zone Factor (***A_o***)	0.3
Building Importance Factor (***I***)	1.0
Soil Class	Z4 (TA=0.2 T_B=0.9)
Live Load Participation Factor	0.3

Table 12. Storey equivalent lateral forces of Existing properties due to TEC2007.

Story	g_i	q_i	LLPC	w_i	H_i	$F_i(x)$	$F_i(y)$	X_m	Y_m
9.	1570	761	0.3	1798.3	30	65.53	65.53	12.35	4.4
8.	1570	761	0.3	1798.3	27	58.98	58.98	12.35	4.4
7.	1570	761	0.3	1798.3	24	52.43	52.43	12.35	4.4
6.	1570	761	0.3	1798.3	21	45.87	45.87	12.35	4.4
5.	1570	761	0.3	1798.3	18	39.32	39.32	12.35	4.4
4.	1570	761	0.3	1798.3	15	32.76	32.76	12.35	4.4
3.	1570	761	0.3	17983.3	12	26.21	26.21	12.35	4.4
2.	502.5	761	0.3	730.8	9	7.88	7.88	12.35	4.4
1.	502.5	761	0.3	730.8	6	5.32	5.32	12.35	4.4
Ground	96	761	0.3	324.3	3	1.18	1.18	12.35	4.4

Where;

g_i :The total dead load in the i'th storey of the structure [kN].

q_i : The total live load in the i'th storey of the structure [kN].

LLPC: Live load participation coefficient.

w_i :The weight of the i'th storey of the structure calculated using live load participation coefficient [kN].

H_i : The height of the i'th storey of the structure measured from the upper surface of the foundations (in structures with rigid concrete walls around the basement, the height of the i'th storey from the ground floor) [m].

$F_i(x)$:The equivalent seismic load acting on the i'th storey, in equivalent seismic load method (x direction) [kN].

$F_i(y)$: The equivalent seismic load acting on the i'th storey, in equivalent seismic load method (y direction) [kN].

X_m,Y_m :The coordinates of the center of gravity of the storey [m].

Geometric properties for the existing building of the Seaside hotel, section conception, hinges for frame members, and load replacement on frame have to be recognized by Sap2000.

The additional load for all members was identified 1.5 kN/m^2 with the gravity direction, 3.5 kN/m^2 was identified as live load in the direction of gravity. The self-weight of slabs was found for 20cm thickness as 5.0 kN/m^2 distributed in the gravity direction, 3 kN/m^2 was defined as the wall load for one meter square distributed along the beams over the direction of gravity.

Total loads identified according to ACI 318-08, calculated by yield line triangular method while considering all other loads, as shown in (Figure 68).

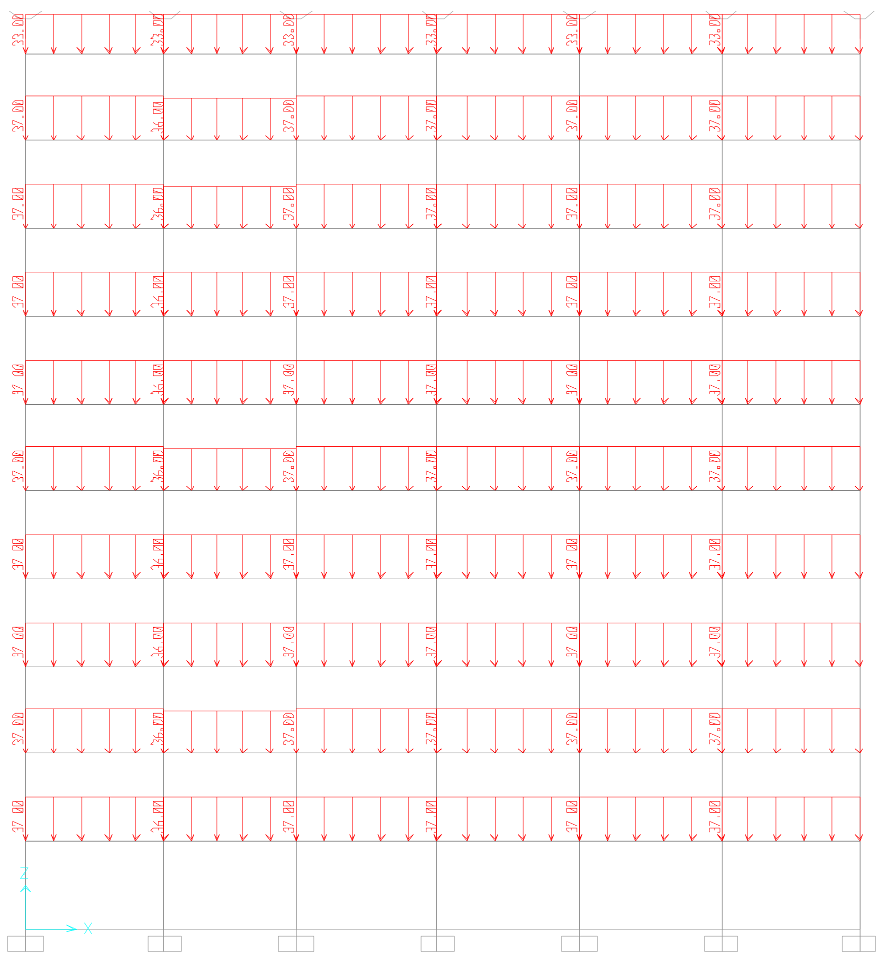


Figure 68.Total axial loads on the X-Direction frame model (Seaside Hotel).

After determining the gravity loads such as dead loads and live loads, all element details were identified to calculate the total axial load of the X-direction frame provided by Sap2000.

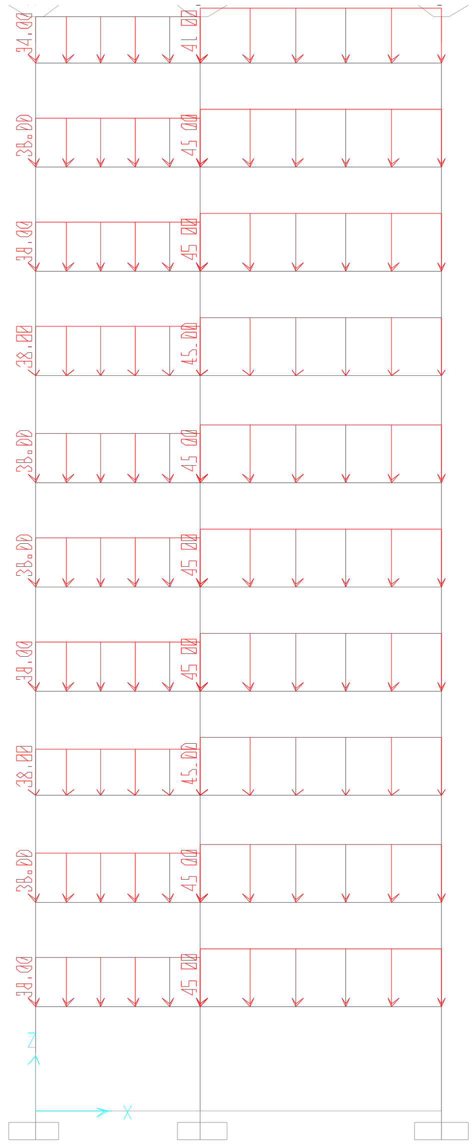


Figure 69.Total axial loads on the Y-Direction frame model (Seaside Hotel).

After determining gravity loads such as dead loads and live loads, all element details were identified to calculate the total axial load of the Y-direction frame provided by Sap2000.

4.2 Sectional Analysis of Reinforced Concrete Members

The sectional analysis of RC members is the approach taken to calculate the strength and deformation characteristics through moment-curvature relation for reinforced concrete members. In this study, software such as the "Response-2000" program was used. This allowed for a 2D Sectional analysis of beams and columns for determining the Moment-Curvature relation. The non-linear characteristics of cracked reinforced concrete sections were also considered. "Evan C. Bentz (2000)" states that the "Response-2000 program is authenticated to be the most immediately useful program". It can calculate the strength and deformation of reinforced concrete members subject to axial load, moment and shear [59].

To obtain a sectional analysis for columns and beams sections, sections were modeled as two dimensional sections in the analysis computer program "Response-2000"[4]. First the sections characteristics were defined, and the material properties and axial loads were applied as constant loads that were calculated from the "SAP2000" program. Reinforcement bars were defined in each section either by selecting the diameter or calculating the area of these bars. In order to perform a sectional analysis, several steps should be considered;

1- Creating columns and beams sections due to their characteristics.

2- Defining reinforcement bars for columns and beams sections.

3- Assigning axial loads that were taken from the "SAP2000" program for each member.

[4]Evan Bentz and M.P.Collins , (1996-1999), "Response 2000" 2D Sectional analysis of beams and columns, These programs were written over the years 1996-1999 by Evan Bentz, PhD candidate at the University of Toronto under the supervision of Professor M. P. Collins. Together they represent over 150,000 lines of C++.

4- Running a response analysis for each member and provide output data of moment-curvature curves to adjust plastic hinges properties.

The "Response-2000" provides an intensive sectional analysis for the full member behavior for a prismatic section which helps to identify the deformation of the 180 sections and allows engineers to calculate the deflection at any shear force until attaining to maximum deflection. This will allow engineers to observe the behavior of the full member. The moment-curvature relationships for all columns and beams sections are provided in Appendix C.

4.3 Seismic Performance Assessment Analysis before Strengthening

In this study, two methods of seismic analysis of an existing building were considered to verify the seismic demand. The procedure is described each individually as in the following;

4.3.1 Non-Linear Static Pushover Analysis Method

Nonlinear static pushover analysis has been performed on two dimensional models of two weak frames of the building. The results are described in the following sections.

4.3.1.1 Static Pushover Analysis

A static pushover analysis is one method of assessing the seismic demand of an existing building. It represents a static approximation of the structure response under dynamic loads such as earthquake loading by applying a vertical distribution of monotonically increasing lateral loads to a model which appropriates the material non-linearity of the structure [60].

The “SAP 2000” software was used in this study for providing the seismic performance assessment of the Seaside Hotel building. Using this method we created a 2-dimensional modeling of two frames from both directions of a case study building, identified all sections material characteristic including reinforcement details and assigning dead load pattern and live load pattern to verify its pushover curve.

"SAP 2000" provides an effective nonlinear static analysis preference which helps to provide structure mode failure as shown in (Figure 70 and 71). Plastic hinges can be inserted at both ends of a clear length of frame element. Each hinge represents concentrated post-yield behavior in one or more degrees of freedom. In this study, the properties of created hinges were taken from a moment-curvature relationship that was created by the "Response-2000" program for columns and beam sections.

Inel et. al. (2006) stated that "In pushover analysis, the behavior of the structure is characterized by a capacity curve that represents the relationship between the base shear force and the displacement of the roof. This is a very convenient representation in practice, and can be visualized easily by the engineer" [18]. For this reason in order to calculate any response by the load pattern, the load case should be identified while also taking into account the P-Δ effects.

In this study, the structure modeled by SAP2000 according to non-linear static analysis to carry out a pushover analysis (as laid down in /according to) the regulations of FEMA440 a two load pattern (Triangular and Uniform load pattern) that is shown in this section. The performance of nonlinear static pushover analysis was provided by the procedures below:

1- Model the frame elements that consist of beams and columns.

2- Identify section material and the characteristics of beams and columns.

3- Assign frame elements by identify sections of beams and columns.

4- Define and assign load patterns of dead and live loads. Rectangular and triangularly distributed shapes are used for the lateral load pattern in joints.

5- Define plastic hinge properties that will be taken from the moment-curvature relationship calculated by the "Response-2000" program for beams and column sections. The plastic hinge length is defined according to FEMA440.

7- Assign plastic hinges at both ends of each member of the frame model.

8- The pushover load case must be defined, nonlinear static analysis was identified, and P-Δ effects were taken into account.

9- The analysis is then run and output data is verified to emphasize the capacity curve and distinguish the performance of the structure.

After performing pushover analysis and the Roof Displacement – Base Shear curve (Capacity Curve) obtained, the target displacement for selected performance level has to be determine according FEMA 440. According to the TEC 2007 earthquake code for any storey, for all vertical members, immediate occupancy limits should be satisfied with 10% probability of being exceeded in 50 years. In this study the displacement was obtained for both frames of the structure according to FEMA440 and discussed in the following sections.

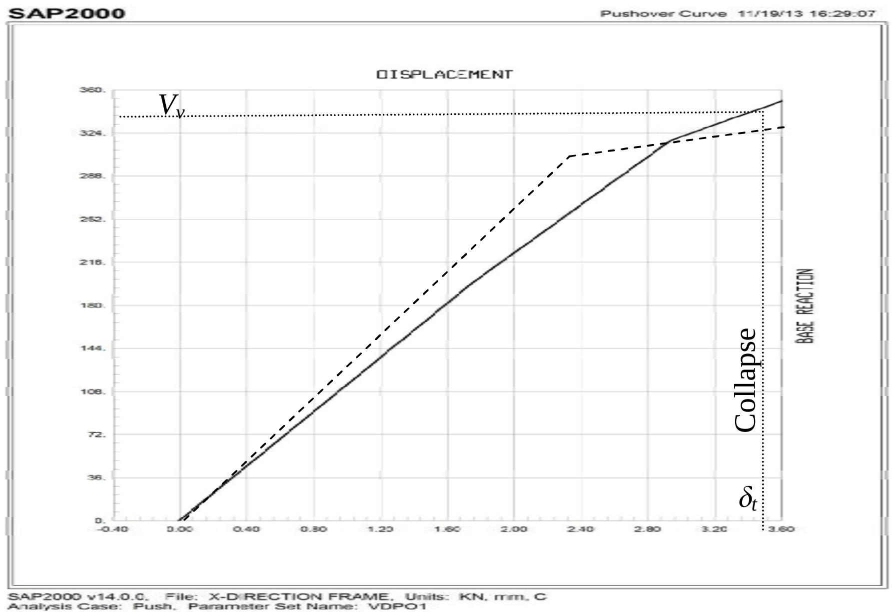


Figure 70. X-Direction frame pushover curve for existing building before strengthening.

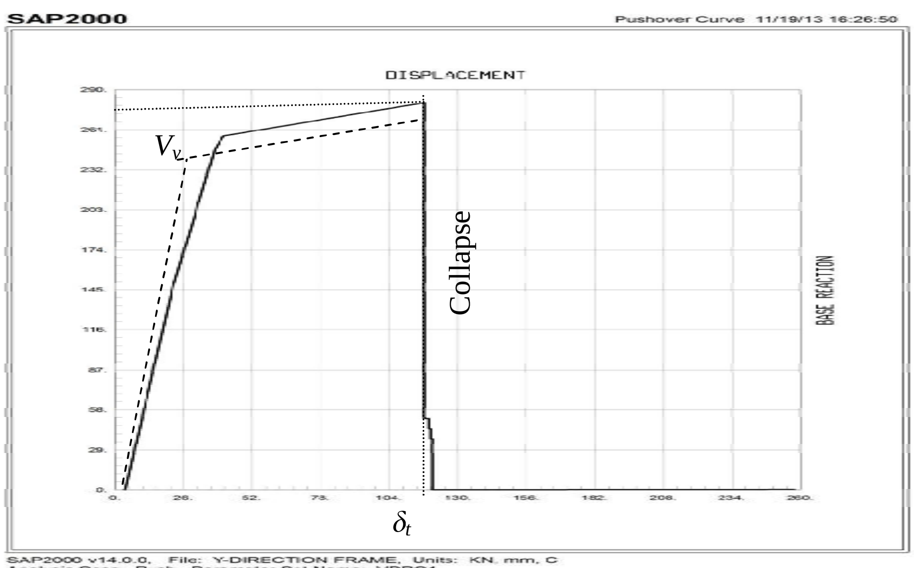


Figure 71. Y-Direction frame pushover curve for existing building before strengthening.

Target displacement relation with base shear force are calculated according to FEMA440 procedure described in (ASCE, 2000).Capacity curve (Pushover Curve) for x direction frame model of the building is shown in (Figure 70) where target displacement is calculated as δ_t = 10.20 mm and base shear force causing this displacement is equal to 300 kN, the capacity curve yielding will start when roof is displaced 2.00 mm and its first collapse occurs at 12 mm. Capacity curve (Pushover Curve) for y-direction frame model of the building is shown in (Figure 71) where target displacement is calculated as δ_t = 108.20 mm and base shear force causing this displacement is equal to 285 kN, the capacity curve yielding will start when roof is displaced 26.00 mm, and its first collapse occurs at 120 to 125 mm. At this point the building cannot resist any more loading and will be in mechanism.

4.3.1.2 Performance limit states

The member's plastic hinges' behavior is affected during the pushover analysis. The displacement function results obtained in tables in the following sections for each frame model, and the displacement of each concrete hinge for control joints causes change in limit states. The displacement of the structure beyond the inelastic limit, meaning the plastic hinges generated start to resist the moments and changes its performance levels until the collapse of the frames. FEMA440 coefficient method parameters are shown in Appendix B.

In the pushover steps taken for the frames, it can be observed that increasing the horizontal loads causes top displacement to increase. This displacement is defined as a ratio to the height of the building the roof displacement is observed with respect to the height of building. The base shear is increased up to a point where the first plastic hinge fails, at the point where the base sear decrease has lost stiffness so smaller horizontal loads can cause more roof displacement. The performance levels of the frames in the structure on their capacity curve are discussed in sections below.

4.3.1.2.1 X-Direction Frame

Table 13. Pushover Curve – (X-direction frame)

Step	Base Force (kN)	Displacement (mm)	Drift Ratio %
0	0	-0.007086	0.000
1	198.904	1.750492	0.003
2	317.93	2.93716	0.005
3	351.555	3.625041	0.053

Table 13 shows the pushover curve displacement and lateral forces results for the X-direction frame model via CSI SAP2000. The results obtained show the structure starts at 2.00mm top displacement. Step 2 is where the ground motion columns plastic hinges change their limit states to immediate occupancy. According to FEMA 440 this level of damage, structures can be repaired and used safely. After 2.93 mm displacement changes to 3.62 mm the frame will change its state to life safety. At this level of damage, structures can no longer be used because of the high risk of injury. In step 3 the limit states will change to 'collapse prevention' and after that will change states to 'collapse'. The performance levels are shown in Table 13 and (Figure 72).

Table 14. Limit states – (X-direction frame).

Limit staes	Start (mm)	End (mm)
Yield	2.00	-
IO	2.00	2.93
LS	2.93	3.62
CP	3.62	12.00
Collapse	12.00	14.00

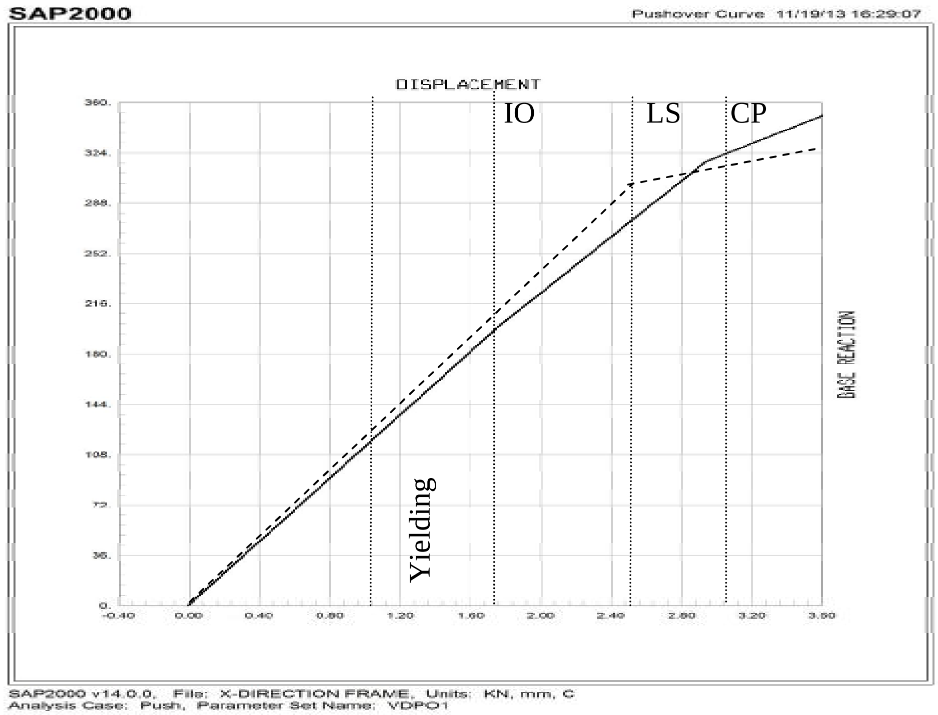


Figure 72. Structure performance limit states of frame model – (X-Direction).

Drift ratio % is obtained by the equation of displacement divided by height of the building.

4.3.1.2.2 Y-Direction Frame

Table 15. Pushover Curve - (Y-direction frame).

Step	Base Force (kN)	Displacement (mm)	Roof Drift %
0	0	3.829967	0.000
1	144.328	21.480865	0.007
2	246.772	37.903703	0.012
3	256.485	41.02782	0.013
4	265.832	70.451075	0.02
5	275.358	100.83069	0.03
6	280.518	117.49325	0.036
7	51.327	117.49579	0.039
8	51.329	118.98323	0.039
9	44.115	118.98577	0.039
10	44.116	119.65942	0.04
11	36.777	119.66196	0.04
12	36.778	120.45679	0.04
13	0.025	120.45933	0.04
14	0.072	145.85933	0.046
15	0.119	171.25933	0.056
16	0.165	196.65933	0.065
17	0.212	222.05933	0.074
18	0.259	247.45933	0.08
19	0.278	257.82997	0.086

Table 15 shows the pushover curve displacement and lateral forces results for the Y-direction frame model via CSI SAP2000. Obtained results the structure start at 26 mm top displacement, Step 4 is where the ground columns plastic hinges change their limit states to immediate occupancy, according to FEMA 440 this structure can be repaired and used safely. After 41.02 mm displacement changes to 70.45 mm the frame will change its state to life safety with displacement 119.65 mm, in this level of damage, structures cannot be used anymore because of high risk of injury. In step 13 will change to CP and after to C. The performance levels are shown in the Table 16 and (Figure 73).

Table 16. Limit states – (Y-direction frame).

Limit staes	Start (mm)	End (mm)
Yield	26.00	-
IO	26.00	41.02
LS	41.02	70.45
CP	70.45	119.65
Collapse	119.65	120.45

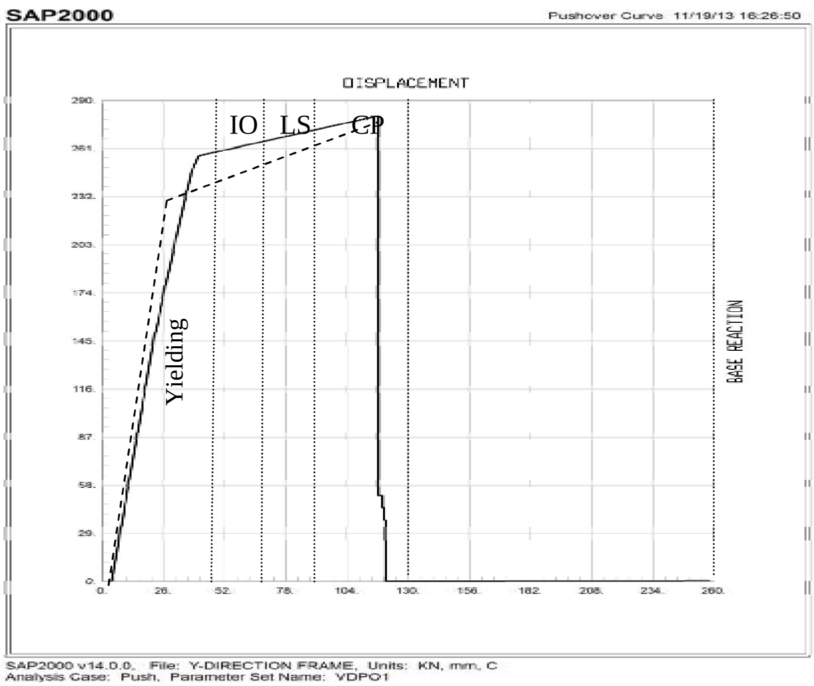


Figure 73. Structure performance limit states of frame model – (Y-Direction).

4.3.1.3 Damage Prediction

According to the pushover analysis obtained for each frame structure proposed by using (Lang, 2002), first plastic hinges where identified. It at this is the point where the structure starts to lose its stiffness, cracks and moderate damage to structural elements is expected to damage grade 2. Damage grade 3 represents a point in pushover analysis where the last plastic section is identified as causing the risk of the concrete cove falling and large cracks appearing. Damage grade 4 is where the structure causes the collapse of one or more elements and classified first plastic hinge failure during the pushover process. Finally, the collapse of the ground columns or complete collapse of structure is classified as damage grade 5. The following damage grades will be discussed for both frames in the following sections.

4.3.1.3.1 X-Direction Frame

Displacement demand (Δ_{top}) is calculated for frame model x-direction as 8.6 mm. This point is expected to be damage grade 4 for this side of the structure. According to classifications of damage grades by using (Lang,2000), the structure is expected to have very heavy damage with heavy structural damage, large cracks in structural elements, failure of concrete , collapsing concrete cover, rods buckling and the collapse of a few elements of the 4^{th} floor **.** As shown in table 17 are the limit states of damage grades with respect to top displacement and in (Figure 74 and 75) the formations of plastic hinges by SAP2000.

Table 17. Damage grade limits of X-direction frame.

Damages	**Start (mm)**	**End(mm)**
1	0	2.93
2	2.93	3.62
3	3.62	4.00
4	4.00	12.00
5	12.00	**-**

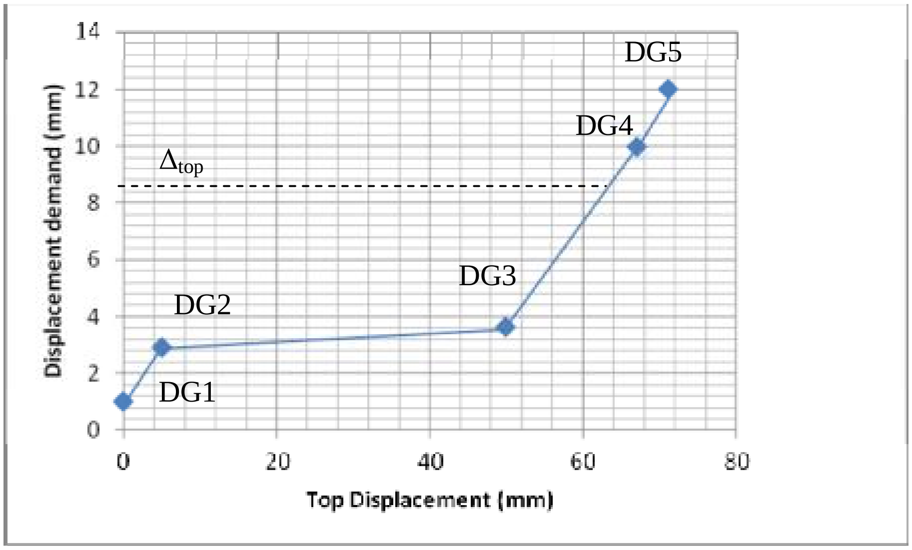


Figure 74. Damage grades of frame model by (Using Lang, 2000) – (X-Direction).

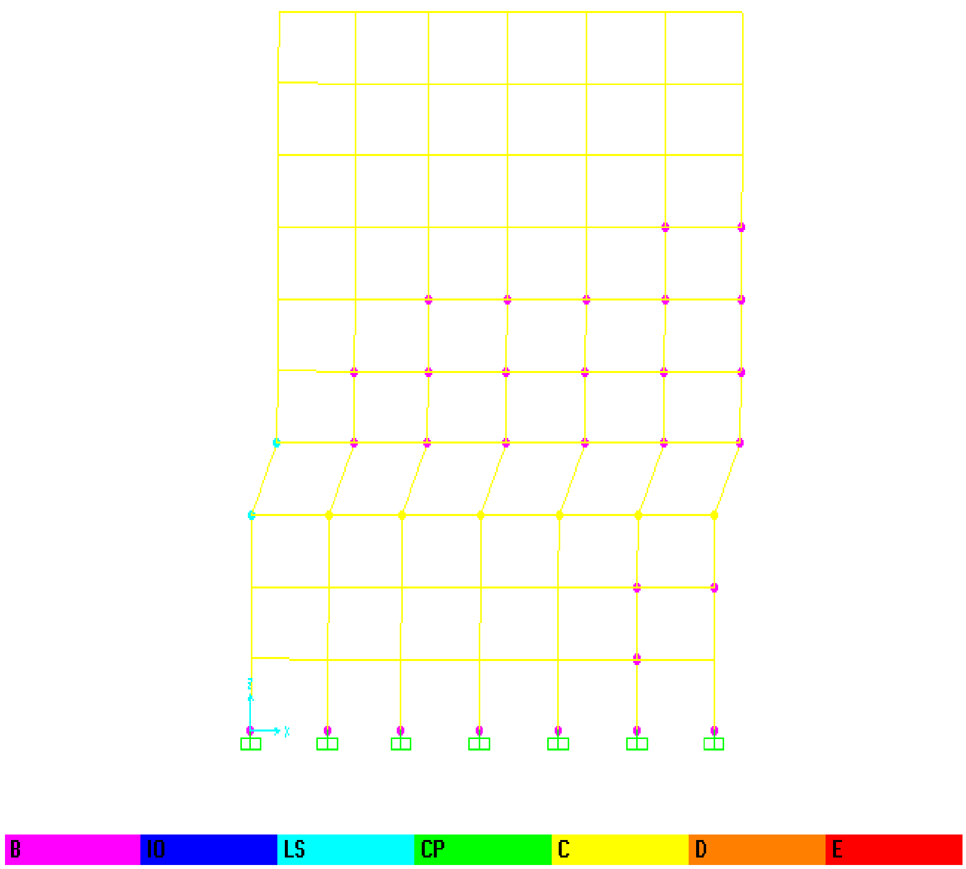


Figure 75. Plastic Hinges formation of frame model before strengthening by SAP 2000 – (X-Direction).

4.3.1.3.2 Y-Direction Frame

The displacement demand (Δ_{top}) is calculated for frame model x-direction as 52 mm. This point is expected to have a damage grade 4 for this side of structure. According to classifications of damage grades by using (lang,2000), the structure is expected to have very heavy damage with heavy structural damage, large cracks in structural elements, failure of concrete , collapsing concrete cover, rods buckling and the collapse of a few elements of the 4th floor. Shown in table 18 are the limit states of damages grades with respect to top displacement and in (Figure 76 and 77) the formations of plastic hinges by SAP2000.

Table 18. Damage grade limits of Y-direction frame.

Damages	Start (mm)	End(mm)
1	0	3.82
2	3.82	21.48
3	21.48	41.02
4	41.02	120.45
5	120.45	**-**

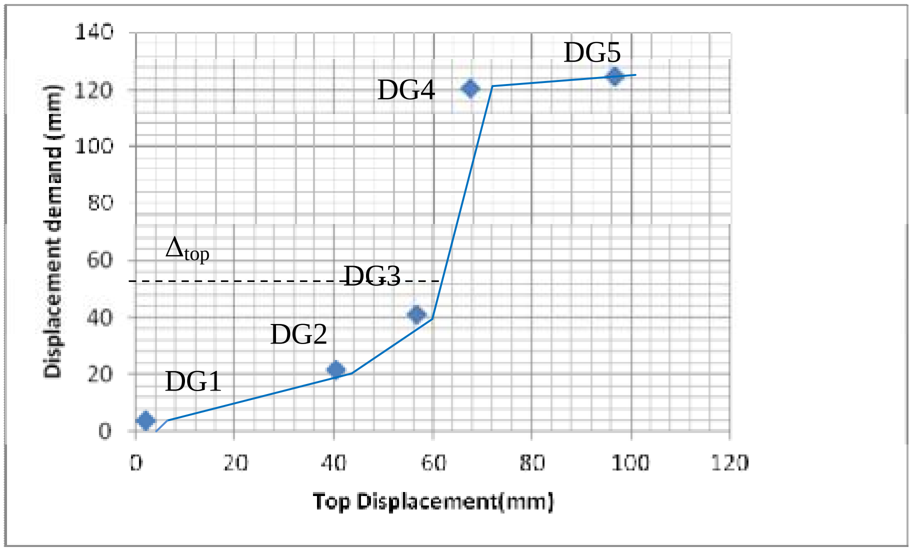


Figure 76.Damage grades of frame model by (Using Lang, 2000) – (Y-Direction).

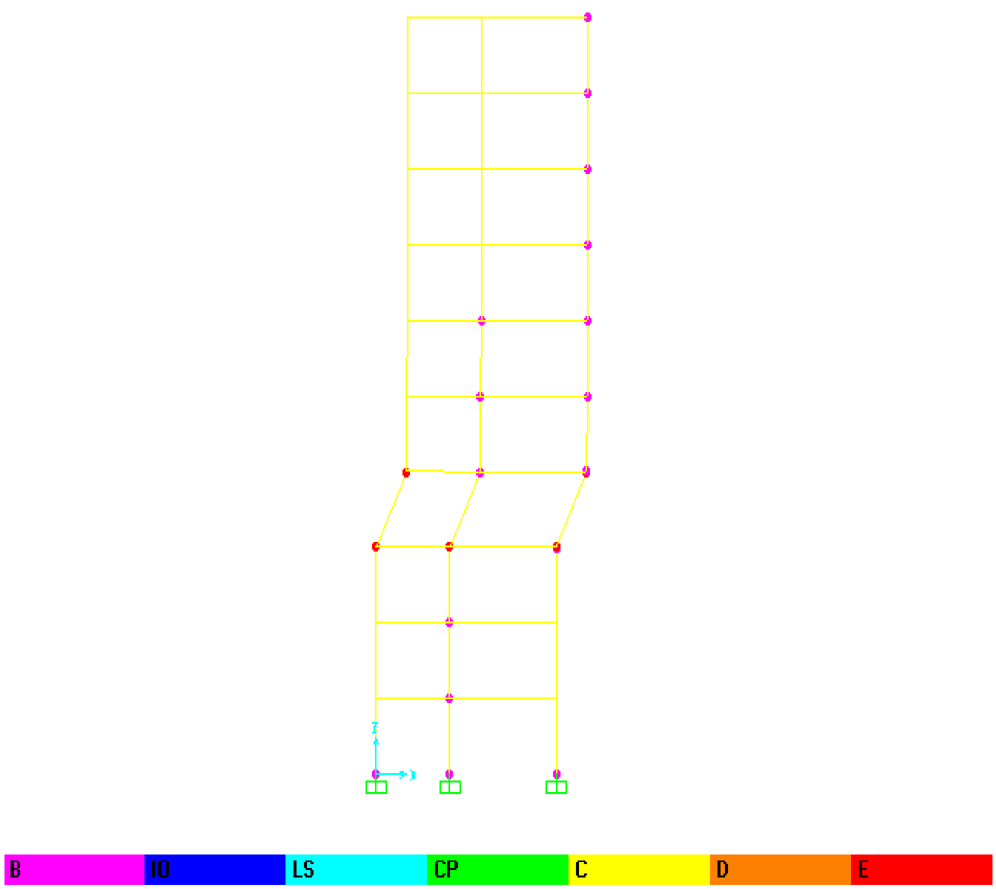


Figure 77.Plastic Hinges formation of frame model before strengthening by SAP 2000 – (Y-Direction).

According to FEMA440, the residential buildings in the hotel are expected to satisfy LS performance levels under a designed earthquake. The existing residential building is far from satisfying the expected performance levels. The performance level of the building is determined as collapse prevention (CP) in both directions on the building from x-direction and y-direction. The building is associated with the existence of a weak column–strong beam mechanism and an insufficient amount of transverse reinforcement. On the 4th floor the columns dimension change to the smallest dimensions.

4.3.2 Non-Linear Dynamic Time History Analysis Method

The non-linear time history analysis (NDTHA) method is one of the most powerful dynamic analysis methods for assessment of an existing structure; this method depends on the structure size and time-steps requested for assessment.

Generally this method is the most correct way for obtaining response for structures to earthquake loading (Pinho, 2007). In this study, 20 different ground accelerations were used. These were obtained in PEER website and SAP2000 to be used to investigate the behavior of the structure. Time history analysis method records the displacement of any point on the structure caused by earthquake recorded accelerations data during the chosen time period. For obtaining the accurate results the frames were selected in a critical condition from both sides of the structure, in x-axis and y-axis then subjected to 20 different ground accelerations that will be selected and scaled in the next section. Interpreting the results for finding acceleration-displacement relationship for each time series is difficult. Hence, the same control joint from static pushover analysis was selected in order to compare the displacement caused by ground acceleration records with displacement caused from accelerating lateral loads and determining the limit state capacity of the structure with respect to displacement function.

In order to find a probabilistic risk analysis on structure in this study, spectral displacement of control joint is constructed with respect to spectral acceleration causing it. The structure limit states for yielding, immediate occupancy, life safety and collapse prevention were identified. Non-linear static pushover analyses on the same frames for the control joint were used to find all the features and percentages of each performance level.

The acceleration needed the structure against top to cause those top displacement and plotting it against the frequency percentage of points located in any limit state presenting vulnerability of structure allows engineer to observe and predict the vulnerability percentage of building subjected to ground accelerations. The performance of nonlinear dynamic time history analysis was provided by the following procedures;

1. Time history function should be defined; the scaled ground motion acceleration (Accelerograms) will be uploaded as time history function in SAP2000.
2. Modal load case shall be set to use Ritz vectors and load type as acceleration.
3. Defining time history load case, using the defined time series function with load type *Ux* of acceleration with scale factor of 9.81 to convert the g unit to meters
4. Number of output time steps and time step size shall be specified, the more output time will simply provide more details on output. We used 20 steps per second for duration of 10 seconds.
5. Running analysis and using output data to investigate the performance level of the structure according to its maximum displacement and constructing probability of being at a different performance level with respect to structure's top displacement.
6. Drawing a probabilistic risk curve that considers performance level.
7. Obtain vulnerability percentage of structure stability.

4.3.2.1 Specification of design acceleration spectrum

The existing reinforced building is located in Famagusta in the Turkish Republic of Northern Cyprus was designed and constructed in the early 1970's. However the Turkish earthquake code has been considered. The "Part III-Earthquake disaster prevention" of 2007 earthquake code defines the spectral acceleration coefficient and design acceleration spectrum.

Design acceleration spectrum is equal to spectral acceleration coefficient times the acceleration of gravity, g.

$$A(T) = A_o I \, S(T) \tag{4.1}$$

$$Sae(T) = A(T) \, g \tag{4.2}$$

Ao = Effective Ground Acceleration Coefficient obtained in Table 19,

I = Building Importance Factor obtained in Table 20,

S(T) = Spectrum Coefficient , calculated according to building natural period and local site conditions classified in Table 21.

$$S(T)=1+1.5\,\frac{T}{TA} \qquad (0 \le T \le TA) \tag{4.3}$$

$$S(T)=2.5 \qquad (TA \le T \le TB) \tag{4.4}$$

$$S(T)=2.5\,\left(\frac{TB}{T}\right)^{0.8} \qquad (0 \le T \le TA) \tag{4.5}$$

Table 19. Effective ground acceleration coefficient (Ao), (TEC, 2007).

Seismic Zone	A_o
1	0.40
2	0.30
3	0.20
4	0.10

Table 20. Building importance factor (i), (TEC, 2007).

Purpose of Occupancy or Type of Building	*Importance Factor (I)*
1. Buildings required to be utilized after the earthquake and buildings containing hazardous materials **a)** Buildings required to be utilized immediately after the earthquake (Hospitals, dispensaries, health wards, fire fighting buildings and facilities, PTT and other telecommunication facilities, transportation stations and terminals, power generation and distribution facilities; governorate, county and municipality administration buildings, first aid and emergency planning stations) **b)** Buildings containing or storing toxic, explosive and flammable materials, etc.	1.5
2. Intensively and long-term occupied buildings and buildings preserving valuable goods **a)** Schools, other educational buildings and facilities, dormitories and hostels, military barracks, prisons, etc. **b)** Museums	1.4
3. Intensively but short-term occupied buildings Sport facilities, cinema, theatre and concert halls, etc.	1.2
4. Other buildings Buildings other than above defined buildings. (Residential and office buildings, hotels, building-like industrial structures, etc.)	1.0

Table 21 . Spectrum characteristic periods (TA, TB), (TEC, 2007).

Local Site Class according to Table 6.2	T_A *(second)*	T_B *(second)*
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

Turkish earthquake design code has specified a limit for acceleration spectrum that "spectral acceleration coefficient corresponding to so obtained acceleration spectrum ordinates shall in no case be less than those determined by equation 4.3" as *S(T)*, (Aydinoglu, 2007). Local site class is specified as *Z4* (T_A= 0.2, T_B= 0.9) for site of "Seaside Hotel" According previous research and investigations of Cyprus (Cagnan and Tanircan, 2009). The buildings on the site fit the 4th category for importance factor for hotels and other buildings according to the Turkish earthquake code and obtained as 1.0. Researches done on seismicity of Cyprus showed that the island is located on seismic zone 2 (Cagnan and Tanircan, 2009), with effective ground acceleration coefficient specified as 0.3 from the Turkish earthquake code (Aydinoglu, 2007). There is no official seismic hazard assessment to Northern Cyprus but previous studies (Cagnan and Tanircan, 2009) shows that there are no records of major destructive earthquakes in the history of Cyprus, and studies show that most of the destructive earthquake were within a range of 6 to 7 in magnitude, zero to 40 km distance from epicenter and zero to 40 km depth to the surface. These specifications are used as search criteria to determine the best matching earthquake time series for Cyprus with respect to design response spectrum found for buildings according to local site conditions.

Spectral acceleration coefficient has been obtained based on local site class specification for *Z4*, seismic zone 2 (Cagnan and Tanircan, 2009), building importance factor for hotels and applicable distinctive periods specified for time period of ten seconds and the result is plotted as Spectral coefficient versus time period shown in (Figure 78) and calculated Spectral acceleration versus time periods shown in (Figure 79).

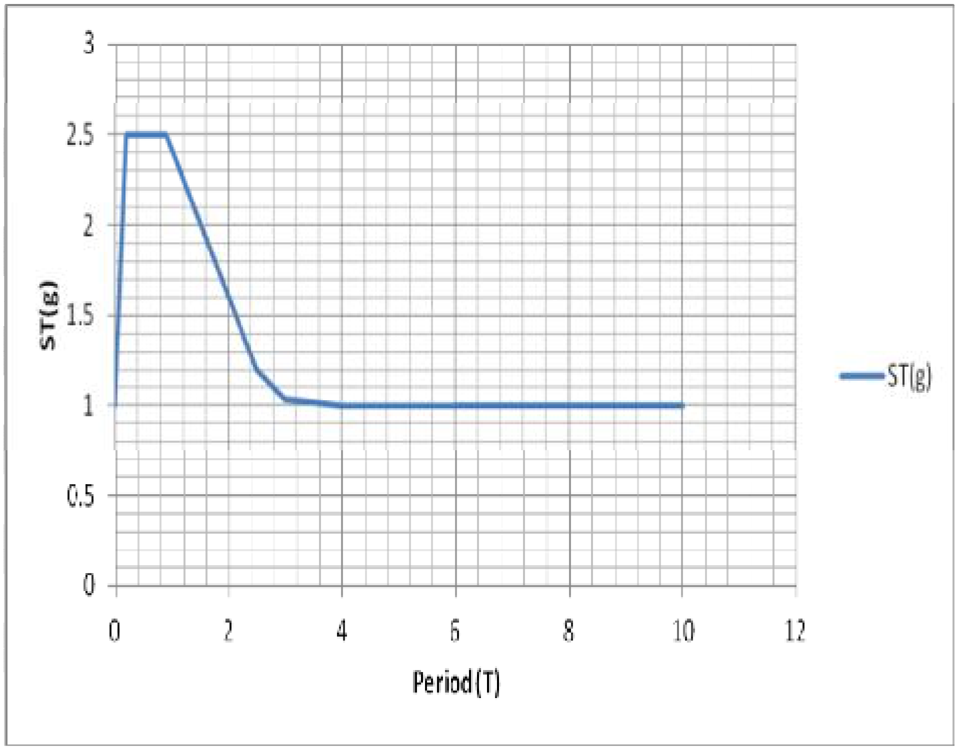


Figure 78. Design spectral acceleration coefficients-time period spectrum.

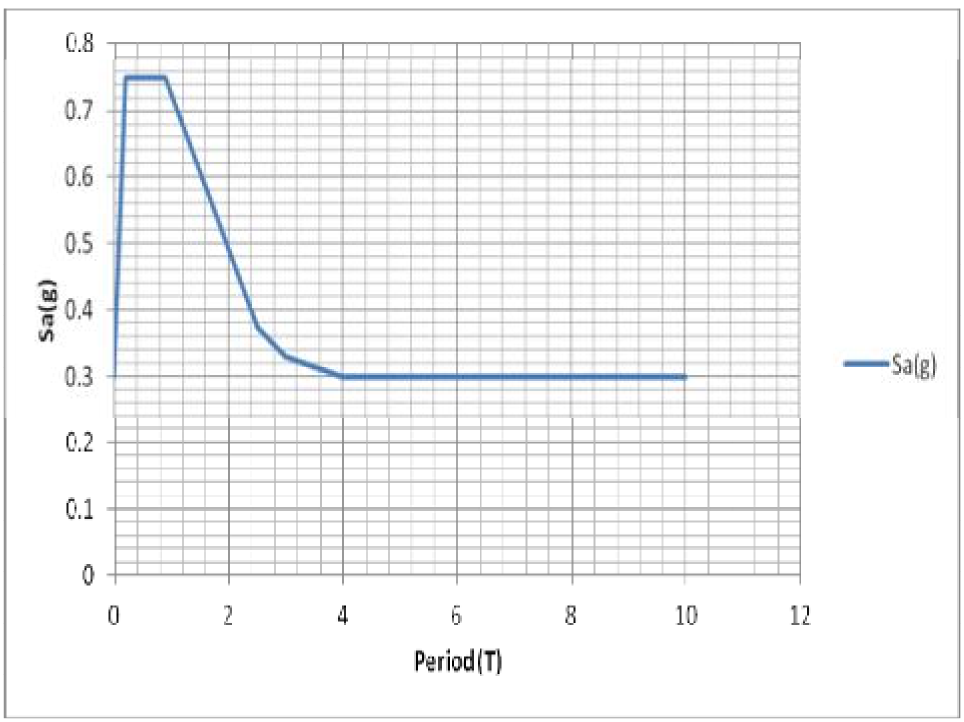


Figure 79. Spectral acceleration-time period spectrum obtained for seismic zone 2.

4.3.2.2 Selection of ground motion accelerations

The Pacific Earthquake Engineering Research Center (PEER) at the University of Berkley, California has all earthquake NGA strong motion database record (http://peer.berkeley.edu/peer_ground_motion_database,2010 BETA version). It also has a web application to search and scale the selected ground accelerations, Accelerograms (PEER, 2010). In this study PEER ground motion acceleration database web application has been used to select the time series and scaling them to specified design acceleration spectra [63].

PEER database has a three step process to form the acceleration response spectrum (PEER, 2010);

1- Specification of target design spectrum
2- Specification of limits for time series
3- Searching database, selecting and recording the acceleration data

In order to develop the target design spectrum, a user defined spectrum option has been used which allows the user to upload the calculated design response spectrum according to Earthquake code to the web application. This tool will plot the user defined target design response spectrum that shall be calculated according to the Turkish earthquake code (Aydinoglu, 2007) and from there the user is able to go to the next step and define the limits for search criteria.

4.3.2.2.1 Time series scaling

PEER ground motion database web application uses mean squared error (MSE) of variation among the record's spectral acceleration and the user defined target spectrum within the period to determine the best match. MSE is calculated using logarithms of spectral acceleration and period, and from there this web application searches the database according to the user defined specifications and then sorts the records in an increasing order of MSE with the records having the lowest MSE to match the target spectrum. This tool also provides linear scaling of time series to bring the closest match to the target spectrum (PEER, 2011), [63].

4.3.2.2.2 Mean square error (MSE)

MSE among the response spectrum of the record and defined target spectrum is calculated base on variation in natural logarithm of spectral accelerations. Time period between 0.01 to 10 second is divided into 301 points including the end points. Mean squared error is calculated using following equation (PEER, 2011);

$$MSE = \frac{\sum_i w(T_i)\{ln[Sa\ taregt\ (T_i) - ln[f\ Sa\ record\ (T_i)]]\}^2}{\sum_i w(T_i)} \tag{4.6}$$

Parameter $w(T_t)$ is defined as weight function which let user assign comparative weights to any points of period range; this function should be taken $w(T_t) = 1$, unless user chooses to highlight a match above an expansive period variation.

Parameter "*f*" is defined as the linear scale factor which will be applied to response spectrum of record, application of this scale factor is for minimization MSE to give the best equivalent spectral shape for response spectrum of records with respect to defined target spectrum. Scale factor is calculated using following equation (PEER, 2010);

$$\ln f = \frac{\sum_i w(Ti)\left(\frac{Sa\,target\,(Ti)}{Sa\,record\,(Ti)}\right)}{\sum_i w(Ti)} \tag{4.7}$$

After the calculation of MSE and minimizing it using the calculated scale factor time series records are selected considering the best matching horizontal components. These are normal and parallel horizontal components where the geometric mean of acceleration is given in following equation (PEER,2010);

$$S_a\,Gm = \sqrt{Sa\,fn + Sa\,fp} \tag{4.8}$$

Or,

$$\boldsymbol{ln\ Sa\ Gm} = \frac{(\ln Sa\ fn + \ln Sa\ fp)}{2} \tag{4.9}$$

4.3.2.2.3 Search criteria specification

The user is able to specify different limits to find the suitable records for study. These may consist of: type of faulting which is selected as Strike-Slip, distance range which is selected as 0 to 40 km to epicenter, duration range which is assumed to be 10 seconds, depth of earthquake which is selected to be from 0 to 40 km to the surface and earthquake magnitude that is limited up to 7 Richter for the purpose of this study. However, after scaling the time series, the magnitude will not affect the results since the accelerations are scaled to the defined design response spectrum respectively.

A total of 20 Earthquake records were searched scaled and recorded to be used for this study, (Figure 79) shows the geometric mean and specified target spectrum plotted by PEER ground motion database web application, and these records are shown in (Appendix E).

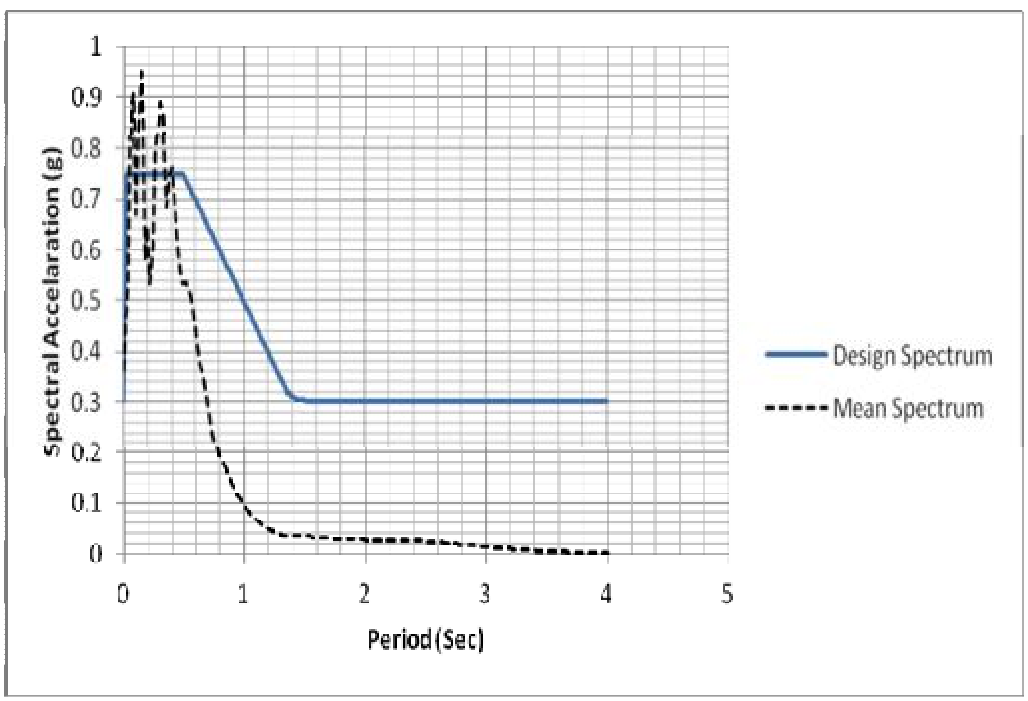


Figure 80. Target response spectrum and geometric mean spectrum (PEER, 2010).

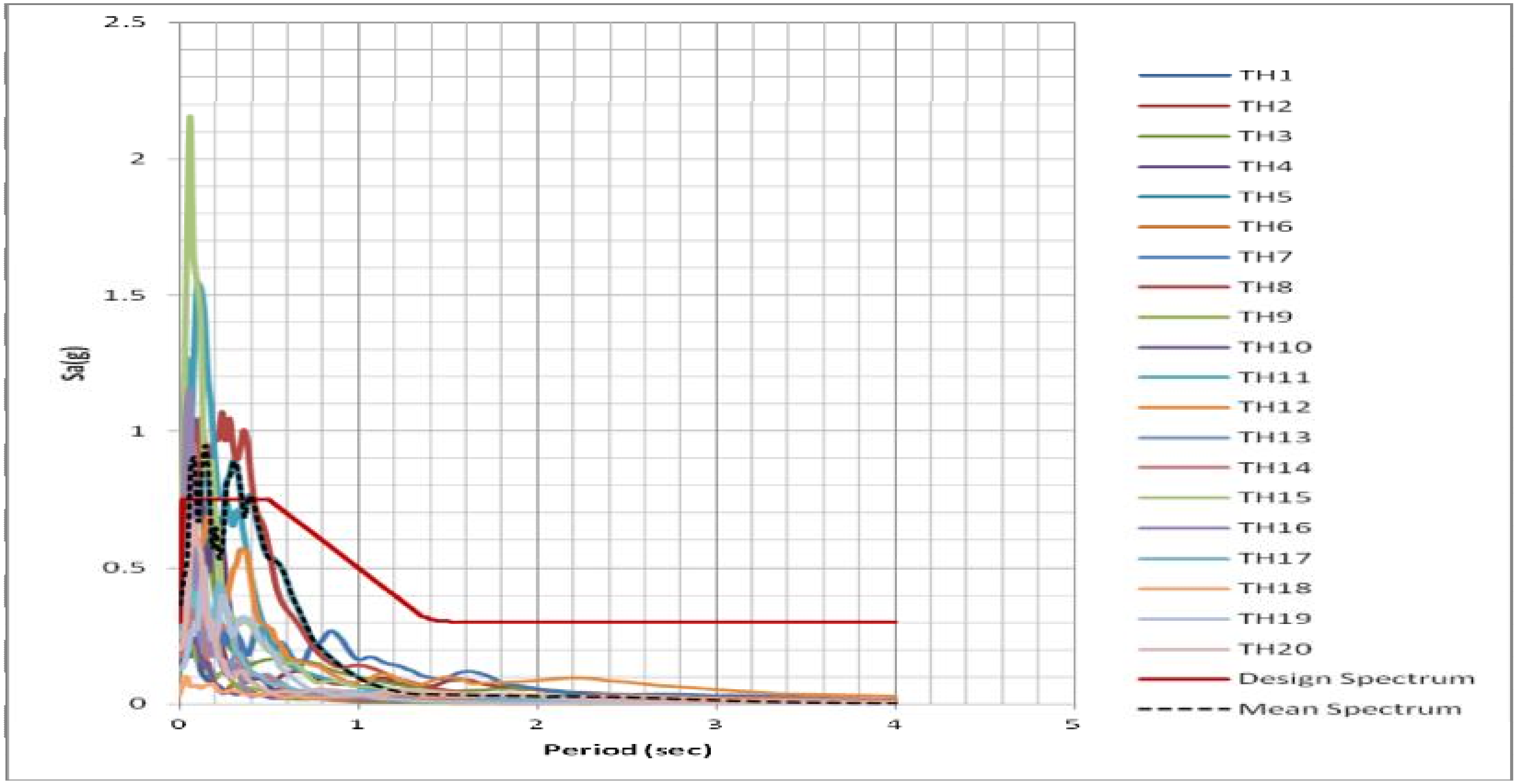


Figure 81. Target response spectrum and Geometric mean spectrum with spectrum of each time history ground motion records.

Name	MSE	Scale Factor	Event	Station	Year	Magnitude	Mechanisim	Fualt Distance	PGA(g)
TH1	0.3140	10.8413	Park Field	Cholame - Shandon Array #5	1966	6.19	Strike-Slip	9.6	0.6470
TH2	0.0687	2.6382	Imperial Valley-06	El Centro Array #4	1979	6.53	Strike-Slip	7	0.4741
TH3	0.0703	6.6741	Imperial Valley-06	El Centro Array #5	1979	6.53	Strike-Slip	4	0.5122
TH4	0.1677	4.7040	Victoria-Mexio	Cerro Prieto	1980	6.53	Strike-Slip	14.4	0.4950
TH5	0.0869	2.7845	Wesmorland	Westmorland Fire Sta	1981	5.90	Strike-Slip	6.5	0.4788
TH6	0.3016	16.8445	Morgan Hill	Morgan Hill	1984	6.19	Strike-Slip	13.7	0.5485
TH7	0.1464	2.6054	Superstition Hills-02	Wildlife Liquef. Array	1987	6.54	Strike-Slip	23.9	0.5476
TH8	0.1332	2.5265	Superstition Hills-02	Parachute Test Site	1987	6.54	Strike-Slip	0.9	0.5226
TH9	0.0671	5.2387	Landers	Amboy	1992	7.28	Strike-Slip	69.2	0.7231
TH10	0.0837	2.7455	Landers	Desert Hot Springs	1992	7.28	Strike-Slip	21.8	0.6087
TH11	0.1118	6.7796	Kobe-Japan	Takarazuka	1995	6.90	Strike-Slip	0.3	0.4981
TH12	0.0892	1.7984	Kocaeli-Turkey	Ambarli	1999	7.51	Strike-Slip	69.6	0.5080
TH13	0.0403	1.8289	Kocaeli-Turkey	Arcelik	1999	7.51	Strike-Slip	13.5	0.5098
TH14	0.2194	3.2088	Kocaeli-Turkey	Cekmece	1999	7.51	Strike-Slip	66.7	0.4887
TH15	0.0700	16.3662	Duzce-Turkey	Bolu	1999	7.14	Strike-Slip	12	0.6735
TH16	0.2197	7.6520	Duzce-Turkey	Lamont 375	1999	7.14	Strike-Slip	3.9	0.8761
TH17	0.4343	4.8329	Hector Mine	Hector	1999	7.13	Strike-Slip	11.7	0.6601
TH18	0.0523	1.6906	Denali-Alaska	TAPS Pump Station #11	2002	7.90	Strike-Slip	126.4	0.5539
TH19	0.0403	8.8293	Chi-Chi-Taiwan-04	CHY035	1999	6.20	Strike-Slip	25.1	0.4964
TH20	0.0564	12.0436	Chi-Chi-Taiwan-04	CHY028	1999	6.20	Strike-Slip	17.7	0.5495

Table 22. MSE and SF for Ground motion records and earthquake specification.

4.3.2.3 Time History Analysis Results

The maximum top displacement and spectral acceleration obtained during the application of each earthquake ground motion to structure, shown in below sections.

4.3.2.3.1 Frame Model (X-Direction) of Building

The maximum displacement due to applied time history records for frame model x-direction is shown in Table 23. This shows the behavior of control nodes in term of (S_a-S_d) relationship where the structure is responding elastically to the applied time series.

Table 23. Spectral displacement & spectral acceleration (x-direction frame).

GM	**S_a(g)**	**S_d (mm)**
TH1	0.05	6.68
TH2	0.03	4.01
TH3	0.03	4.57
TH4	0.04	5.32
TH5	0.05	6.53
TH6	0.03	3.51
TH7	0.07	9.58
TH8	0.05	7.18
TH9	0.04	5.64
TH10	0.06	8.39
TH11	0.04	5.46
TH12	0.02	3.20
TH13	0.07	9.38
TH14	0.07	9.90
TH15	0.03	4.68
TH16	0.02	3.15
TH17	0.03	4.48
TH18	0.05	6.66
TH19	0.04	5.15
TH20	0.04	5.68
	Mean (S_d)	6.20
	Standard Deviation(SD)	2.20
	Mean (S_d) ± SD	(8.41, 3.99)

According to selected earthqaukes results obtined for x-direction frame as following; The structure in x- direction side responded 10% elastically, 15% in immediate occupancy level, 15% in life safety level and 60% of collapse level.involving that this direction of the structure is 40% in accepted criteria and 60% will be unstable. Figure 88 showns probability increase of x-direction frame from the case study buidling to be not in performance level due to increase displacement caused by selected time histories applied to the structure with considering statistical approach “lognormal distrbution”. Target dispalcement obtained for x-direction frame is 10.20 mm by pushover analysis, at this displacment structure has probality of 30% to be in IO level, 40% to be in LS level and 65% to be in collapse level according to time history analysis results.

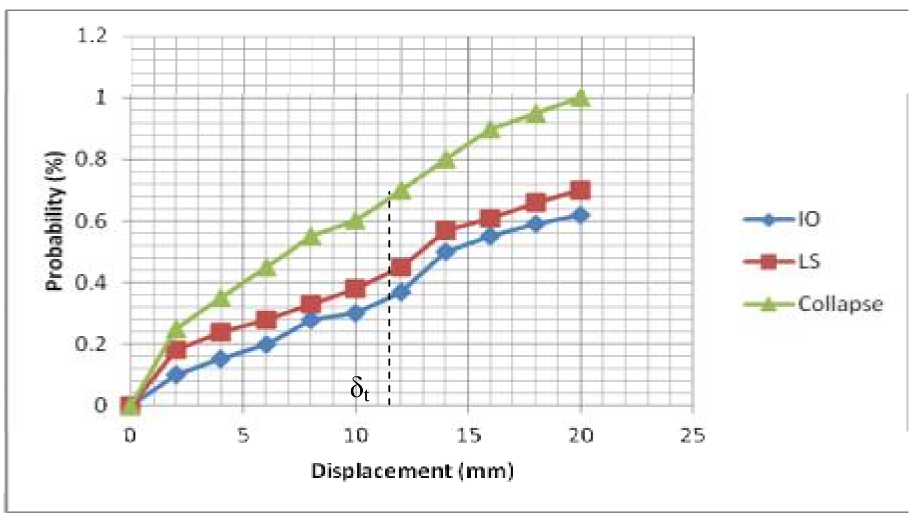


Figure 82. Probability of structure performance in different levels -(X-Direction).

The structure according to x-direction analysis results will suffer very heavy damage , there will be collapse in a few elemnts of the building or part of the building with a serious risk to life safety. In this particular case the structure’s critical points must be identified and strengthed to decrease the risk to life and collapse and in order to make the building secure or occupancy after the earthquake.

4.3.2.3.2 Frame Model (Y-Direction) of Building

Maximum displacement due to applied time history records for frame model x-direction is shown in Table 24. This shows the behavior of control nodes in terms of (S_a-S_d) relationship where the structure is responding elastically to the applied time series.

Table 24. Spectral displacement & spectral acceleration (y-direction frame).

GM	**S_a(g)**	**S_d (mm)**
TH1	0.07	22.40
TH2	0.51	71.20
TH3	0.72	100.01
TH4	0.73	101.20
TH5	0.48	66.90
TH6	0.47	65.99
TH7	0.30	41.15
TH8	0.52	72.98
TH9	0.17	24.20
TH10	0.26	36.00
TH11	0.40	55.35
TH12	0.53	74.34
TH13	0.12	16.33
TH14	0.08	11.30
TH15	1.44	201.01
TH16	0.10	14.13
TH17	0.56	78.19
TH18	0.05	7.15
TH19	0.15	21.3
TH20	0.17	24.13
	Mean (S_d)	55.26
	Standard Deviation(SD)	45.45
	Mean (S_d) ± SD	(100.71, 9.80)

According to selected earthqaukes results obtined for y-direction frame as following; The structure in y- direction side responded 5% elastically,15% in immediate occupancy level, 15% in life safety level and 65% of collapse level.involving that this direction of the structure is 30% in accepted criteria and 70% will be unstable.Figure 89 shows probability increase of y-direction frame from the case study buidling to be not in performance level due to increase displacement caused by selected time histories applied to the structure with considering statistical approach “lognormal distrbution”.

Target dispalcement obtained for y-direction frame is 108.20 mm by pushover analysis, at this displacment structure has probality of 45% to be in IO level, 55% to be in LS level and 75 % to be in collapse level according to time history analysis results.

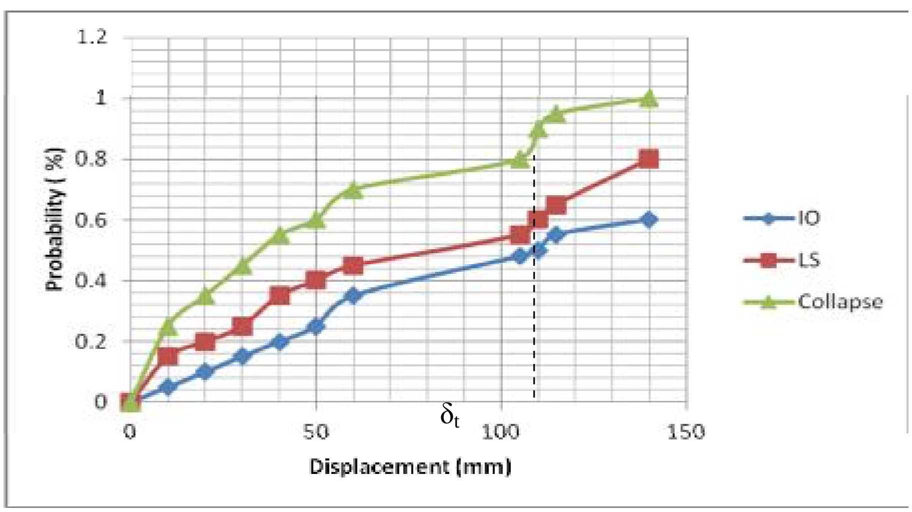


Figure 83. Probability of structure performance in different levels -(Y-Direction).

The structure in y-direction analysis results will suffer very heavy damage, there will be total collapse of a few elements of the building, and a risk to life in part of the building. In this case, the critical points of the structure need to be identifed and strengthened to reduce the risk to life and total collapse and also to allow for the building to be resecured for occupancy after the earthquake.

4.3.2.4 Comparison of Seismic Analysis Methods

By comparing the damage level obtained with the damage predicted using both the Non-linear static pushover analysis and Non-linear time history dynamic analysis methods, both procedures were found to the same level of performance. The actual damage level, acceptance limits and instability of both methodologies can be showing below.

Table 25. Seismic performances of building.

Structure Frame direction	Pushover Analysis		Time history analysis	
	δ_t (FEMA440)	DG(Lang)	Accepted	Unstable
X-direction	CP	4	40%	60%
Y-direction	CP	4	30%	70%

Accepted criteria: Life safety, Unstable: Collapse prevention and Collapse

Chapter 5

STRENGTHENING AND EVALUATION

5.1 Introduction

This section presents the analytical results of the strengthening case study of the building. Appropriate seismic strengthening techniques were applied to enhance the seismic performance of the structure. The seismic behavior of the strengthened structure and the seismic evaluation using FEMA 440 were conducted through nonlinear analyses. In addition, the probabilistic curves for the strengthened structure were developed and compared with the original structure before strengthening [62].

5.2 Strengthening Technique Methods

From the structural engineering design point of view, the selection of the most appropriate strengthening and retrofitting strategy depends on the structural characteristics of the building and the inelastic behavior of each member. This implies that the most vulnerable structural characteristic and the weakest section of the structure should be considered prior to others. It is also important to consider the effects of different strengthening techniques on the seismic performance assessment, including the dynamic response of the structure and each member to conduct structure stiffness behavior [64]. See Table 26 for limit states with appropriate strengthening strategy.

Table 26 . Strengthening objectives for each limit state criteria (FEMA440).

Limit state	Strengthening objective	Strengthening technique
IO	Increase stiffness and strength	Add shear wall to frames
LS	Increase stiffness and strength	Add RC Column jacketing
CP	Increase durability	Confine columns plastic hinge zones with steel plates

5.2.1 Addition of Shear Wall

The method of strengthening is adding a RC shear wall to the structure frames to reduce the stiffness of the structure. This method is common when there is a stiffness problem in the structure, by using this technique the stiffness and the strength of the building is increased. Lateral stiffness is the most significant effect to the strengthening of the structure [64].

The shear walls were then designed by *ACI 318-08* (ACI Comm. 318 117 2002) Chapter 21 provisions for special RC shear walls to better satisfy the requirements for seismic design category D. The shear walls are 200 mm in thickness. Two layers of #10 reinforcing bars at 300 mm spacing were selected for the vertical and horizontal reinforcement. For special boundary elements, sixteen #10 reinforcing bars were selected for flexure, and #10 hoops and crossties were placed around every longitudinal bar at each end of the wall. Figure 83 shows the elevation view of the both direction frames after adding shear walls and Figure 84 shows the details of the shear wall members.

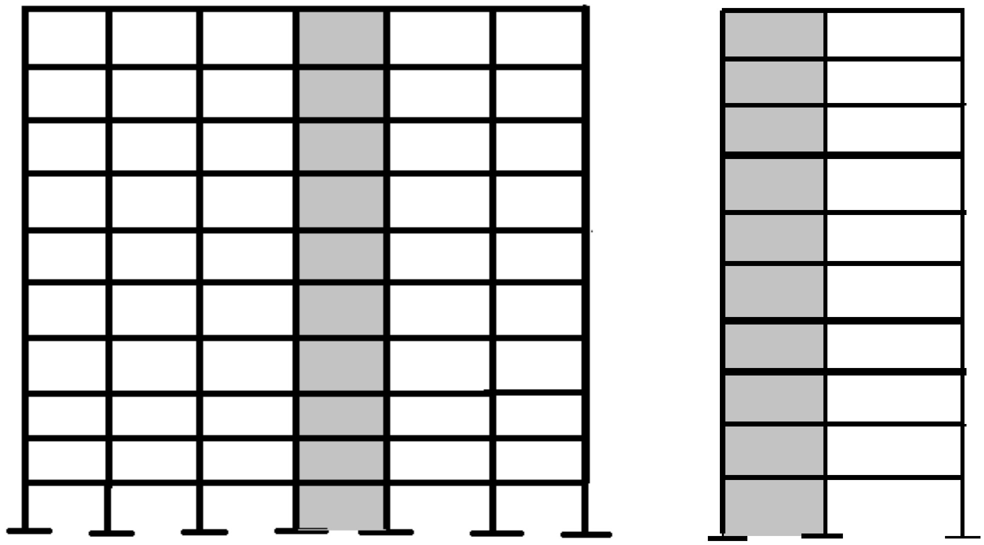


Figure 84. Elevation views of both direction frames after adding shear wall.

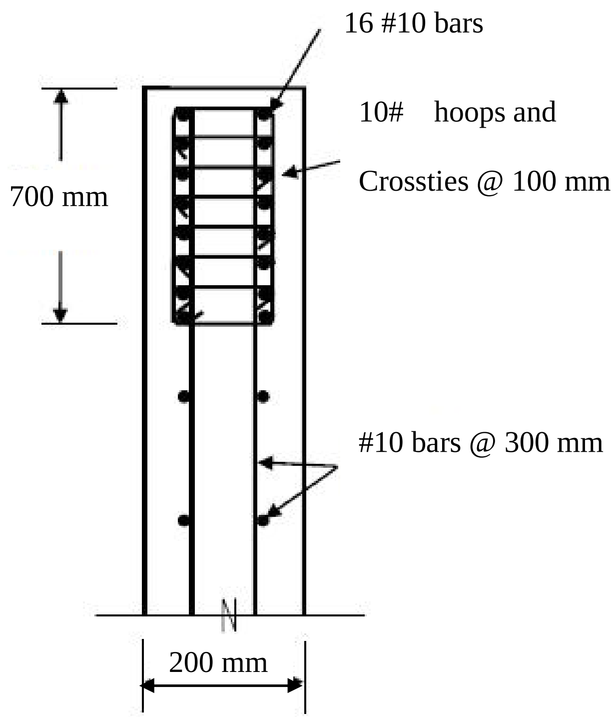


Figure 85. Cross sectional details of RC shear wall.

There were no shear walls in the building system before strengthening and because of this the building became very flexible under the effect of seismic loads. Reinforced concrete shear walls were often used to eliminate stiffness eccentricities in a structure or to increase lateral load carrying capacity. The new shear wall was attached between the two weakest columns at the weakest part of the structure to carry out overturning moments due to the lateral loads applied.

5.3 Seismic Performance Assessment Analysis after Strengthening

After strengthening the building by adding a shear wall, a new assessment of the building's performance should be made in order that the difference in the status of the building can be observed before and after strengthening.

5.3.1 Non-Linear Static Pushover Analysis Method

A pushover analysis indicates the difference after strengthening the building as follows;

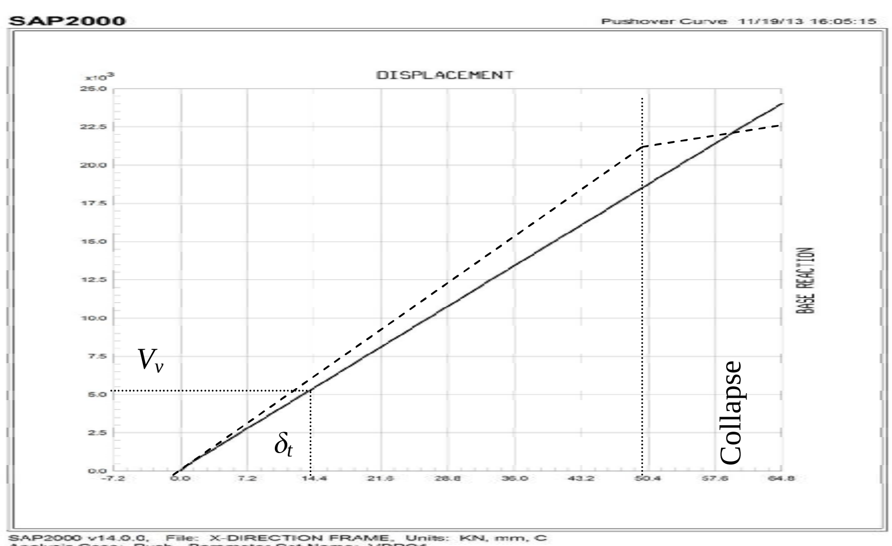


Figure 86. X-Direction frame Pushover curve for existing building after strengthening.

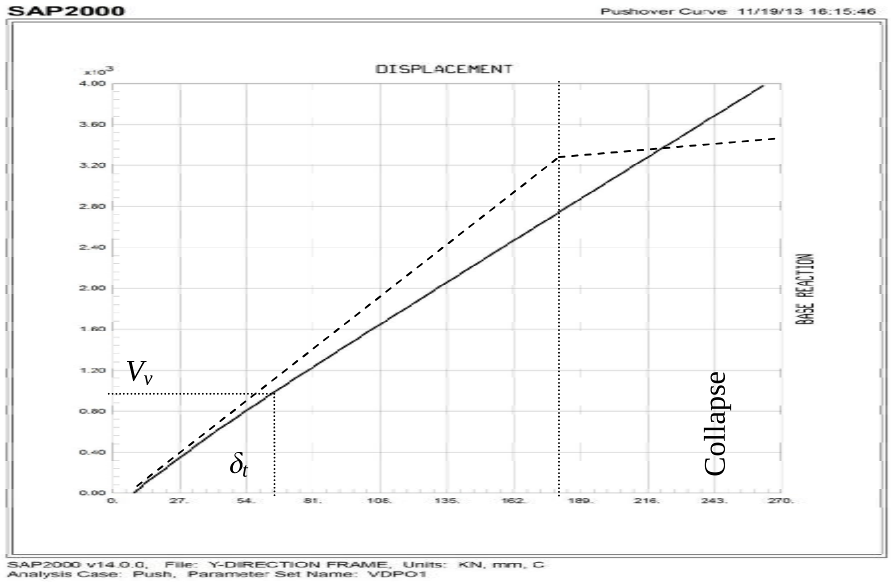


Figure 87. Y-Direction frame Pushover curve for existing building after strengthening.

After strengthening the building by adding shear wall. Target displacement relation with base shear force is calculated according to FEMA440 procedure described in (ASCE, 2000). the Capacity curve (Pushover Curve) for x direction frame model of the building is shown in the figures above where target displacement is calculated as δ_t = 14.60 mm and base shear force causing this displacement is equal to 5000 kN, the capacity curve yielding will start when the roof is displaced 10.00 mm and its first collapse occurs at 45.00 mm. Capacity curve (Pushover Curve) for y-direction frame model of the building is shown in the figures above where target displacement is calculated as δ_t = 60.20 mm and base shear force causing this displacement is equal to 1010 kN, the capacity curve yielding will start when the roof is displaced 25.00 mm and its first collapse occurs at 165 mm. At this point the building cannot resist any more loading and will be in mechanism.

5.3.1.2 Performance limit states and damage perdition

5.3.1.2.1 X-Direction Frame

Table 27 shows the pushover step taken for X-direction frame model via CSI SAP2000.

Table 27. Pushover Curve – (X-direction frame).

Step	Base Force (kN)	Displacement (mm)	Drift Ratio %
0	0	0	0.000
1	352.56	0.61	0.000
2	22727.17	61.15	0.020
3	22723.62	61.16	0.020
4	23250.02	62.61	0.021
5	23243.98	62.62	0.021
6	23873.20	64.34	0.021
7	23866.28	64.35	0.021
8	24042.43	64.83	0.022
9	23950.765	64.68	0.022

Table 28 . Limit states – (X-direction frame).

Limit states	Start (mm)	End (mm)
Yield	0.61	-
IO	0.61	62.61
LS	61.61	64.34
CP	64.34	64.38
Collapse	64.38	65.00

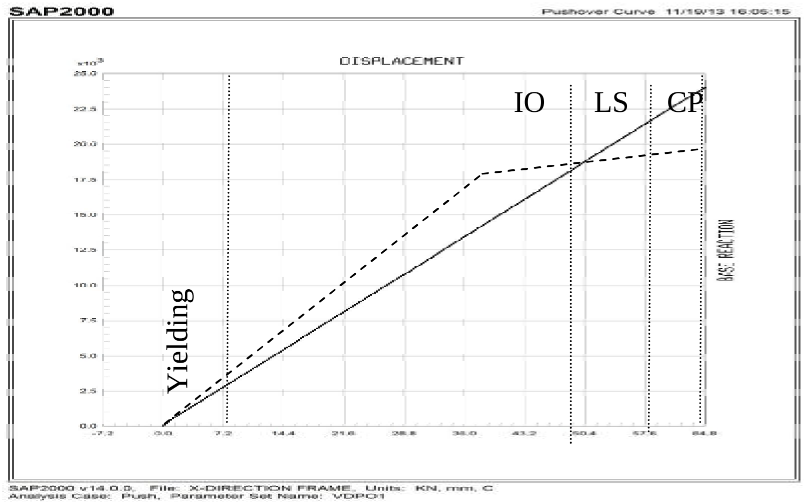


Figure 88. Structure performance limit states of frame model – (X-Direction).

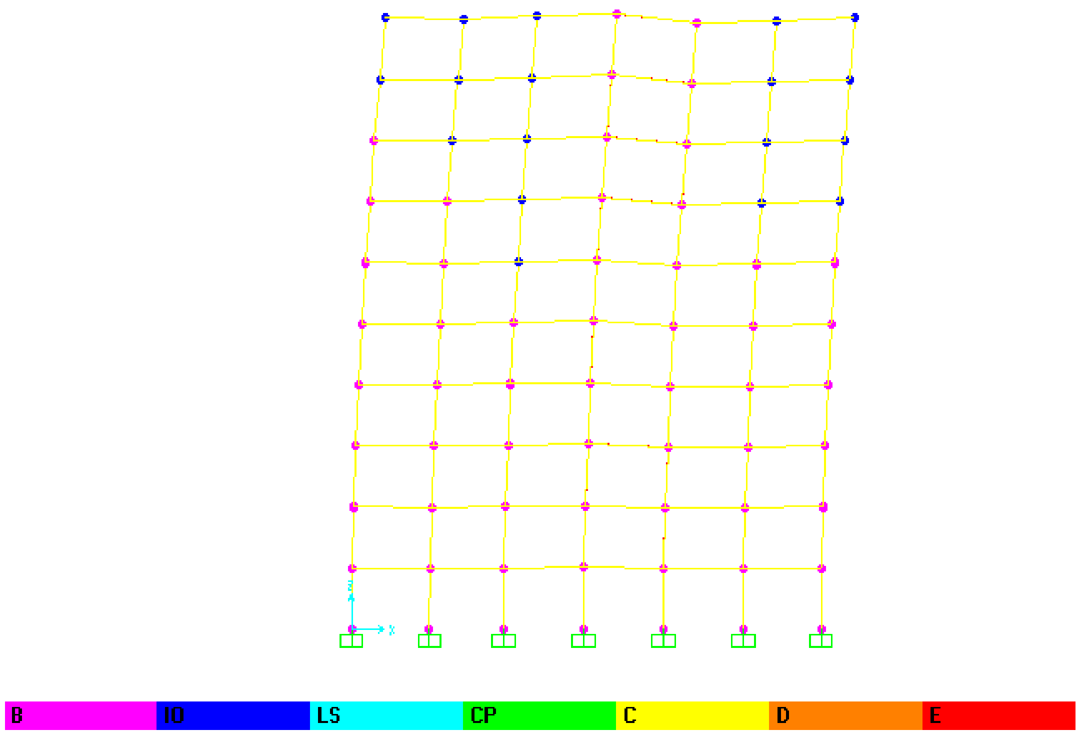


Figure 89. Plastic Hinges formation of frame model after strengthening by SAP 2000 – (X-Direction).

5.3.1.2.2 Y-Direction Frame

Table 29 shows the pushover step taken for Y-direction frame model via CSI SAP2000.

Table 29. Pushover Curve - (Y-direction frame).

Step	**Base Force (kN)**	**Displacement (mm)**	**Roof Drift %**
0	0	8.60	0.003
1	79.74	12.18	0.004
2	579.34	40.06	0.013
3	1012.21	66.73	0.022
4	1535.73	100.61	0.034
5	1938.59	127.01	0.042
6	2619.11	171.91	0.057
7	3236.43	212.99	0.071
8	3790.39	250.07	0.083
9	3977.41	262.60	0.088

Table 30 . Limit states – (Y-direction frame).

Limit staes	Start (mm)	End (mm)
Yield	8.66	-
IO	8.66	66.73
LS	66.73	100.61
CP	100.61	127.01
Collapse	127.01	262.60

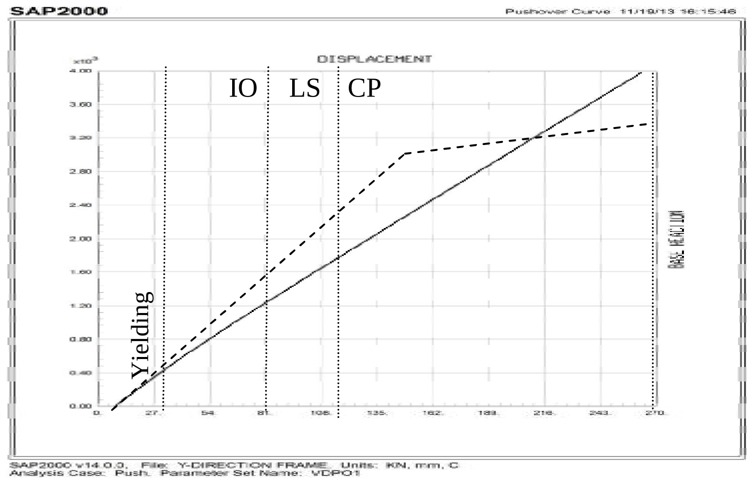


Figure 90. Structure performance limit states of frame model – (Y-Direction).

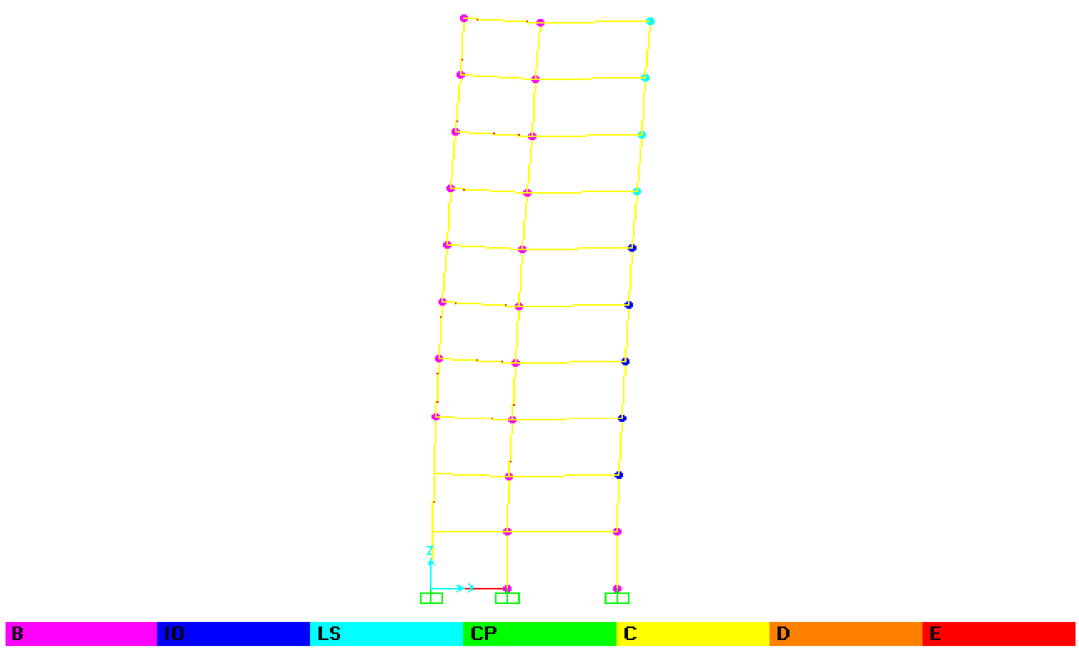


Figure 91. Plastic Hinges formation of frame model after strengthening by SAP 2000 – (Y-Direction).

According to FEMA440, after adding a shear wall in both directions of the building, the existing residential building satisfies the expected performance level. The performance level of the building after strengthening allows for immediate occupancy (IO) in both directions of the building; from x-direction and y-direction.

5.3.2 Time History Analysis Results

Maximum top displacement and spectral acceleration was obtained during the application of each earthquake ground motion to the structure after the strengthening with shear wall.

5.3.2.1 Frame Model (X-Direction) of Building

Maximum displacement due to applied time history records for frame model x-direction after strengthening by shear wall is shown in Table 31. It shows the behavior of control nodes in term of (S_a-S_d) relationship.

Table 31. Spectral displacement & spectral acceleration (x-direction frame).

GM	**S_a(g)**	**S_d (mm)**
TH1	0.012	2.94
TH2	0.026	6.33
TH3	0.008	1.97
TH4	0.004	1.02
TH5	0.011	2.83
TH6	0.006	1.52
TH7	0.017	4.12
TH8	0.013	3.11
TH9	0.010	2.41
TH10	0.015	3.59
TH11	0.009	2.35
TH12	0.006	1.53
TH13	0.007	1.82
TH14	0.005	1.14
TH15	0.008	2.00
TH16	0.006	1.42
TH17	0.012	3.01
TH18	0.005	1.27
TH19	0.008	1.98
TH20	0.010	2.43
	Mean (S_d)	2.43
	Standard Deviation(SD)	1.24
	Mean (S_d) ± SD	(3.67, 1.19)

After adding a shear wall to the x-direction frame, the results obtained according to selected earthquake histories were follows: The structure in x- direction side responded 15% elastically, 45% at immediate occupancy level, 20% at life safety level and 20% at collapse level, so indicating that in this direction of the structure it satisfies 85% of the accepted critera demanded and 15% indicating that it will be unstable. (Figure 91) shows probability decrease of the x-direction frame from the case study building to be at an (acceptable) performance level due to decreased displacement caused by adding a shear wall to the structure taking into account the statistical approach “lognormal distrbution”.

Target dispalcement obtained for x-direction frame is 14.3 mm using a pushover analysis. With this displacment the structure has a probality of 58% to be at IO level, 35% to be at LS level and 20% to be at collapse level according to NDTHA results.

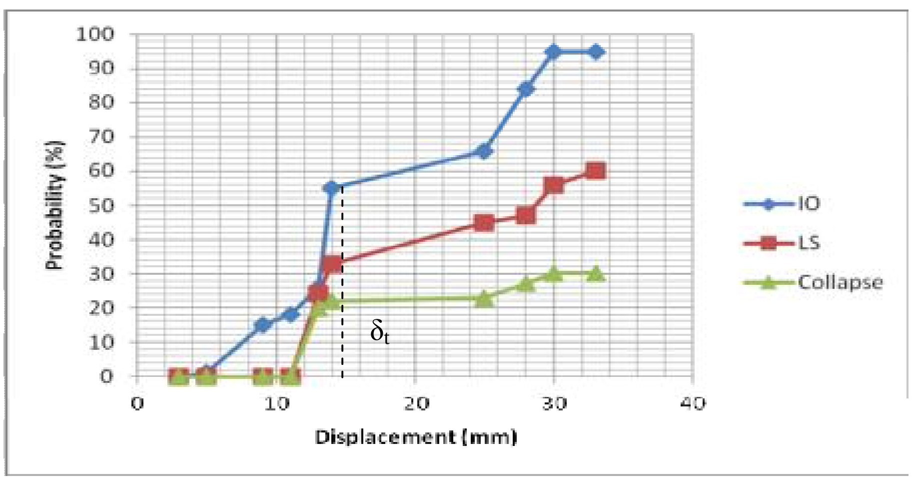


Figure 92. Probability of structure performance in different levels (x-direction).

The structure in x-direction analysis results after strengthening by adding a shear wall in the frame increased its stiffness and became (IO) Immediate Occupancy in performance level.

5.3.2.2 Frame Model (Y-Direction) of Building

Maximum displacement due to applied time history records for frame model x-direction after strengthening by a shear wall is shown in Table 32. It shows the behavior of control nodes in term of (S_a-S_d) relationship.

Table 32. Spectral displacement & spectral acceleration (y-direction frame).

GM	S_a(g)	S_d (mm)
TH1	0.003	1.70
TH2	0.011	5.48
TH3	0.003	1.51
TH4	0.015	7.81
TH5	0.004	2.15
TH6	0.010	5.24
TH7	0.006	3.13
TH8	0.046	23.7
TH9	0.004	1.89
TH10	0.005	2.72
TH11	0.023	11.79
TH12	0.011	5.74
TH13	0.003	1.38
TH14	0.017	8.72
TH15	0.030	15.23
TH16	0.009	4.50
TH17	0.012	5.98
TH18	0.019	9.69
TH19	0.015	7.47
TH20	0.023	11.85
	Mean (S_d)	6.88
	Standard Deviation(SD)	5.57
	Mean (S_d) ± SD	(12.45, 1.31)

After adding a shear wall to the y-direction frame, the results obtained according to selected earthquakes were as follows: The structure in y- direction side responded 15% elastically, 50% at immediate occupancy level, 25% at life safety level and 10% at collapse level, indicatiing that in this direction the structure is 75% in compliance with accepted criteria but 25% will remain unstable. (Figure 99) shows the probability decrease of y-direction frame from the case study building at a performance level due to decreased displacement caused by adding a shear wall to the structure and taking into account the statistical approach “lognormal distrbution”.

Target dispalcement obtained for y-direction frame is 60.20 mm by pushover analysis. With this displacment the structure has a probality of 85% to be at IO level, 60% to be at LS level and 25% to be at collapse level according to NDTHA results.

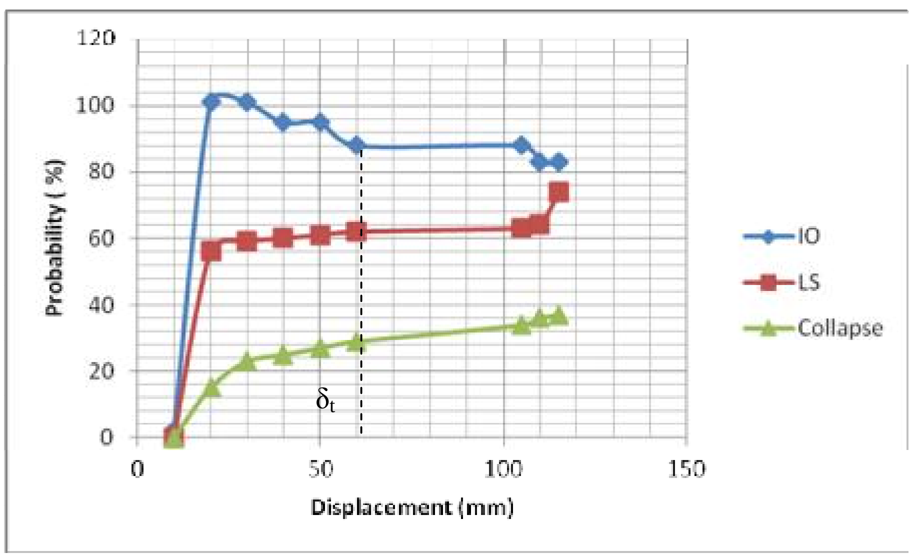


Figure 93. Probability of structure performance in different levels -(Y-Direction).

The structure in y-direction analysis results after strengthening by adding a shear wall in the frame increased its stiffness and became (IO) Immediate Occupancy in performance level.

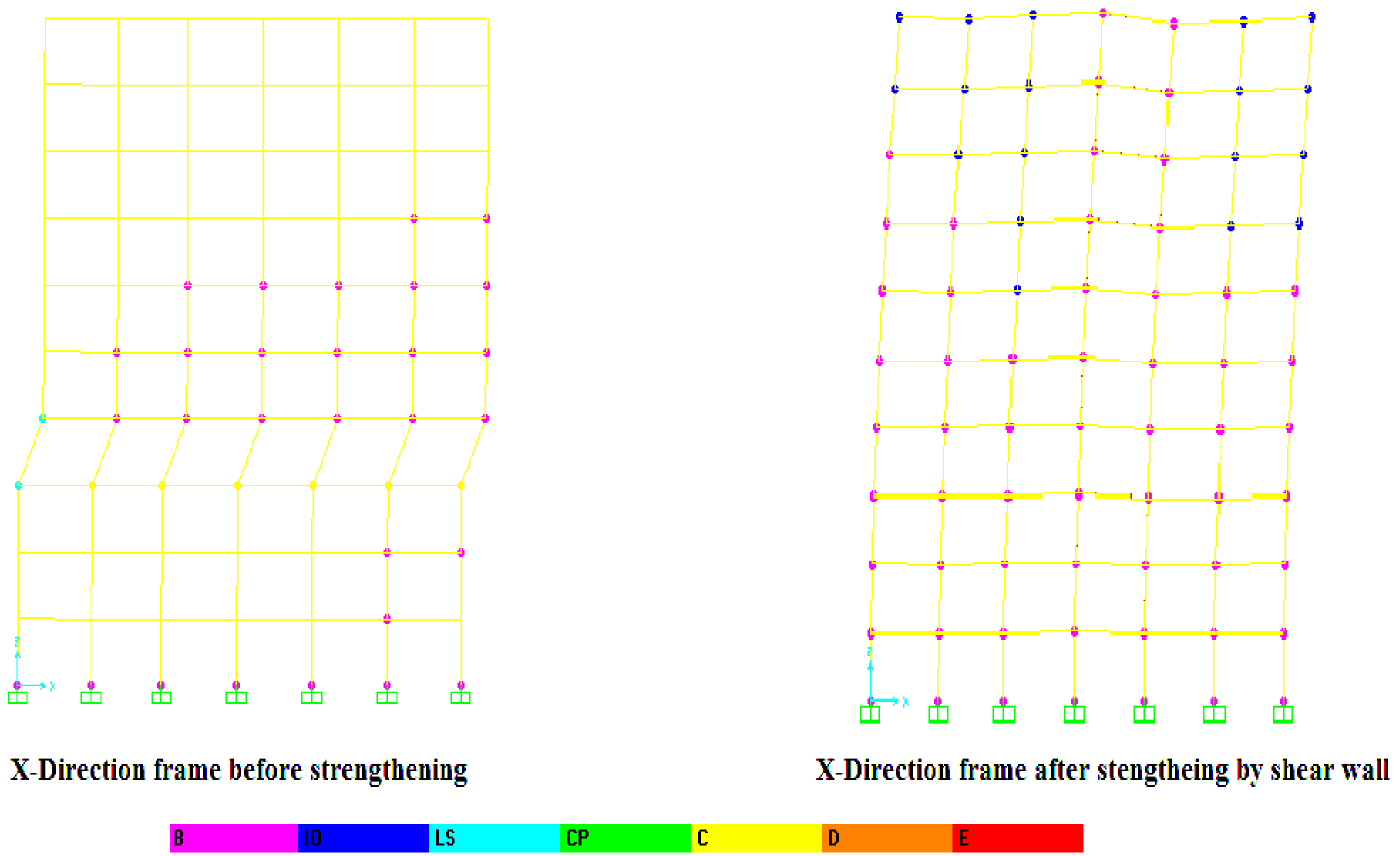


Figure 94. Plastic Hinges formation of frame model after Strengthening by SAP 2000 – (X-Direction).

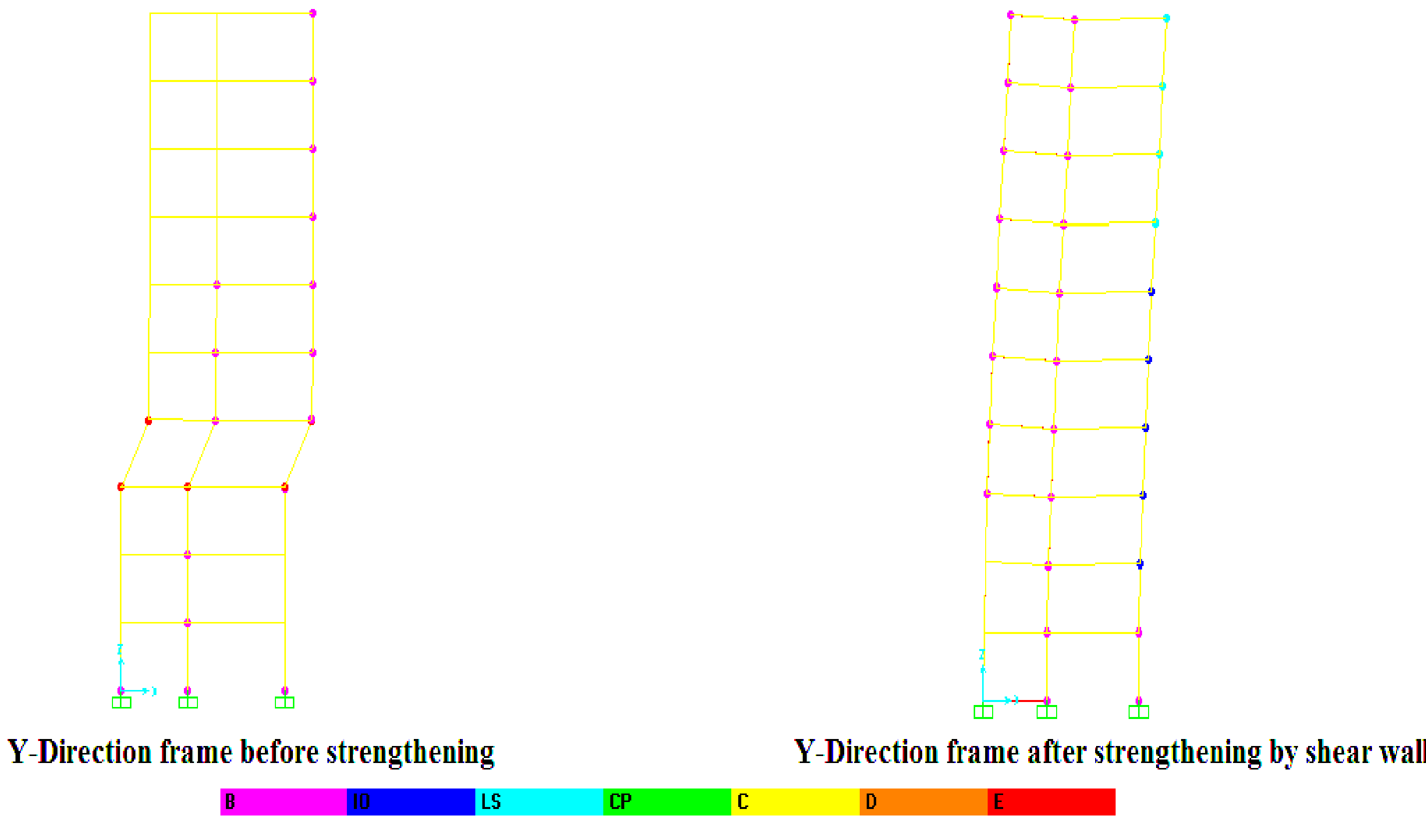


Figure 95. Plastic Hinges formation of frame model after strengthening by SAP 2000 – (Y-Direction).

Chapter 6

CONCLUSION AND RECOMMENDATIONS

6.1 Summary

A reinforced concrete structure, which was constructed in 1970 as a hotel building, is assessed by employing various codes or guidelines and different analysis procedures in this study. The building which is located in Famagusta is a ten storey reinforced concrete structure serving as a hotel building.

In the previous chapters, both existing and strengthened structures are first processed with non- linear static pushover analysis and non-linear dynamic time history analysis by being subjected to twenty ground motion records. Displacements were calculated as a result of this analysis and finally, before and after the strengthening, the structural systems were assessed by using the nonlinear procedures proposed in the FEMA 440. By using the analysis results, although the existing building achieved a collapse prevention performance level it was obvious that the mechanism would cause catastrophic failure with a flexible stiffness. The structural systems were assessed according to the FEMA440 and by placing the new shear walls in the existing moment resisting frames, the existing system was strengthened.

6.2 Conclusion

Two different alternative procedures were introduced, namely non-linear static and non-linear dynamic assessment procedures. In this study the performance of both procedures are examined.

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

- The existing structure was in collapse level. More than one strengthening technique was tried. The first method was jacketing columns. After strengthening with this technique almost all of the columns were not got a safe level of performance because the structure was in flexible stiffness system and the problem was in structure stiffness.

- It can be observed that post-yielding of the structure may cause ***"negative stiffness"*** in which under dynamic excitation behavior might be very complicated due to the instability of the structure during the excitation as explained in chapter 5.

- One of the most common modes of failure of a structure is the defeat of stability. The progression of failure from the loss of stability is invariably a dynamic procedure wherein the motion of the structure, normally to no good ends [65].

- After the addition of shear walls, the performance level of the existing structure was increased. FEMA 440 performance point was rising although the capacity curve increased with the shear wall, the performance level was at collapse stage since mechanism has been observed and plastic hinges concentrated at one of the stores. However, at the shear wall stage the Seaside Hotel's performance level allowed to reach immediate occupancy (IO) level as shown in chapter 5.

- According to the FEMA440 code both strengthening methods are suggested. However, one should be careful to select an appropriate strengthening strategy as observed in this study

- A great number of column reinforcing bars were found to be corroding. Their formation results in the delaminating of concrete. This is one of the results that affect the performance level of the structure. In order to stop the corrosion, the application of epoxy coating of steel is a specialized technique which can be helpful in addition to an adequate thickness of cover concrete of low permeability.

- Although the average compressive strength of the concrete of the existing structure was obtained as range (7-36) MPa, as shown in Appendix B,

- Tensile strength of steel reinforcement bars is 220 MPa as standard, described in chapter 3.

6.3 Recommendations

Future researches and works may be recommended as detailed below:

- It is expected that this study will be a point of reference to other RC multi story buildings in the TRNC and around the world.

- The similar approach can be taken for new materials such as composites and fiber reinforced materials but only if the cyclic behaviors are available.

- This study could be extended to other types of structures, including steel, masonry, composite and other concrete structures. The more interesting study is to apply this method on masonry structures that are more brittle in behavior than the modern material such as steel and concrete especially in those specified regions that may or not need strengthening.

- It would be useful to consider the performance of nonstructural members when the limit states are defined.

- An assessment model that evaluates not only the structural performance but also the economic or social impacts of damage would be useful. Then vulnerability functions associated with a specified economic or social impact should be developed. Based on this information, the mitigation option with the optimum cost to-benefit ratio can be determined considering additional important factors.

REFERENCES

[1] USGS (March 3, 1879). The United States Geological Survey, http://www.usgs.gov/.

[2] Antony, W. (2008). *Recommendations for the Seismic Design of High-Rise Buildings*. New York: The Council on Tall Buildings and Urban Habitat.

[3] Krawinkler, H. and Seneviratna, G. D. P. K. (1998). Pros and cons of a pushover analysis of seismic performance evaluation. *Engineering Structures* 20: 452–464.

[4] FEMA 400 (April 2005) NEHRP, *Risk Management Series*, Incremental Seismic Rehabilitation of Hotel and Motel Buildings.

[5] Fardis, M. N. (2003). *Seismic Assessment and Retrofitting of Concrete Buildings Based on EN-Eurocode 8*. New York: Springer.

[6] Khudhur, H. M. and Şensoy, S. (June 2012). Ms Thesis "Discussions on the Seismic Performance Assessment through a Case Study of *Sosyal Konutlar* Buildings in Famagusta". EMU.

[7] Emmons, P. H. (1993). *Concrete Repair and Maintenance Illustrated: Problem analysis, repair strategy, technique*. Kingston, Mass.: RHSMeans.

[8] Mahjoub, N. A. (June 2007). Ms Thesis “Seismic Design of a Current Wood frame Structure and Study of Innovative Products and Damping Systems in Wood Construction”. EMU.

[9] Temecun Yardemic, Eren, Ö, Şensoy, S. (July 2009). Ms Thesis “Seismic Performance Assessment and Strengthening of Gazimağusa Namık Kemal Lisesi”. EMU.

[10] Guevara-Perez, L. T. (2012). “Soft story” and “weak story” in earthquake resistant design: a multidisciplinary approach. *15WCEE*, Lisbon.

[11] Le Corbusier, L. C. (1926). *Five Points for a New Architecture.*

[12] ASCE 7-05 (2005), “Minimum design loads for buildings and other structures”. ASCE/SEI 7-05, American Society of Civil Engineers.

[13] Short Column Effect.http://www.buildingresearch.com.np/services/erd/erd3.php

[14] Behavior of Short Column During Earthquake. http://civil-engg world.blogspot.com /2008/09/behavior-of-short-column-during.html

[15] Çizmecioğlu. F, Doç, O, Çelik, I. T. U. (January 2007), Ms Thesis “Betonarme Okul Binalarinda Depreme Karsi Güçlendirme Öncesi ve Sonrasi Kolon+Perde Kesit Alan Oranlarinin Karsilastirilmasi”. EMU.

[16] Perera, P. F. U. and Jayasinghe, M. T. R. (7th Dec 1993). Earthquake Resistance Detailing For Reinforced Concrete Buildings constructed In Sri Lanka.

[17] Sanders, P. T. (1995), Design and Detailing for Seismic Resistance: An Australian Perspective. In *Earthquake Resistance Design For Reinforced Concrete Structures*, Sydney: Steel Reinforcement Institute Of Australia, 33-41.

[18] Dowrick, D. J. (1977), *Earthquake Resistant Design*. London: John Wiley & Sons.

[19] AS (3600-1994), *Concrete Structures Standards*. Australia, New South Wales.

[20] Tena-Colunga, A. (October 26, 2009). Review of the Soft First Story Irregularity Condition of Buildings for Seismic Design, Departamento de Materiales, *Edificio 4P, Último Piso, Universidad Autónoma Metropolitana Azcapotzalco.* San Pablo, Mexico.

[21] Tuutti, K. (1982). Corrosion of steel in concrete, *Swedish Cement and Concrete Research Institute*, Report 4-82, 469.

[22] http://theconstructor.org/structural-engg/strengthening-structures/1576/

[23] Tankut, T., Ersoy, U., Özcebe, G., Baran, M., Okuyucu, D. (2003), In-service Seismic Strengthening of RC Frames Buildings. *NATO SFP Workshop on Seismic Assessment and Rehabilitation of Existing Building*, NATO Science Series, Volume 29. Amsterdam, the Netherlands: Klıwer Academic Publications.

[24] http://www.dreamghar.com/review4.html

[25] http://theconstructor. org/concrete/causes-of-cracks-remedies/5392/

[26] Eurocode 8 (BS EN 1998-3:2005) British Standard, Design of structures for earthquake resistance- Part 3: Assessment and retrofitting of buildings, ISBN 0 580 46615 9.

[27] Banville, M. H. (July 2008) *Assessment and repair of concrete structures*.

[28] http://theconstructor. org/concrete/methods-of-crack-repair/886/

[29] http://www.dummies.com/how-to/content/how-to-fix-cracks-in-your-housesfoundation.html

[30] http://theconstructor.org/structural-engg/strengthening-techniques-r-c-slab/1921/

[31] Can, H. O. (1996). *Betonarme Kolonların Komsu Yüzünden Mantolanmıs Betonarme Kolonların Deprem Davranısı*, Teknik Dergi, Cilt 7 Sayı 1.

[32] Shrestha, H. D., Pribadi, K. S., Kusumastuti, D., Lim, E. *Manual on Retrofitting of Existing Vulnerable School Buildings – Assessment to Retrofitting Part I.*

[33] Amine, A. *Seismic Design of Reinforced Concrete Structures*

[34] Tonekaboni, M. (2005). *The Strengthening Principles of Buildings*. Iran: Azadeh.

[35] Dynamic Isolation System. (2007). *Seismic Isolation for Buildings and Bridges: The best earthquake protection technology on the world.* Nevada, US.

[36] Naeim, F. and Kelly, J. M. (1999). *Design of Seismic Isolated Structures: From theory to practice*. New York: John Wiley & Sons.

[37] National Information Service for Earthquake Engineering. (1998). *Base Isolation: Original and development*. Retrieved October 7, 2008 from http://nisee.berkeley.edu/lessons/kelly.html

[38]http://www.bridgestone.com/products/diversified/antiseismic_rubber/method.html.

[39] http://nisee.berkeley.edu/lessons/kelly.html

[40] http://nees.org/resources/3832

[41] Aboulhosn, R. S. (2006). OSPHOD Pre-Approved System. Seismic Restraints Multi-Directional Bracing For Electrical Conduit, Cable Tray and Mechanical Piping Systems. *Structural Engineer.*

[42] TEC 2007. *Specification for structures to be built in disaster areas– earthquake disaster prevention, 2007*. Ministry of Public Works and Settlement, Government of the Republic of Turkey.

[43] Applied Technology Council, ATC-40, 1996, Seismic Evaluation and Retrofit of Concrete Buildings, Volume 1-2, Redwood City, California.

[44] Federal Emergency Management Agency (2000). *Prestandard and Commentary for the Seismic Rehabilitation of Buildings.* Washington, DC: FEMA-356.

[45] M. Neville, Brooks, J. J. (2001). *Concrete Technology: Revised Edition.* London: Pearson Education.

[46] Hakan. Y, Serhan. S and Ozgur, E. (2011). Time Dependent Seismic Performance Assessment of a Single-Degree-of-Freedom Frame Subject to Corrosion. *Engineering Failure Analysis,* vol. 19, 1-5.

[47] Mansour, A. E. (16 September, 2007). *AHM531 Schmidt Hammer Test.* http://www.ahm531.com/labconc.html.

[48] Concrete Society *Technical Report No. 11*-Core Testing for Strength.

[49] ASTM C39/C39M. (2011). *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.*

[50] FEMA 440 (June 2005) ATC, improvement of nonlinear static Seismic analysis procedures.

[51] FEMA 356 (November 2000) ASCE, pre standard and commentary for the Seismic rehabilitation of buildings.

[52]http://www.labtesting.com/services/materials-testing/mechanical-testing/tensiletesting/

[53] ASTM E8/E8M-13 *A Standard Test Method for Tension Testing of Metallic Materials.*

[54] Turkish Standard Code (TEC 708 BC-1A, March 1996).

[55] Sucuoğlu, H., The Turkish Seismic Rehabilitation Code. *First European Seismic Rehabilitation Code*, First European Conference on Earthquake Engineering and Seismology (a joint event of the 13th ECEE & 30th General Assembly of the ESC) Geneva, Switzerland, 3-8 September, 2006.

[56] Federal Emergency Management Agency (2005), *Prestandard and Commentary for the Improvement of Nonlinear Static Seismic Analysis Procedures*, Washington, DC: FEMA-440.

[57] Khanlou, A. H. M. and Şensoy, S. (September 2008). Ms Thesis "Evaluation and Assessment of Reinforced Concrete Building by Incremental Dynamic Analysis". EMU.

[58] Mousavi, S. M. F., Şensoy, S. (September 2006). Ms Thesis "Evaluation of Nonlinear Static Procedures in Estimating Seismic Demand". EMU.

[59] Bentz, E. and Collins, M. P. (September 2001). *Sectional Analysis of Reinforced Concrete Members.* University of Toronto, Canada: Acquisitions and Bibliographic.

[60] Martino, R., Spacone E. and Kingsley, G. (2004). Nonlinear Pushover Analysis of RC Structures. *Structures 2000*.

[61] ASCE (2000). Prestandard and Commentary for the Seismic Rehabilitation of Buildings. Reston: Federal Emergency Management and American Society of Civil Engineers.

[62] Gharagozloo, A., Şensoy, S., and Eren, O. (September 2011). Ms Thesis "Seismic Performance Assesement of RC Buildings case of *Brkir Pasa* High School. EMU.

[63] PEER (2010). *Technical Report for the Ground Motion Database web application.* California Pacific Earthquake Engineering Research Center: http://peer.berkeley.edu/peer_ground_motion_database.

[64] Bai, J. and Hueste, M. B. D. (January 2007). Ms Thesis "Deterministic and Probabilistic Evaluation of Retrofit Alternatives for a Five-Story Flat-Slab RC Building". Texas A&M University Zachry Department of Civil Engineering.

[65] Hjelmstad, K. D. and Williamson, E. B. (1998). Dynamic Stability of Structural Systems Subjected to Base Excitation. *Engineering Structures*, vol. 20.

[66] Chopra A. K. (1995). Dynamics of structures; Theory and applications to earthquake engineering. London: Prentice Hall.

[67] ACI Committee 318 (2002), *Building Code Requirements for Reinforced Concrete* (ACI318-02), American Concrete Institute, Farmington, MI.

[68] SAP2000. (2006). Integrated software for structural analysis and design. *Computers and Structures Inc*; Berkeley, California.

[69] Chopra, A. K. and Goel, R. K. (2002). A modal pushover analysis procedure for estimating seismic demands for buildings. *Earthquake Engineering and Structural Dynamics*, vol. 31, 561–582.

[70] Elnashai, A. S. and Pinho, R. (1998). Repair and Retrofitting of RC Walls Using Selective Techniques. *Journal of Earthquake Engineering*, 2, (4), 525-568.

[71] Elnashai, A. S., Pinho, R. and Antoniou (2000), INDYAS – A Program for Inelastic Dynamic Analysis of Structures. *Research Report ESEE/00-2*, Engineering Seismology and Earthquake Engineering Section, Imperial College, London.

[72] Miranda, E. and Akkar, S. (2003). Dynamic instability of simple structural systems. *Journal of Structural Engineering*, ASCE, 129(12), 1722-1726.

[73] Shome, N., and Cornell, C. A. (1999). Probabilistic seismic demand analysis of non-linear structures. *Report No. RMS-35*, Stanford University, Stanford, CA.

[74] Miranda, E. (1991). PhD thesis "Seismic Evaluation and Upgrading of Existing Buildings". Department of Civil Engineering, University of California, Berkeley, CA.

[75] Gupta B, Kunnath SK. Adaptive spectra-based pushover procedure for seismic evaluation of structures. *Earthquake Spectra 2000*, 16, 367-392.

[76] Canbay, E., Ersoy, U. and Özcebe, G. (2003). Contribution of RC Infills to the Seismic Behaviour of Structoral Systems. *ACI Structural Journal*, 100(5).

[77] Uhlig, H. H. (1971). The cost of corrosion to the United States. *Corrosion*, 6, 29-33.

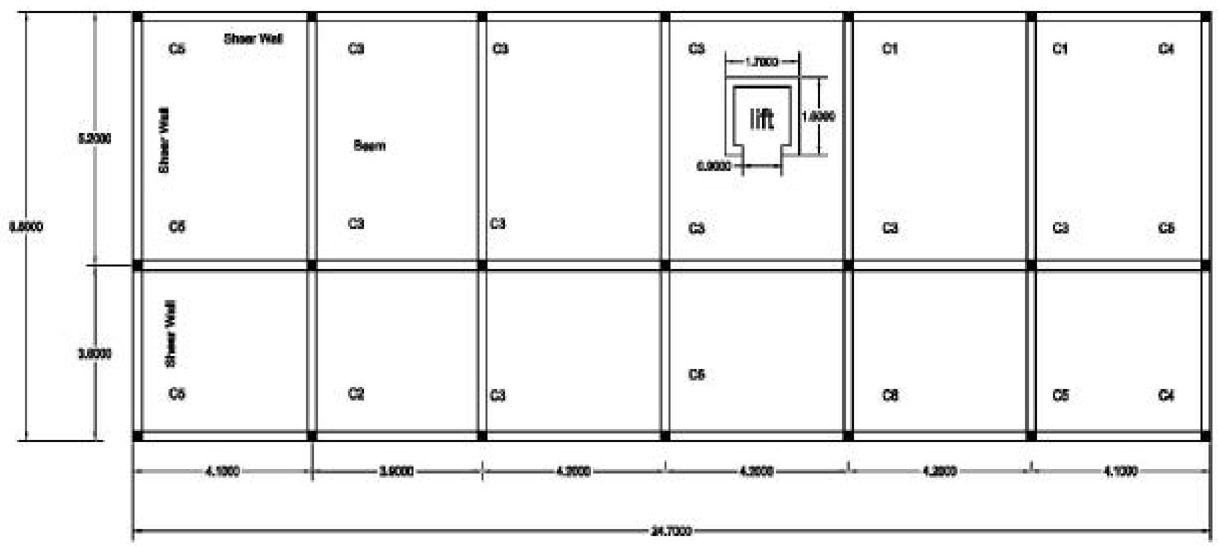
[78] SEAOC (1995). *Performance Based Seismic Engineering of Buildings.* Sacramento: Structural Engineers Association of California.

APPENDICES

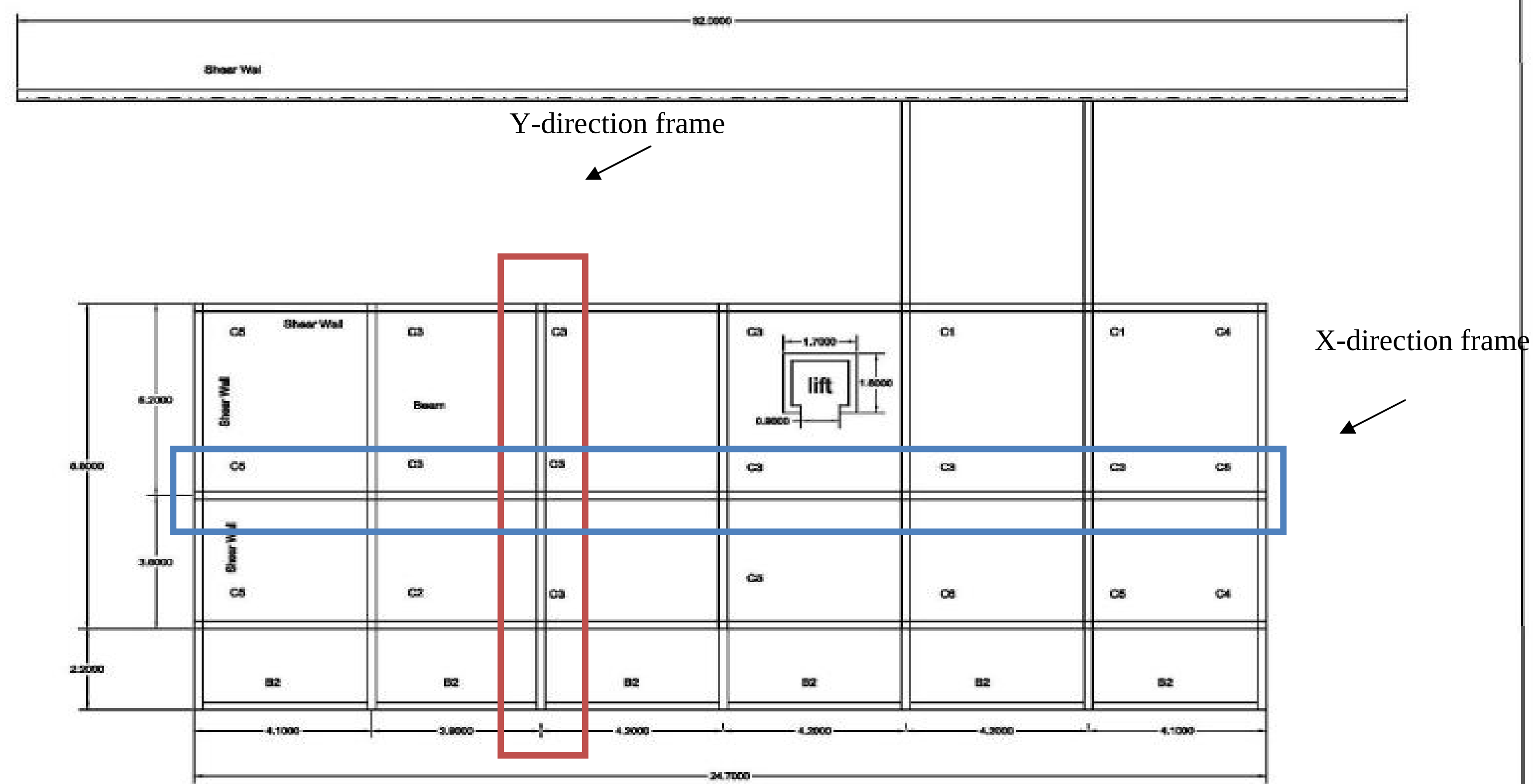
Appendix A: Structural plans of Sea Side Hotel.

The Seaside Hotel is a frame system reinforced concrete building and consists of 10 storeys**,** comprised of two parts: the main section of the hotel and the hotel reception area. The building area dimensions are 24.7 x 8.80 m2 and the height of each floor is 3m.

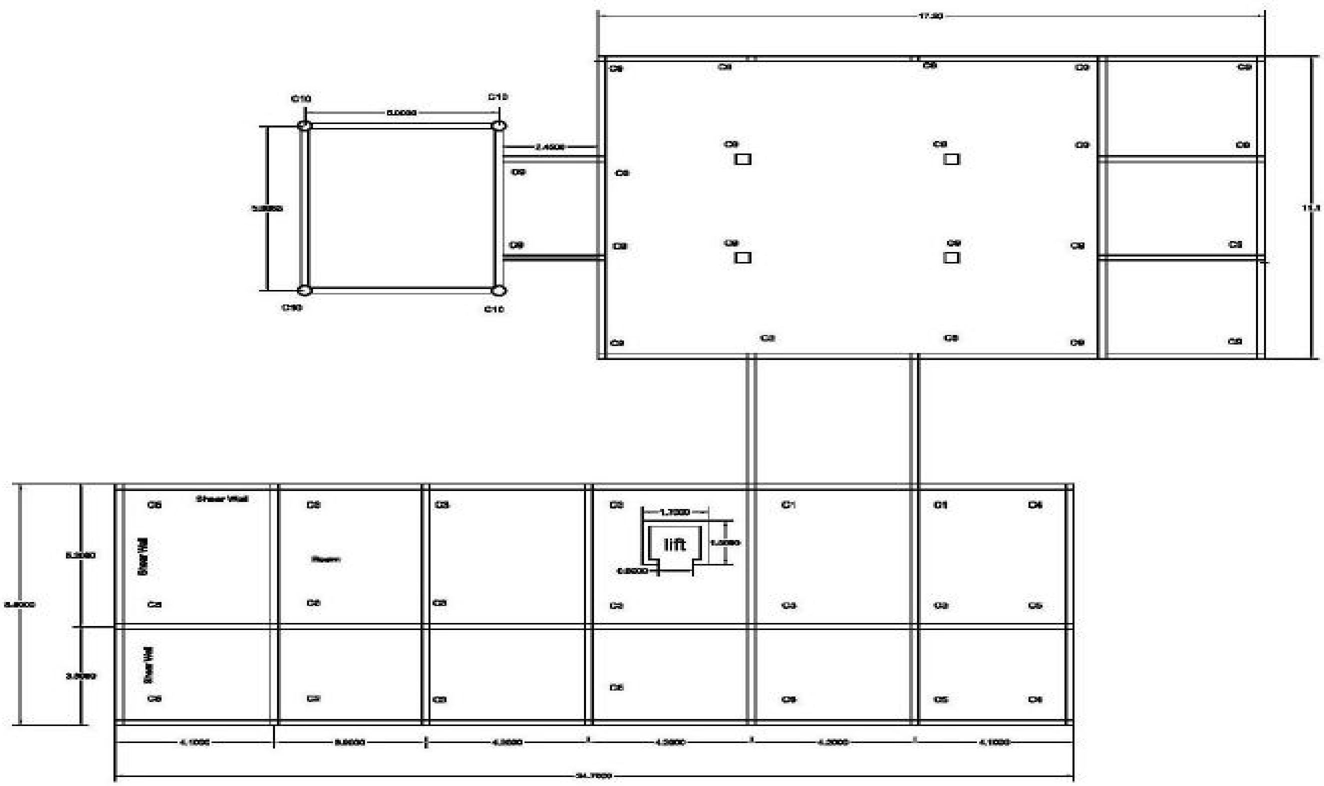
All plans attached with this section, Ground floor plan, First floor plan, Second floor plan, and Third floor plan from this floor till ninth floor the plans of the floors the same.



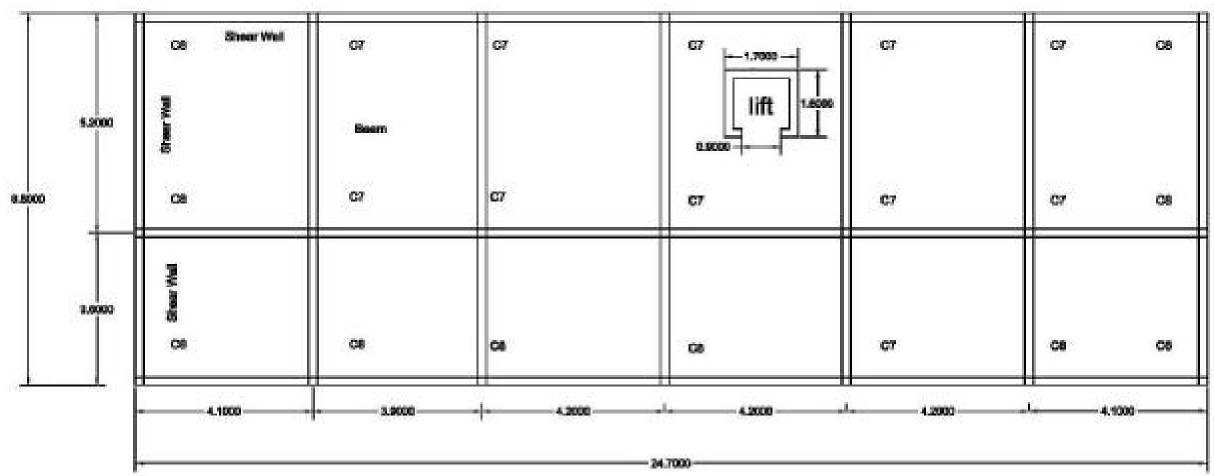
Ground Floor structural plan



First Floor structural plan



Second Floor structural plan



Third – Ninth Floor structural plan

Appendix B: Existing Building Material Properties.

1. The actual concrete compressive strength for each floors of Seaside Hotel and core samples..

No.	Floor Number	Columns Compressive strength	Beams Compressive strength
1	(Ground)	14	36
2	1st floor	14	36
3	2nd floor	33	7
4	3rd floor	8	34
5	4th floor	11	21
6	5th floor	12	25
7	6th floor	8	12
8	7th floor	12	28
9	8th floor	12	20
10	9th floor	11	33

Note: in this study we used the actual compressive strength of each elements on the existing "Seaside hotel" building as a case study.

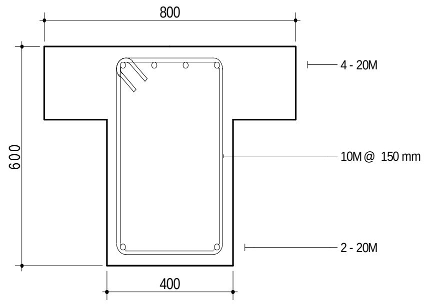
No:	vertical /horizontal	F1	F2	Core no.	core length(mm)	core Diameter(mm)	λ=L/D	1/λ	Cross sectional area(mm2)	crushing value (KN)	crushing value (N)	Core comp. strength (N/mm2)	Actual strength (Mpa)	potentional strength (Mpa)	SF	rebar	diameter	Actual strength (Mpa)	potentional strength (Mpa)
1	**H**	2.5	3.25	**C10.1**	86.1	95	0.906	1.103	7088.2184	145.5	145500	20.5	19.71	25.63	1.000			**19.71**	**26**
2	**H**	2.5	3.25	**C10.2**	86.9	65	1.337	0.748	3318.3072	32.5	32500	9.8	10.89	14.16	1.000			**10.89**	**14**
3	**H**	2.5	3.25	**C10.3**	87.3	65	1.343	0.745	3318.3072	85.4	85400	25.7	28.66	37.26	1.000			**28.66**	**37**
4	**V**	2.3	3.00	**S10.1**	87.0	65	1.338	0.747	3318.3072	126.4	126400	38.1	38.99	50.85	1.027	10.2	10	**40.04**	**52**
5	**V**	2.3	3.00	**S10.2**	86.0	65	1.323	0.756	3318.3072	110.7	110700	33.4	34.01	44.37	1.000			**34.01**	**44**
6	**V**	2.3	3.00	**S10.3**	86.0	65	1.323	0.756	3318.3072	114.8	114800	34.6	35.27	46.01	1.000			**35.27**	**46**
7	**H**	2.5	3.25	**C9.1**	86.6	65	1.332	0.751	3318.3072	No.Record								**No record**	**No record**
8	**H**	2.5	3.25	**C9.2**	86.3	65	1.328	0.753	3318.3072	37.0	37000	11.2	12.37	16.08	1.000			**12.37**	**16**
9	**H**	2.5	3.25	**C9.3**	87.0	65	1.338	0.747	3318.3072	38.3	38300	11.5	12.84	16.69	1.000			**12.84**	**17**
10	**V**	2.3	3.00	**S9.1**	85.3	65	1.312	0.762	3318.3072	99.4	99400	30.0	30.46	39.73	1.000			**30.46**	**40**
11	**V**	2.3	3.00	**S9.2**	85.5	65	1.315	0.760	3318.3072	74.8	74800	22.5	22.94	29.92	1.000			**22.94**	**30**
12	**V**	2.3	3.00	**S9.3**	85.0	65	1.308	0.765	3318.3072	72.8	72800	21.9	22.28	29.06	1.000			**22.28**	**29**
13	**H**	2.5	3.25	**C8.1**	84.8	65	1.305	0.767	3318.3072	44.9	44900	13.5	14.92	19.40	1.000			**14.92**	**19**
14	**H**	2.5	3.25	**C8.2**	85.9	65	1.322	0.757	3318.3072	40.7	40700	12.3	13.59	17.66	1.000			**13.59**	**18**
15	**H**	2.5	3.25	**C8.3**	86.6	65	1.332	0.751	3318.3072	70.0	70000	21.1	23.43	30.46	1.000			**23.43**	**30**
16	**V**	2.3	3.00	**S8.1**	84.6	65	1.302	0.768	3318.3072	91.5	91500	27.6	27.96	36.47	1.000			**27.96**	**36**
17	**V**	2.3	3.00	**S8.2**	87.4	65	1.345	0.744	3318.3072	102.8	102800	31.0	31.76	41.42	1.000			**31.76**	**41**
18	**V**	2.3	3.00	**S8.3**	87.7	65	1.349	0.741	3318.3072	103.1	103100	31.1	31.89	41.59	1.000			**31.89**	**42**
19	**H**	2.5	3.25	**C7.1**	84.5	65	1.300	0.769	3318.3072	58.3	58300	17.6	19.36	25.16	1.000			**19.36**	**25**

20	**H**	2.5	3.25	**C7.2**	85.8	65	1.320	0.758	3318.3072	30.3	30300	9.1	10.11	13.15	1.000			**10.11**	**13**
21	**H**	2.5	3.25	**C7.3**	85.2	65	1.311	0.763	3318.3072	55.6	55600	16.8	18.51	24.06	1.000			**18.51**	**24**
22	**V**	2.3	3.00	**S7.1**	84.9	65	1.306	0.766	3318.3072	119.0	119000	35.9	36.41	47.49	1.000			**36.41**	**47**
23	**V**	2.3	3.00	**S7.2**	85.5	65	1.315	0.760	3318.3072	94.8	94800	28.6	29.07	37.92	1.114	42.2	10	**32.38**	**42**
24	**V**	2.3	3.00	**S7.3**	87.5	65	1.346	0.743	3318.3072	32.6	32600	9.8	10.07	13.14	1.000			**10.07**	**13**
25	**H**	2.5	3.25	**C6.1**	85.8	65	1.320	0.758	3318.3072	33.0	33000	9.9	11.01	14.32	1.000			**11.01**	**14**
26	**H**	2.5	3.25	**C6.2**	83.8	65	1.289	0.776	3318.3072	No.Re cord								**No record**	**No record**
27	**H**	2.5	3.25	**C6.3**	85.7	65	1.318	0.758	3318.3072	87.5	87500	26.4	29.19	37.95	1.000			**29.19**	**38**
28	**V**	2.3	3.00	**S6.1**	87.0	65	1.338	0.747	3318.3072	94.1	94100	28.4	29.03	37.86	1.000			**29.03**	**38**
29	**V**	2.3	3.00	**S6.2**	87.2	65	1.342	0.745	3318.3072	96.8	96800	29.2	29.88	38.97	1.000			**29.88**	**39**
30	**V**	2.3	3.00	**S6.3**	86.5	65	1.331	0.751	3318.3072	80.8	80800	24.3	24.87	32.45	1.000			**24.87**	**32**
31	**H**	2.5	3.25	**C5.1**	83.5	65	1.285	0.778	3318.3072	57.5	57500	17.3	19.01	24.72	1.000			**19.01**	**25**
32	**H**	2.5	3.25	**C5.2**	84.6	65	1.302	0.768	3318.3072	69.4	69400	20.9	23.05	29.97	1.000			**23.05**	**30**
33	**H**	2.5	3.25	**C5.3**	86.5	65	1.331	0.751	3318.3072	56.5	56500	17.0	18.91	24.58	1.000			**18.91**	**25**
34	**V**	2.3	3.00	**S5.1**	84.9	65	1.306	0.766	3318.3072	109.1	109100	32.9	33.38	43.54	1.000			**33.38**	**44**
35	**V**	2.3	3.00	**S5.2**	83.8	65	1.289	0.776	3318.3072	66.0	66000	19.9	20.10	26.22	1.031	11.1	10	**20.72**	**27**
36	**V**	2.3	3.00	**S5.3**	86.5	65	1.331	0.751	3318.3072	130.7	130700	39.4	40.24	52.48	1.000			**40.24**	**52**
37	**H**	2.5	3.25	**C4.1**	84.9	65	1.306	0.766	3318.3072	83.2	83200	25.1	27.67	35.97	1.000			**27.67**	**36**
38	**H**	2.5	3.25	**C4.2**	86.3	65	1.328	0.753	3318.3072	67.6	67600	20.4	22.60	29.38	1.000			**22.60**	**29**
39	**H**	2.5	3.25	**C4.3**	86.0	65	1.323	0.756	3318.3072	53.2	53200	16.0	17.77	23.10	1.000			**17.77**	**23**
40	**V**	2.3	3.00	**S4.1**	85.0	65	1.308	0.765	3318.3072	131.2	131200	39.5	40.15	52.38	1.000			**40.15**	**52**

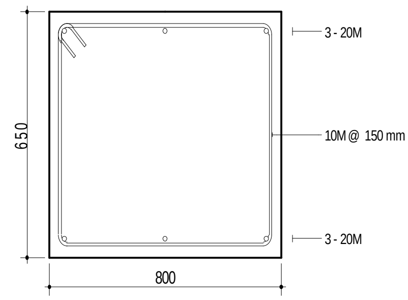
41	**V**	2.3	3.00	**S4.2**	84.0	65	1.292	0.774	3318.3072	112.7	112700	34.0	34.35	44.81	1.000			**34.35**	**45**
42	**V**	2.3	3.00	**S4.3**	86.0	65	1.323	0.756	3318.3072	123.8	123800	37.3	38.04	49.62	1.000			**38.04**	**50**
43	**H**	2.5	3.25	**C3.1**	84.9	65	1.306	0.766	3318.3072	84.0	84000	25.3	27.93	36.31	1.000			**27.93**	**36**
44	**H**	2.5	3.25	**C3.2**	84.8	65	1.305	0.767	3318.3072	97.2	97200	29.3	32.31	42.00	1.000			**32.31**	**42**
45	**H**	2.5	3.25	**C3.3**	84.0	65	1.292	0.774	3318.3072	64.4	64400	19.4	21.34	27.74	1.000			**21.34**	**28**
46	**V**	2.3	3.00	**S3.1**	86.8	65	1.335	0.749	3318.3072	138.6	138600	41.8	42.72	55.72	1.055	20.7	10	**45.07**	**59**
47	**V**	2.3	3.00	**S3.2**	84.8	65	1.305	0.767	3318.3072	155.5	155500	46.9	47.55	62.03	1.000			**47.55**	**62**
48	**H**	2.5	3.25	**C2.1**	84.0	65	1.292	0.774	3318.3072	100.0	100000	30.1	33.13	43.07	1.000			**33.13**	**43**
49	**H**	2.5	3.25	**C2.2**	84.6	65	1.302	0.768	3318.3072	No.Re cord								**No record**	**No record**
50	**H**	2.5	3.25	**C2.3**	83.0	65	1.277	0.783	3318.3072	No.Re cord								**No record**	**No record**
51	**V**	2.3	3.00	**S2.1**	83.6	65	1.286	0.778	3318.3072	121.6	121600	36.6	37.01	48.27	1.000			**37.01**	**48**
52	**V**	2.3	3.00	**S2.2**	84.2	65	1.295	0.772	3318.3072	131.5	131500	39.6	40.12	52.33	1.000			**40.12**	**52**
53	**V**	2.3	3.00	**S2.3**	84.9	65	1.306	0.766	3318.3072	153.7	153700	46.3	47.02	61.33	1.000			**47.02**	**61**
54	**H**	2.5	3.25	**C1.1**	84.4	65	1.298	0.770	3318.3072	58.4	58400	17.6	19.38	25.20	1.000			**19.38**	**25**
55	**H**	2.5	3.25	**C1.2**	85.3	65	1.312	0.762	3318.3072	49.1	49100	14.8	16.35	21.26	1.000			**16.35**	**21**
56	**H**	2.5	3.25	**C1.3**	84.9	65	1.306	0.766	3318.3072	94.4	94400	28.4	31.39	40.81	1.000			**31.39**	**41**

H	2.5	3.25
V	2.3	3.00

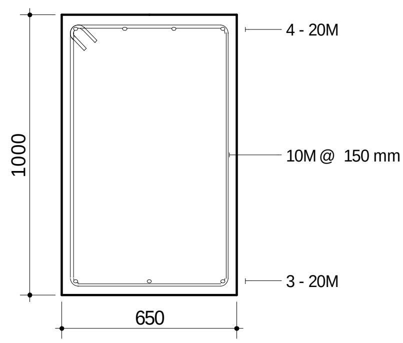
2. Structural elements (Beams and Columns) reinforcement details section:-



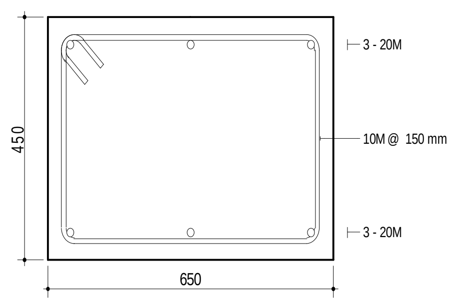
Interior Beam



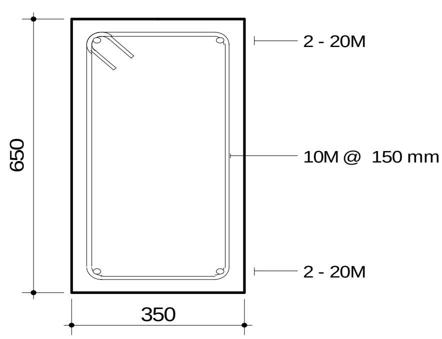
Column (C1)



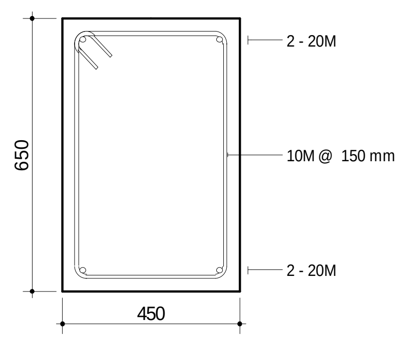
Column (C2)



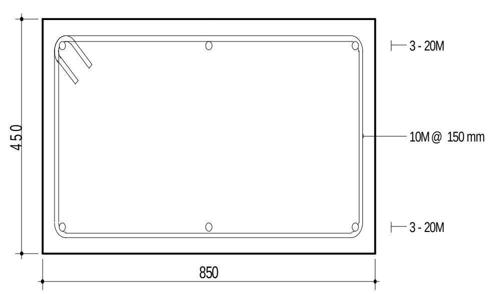
Column (C3)



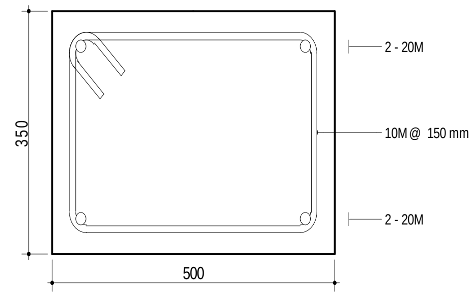
Column (C4)



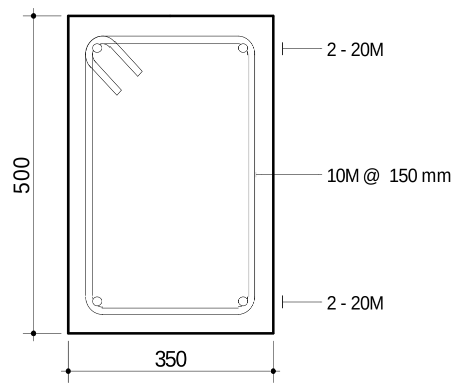
Column (C5)



Column (C6)



Column (C7)

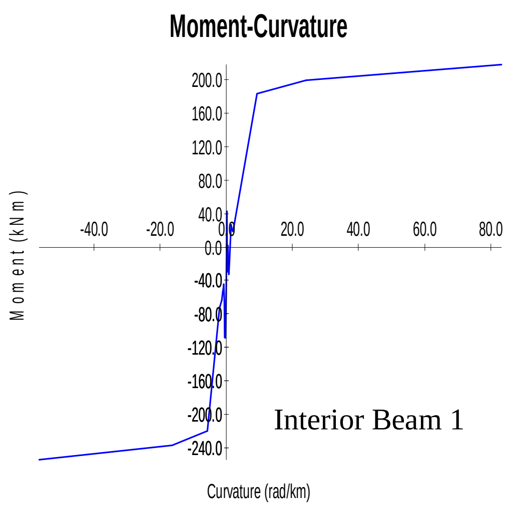


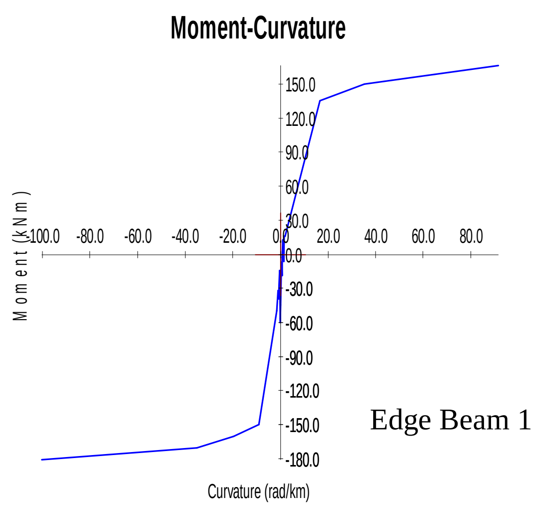
Column (C8)

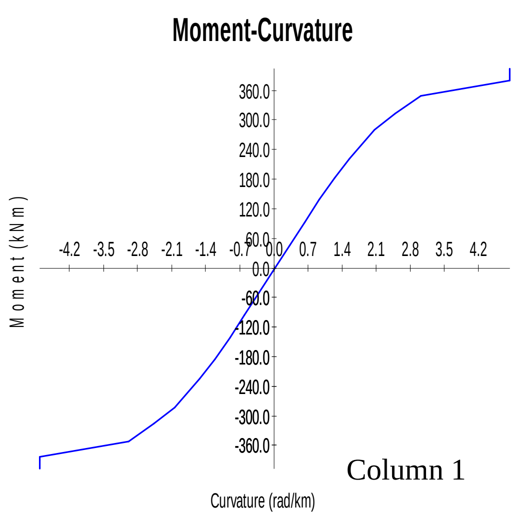
Appendix C: Moment-Curvature Relationships for Members Sections of Building.

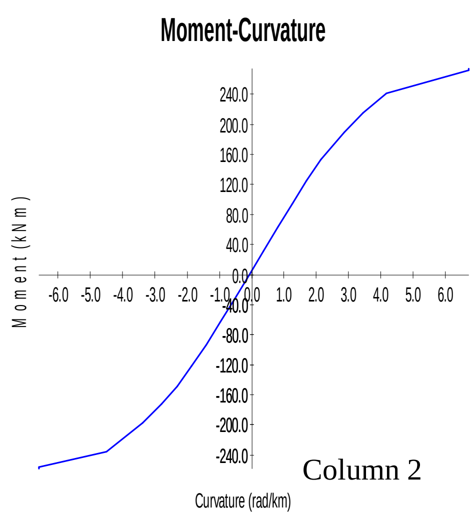
Moment-curvature drawn by Response-2000 program, Horizontal axis represents curvature in (rad/km) and vertical axis represents moment in (kN.m), ---- represents curve, ---- represents idealized curve.

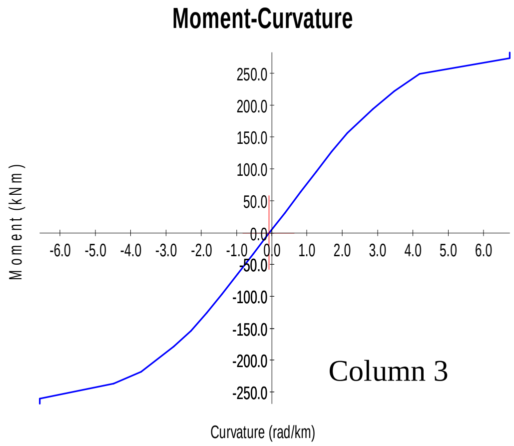
A. Moment-curvature for X-Direction frame elements.

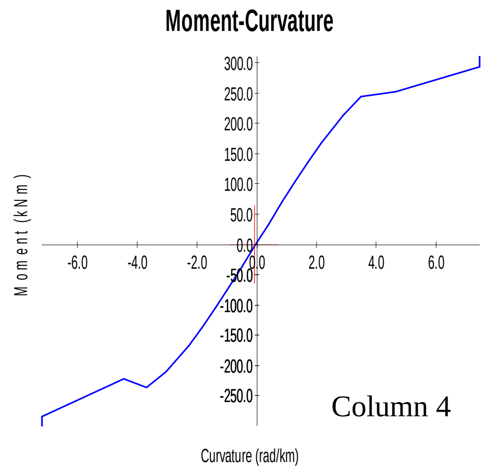


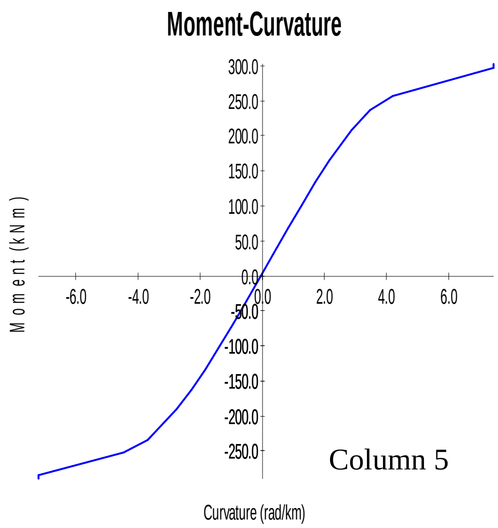


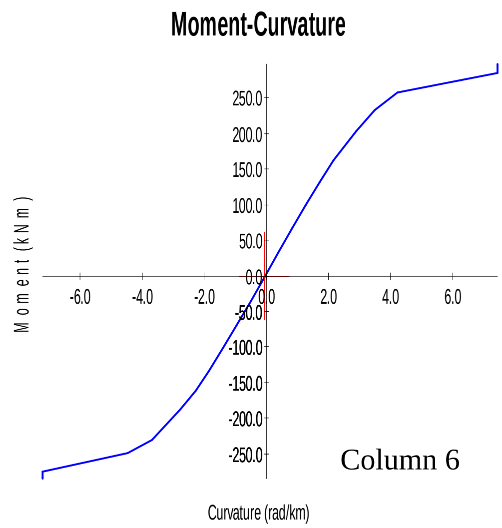


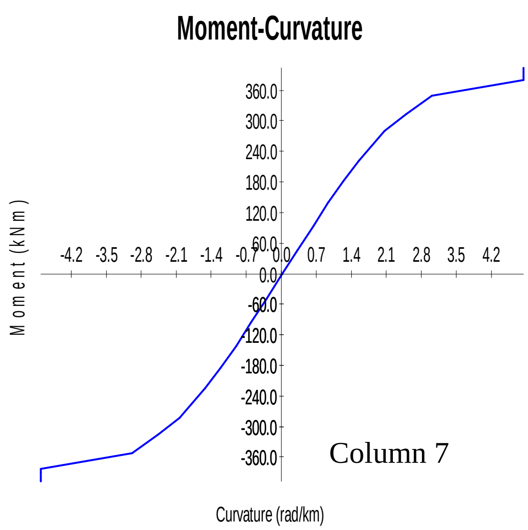


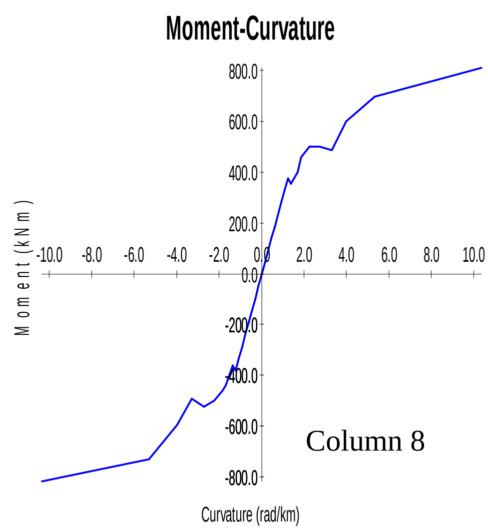

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 3

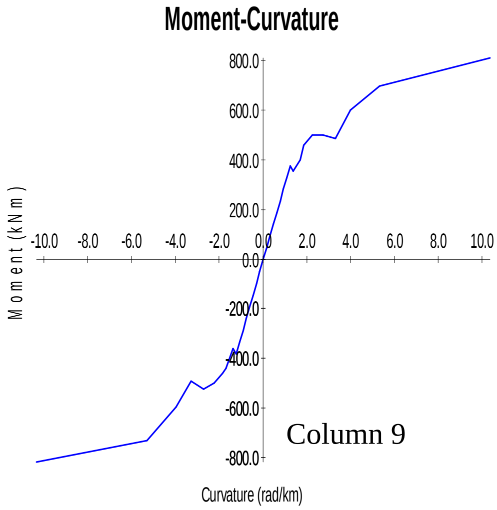

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 4

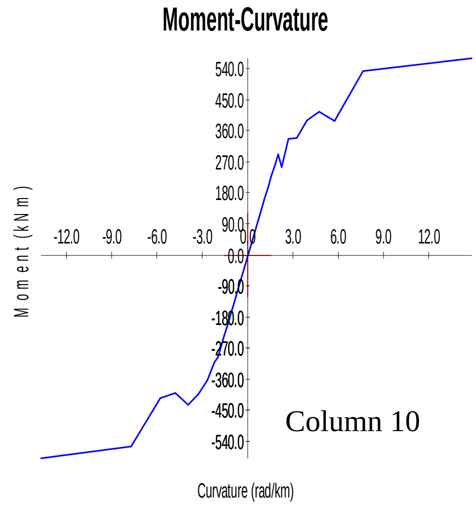

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 5

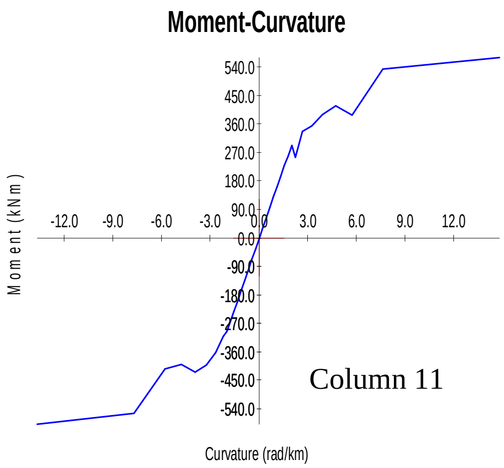

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 6

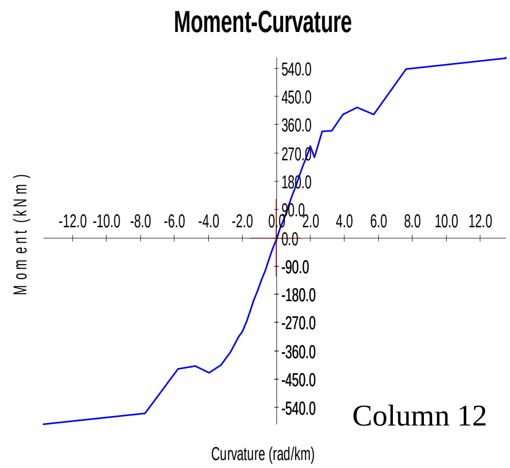

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 7

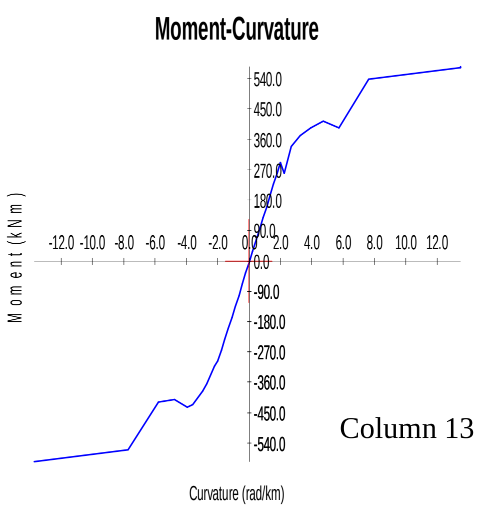

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 8

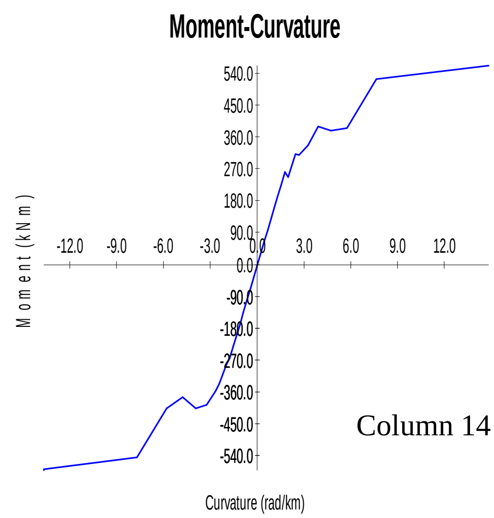

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 9

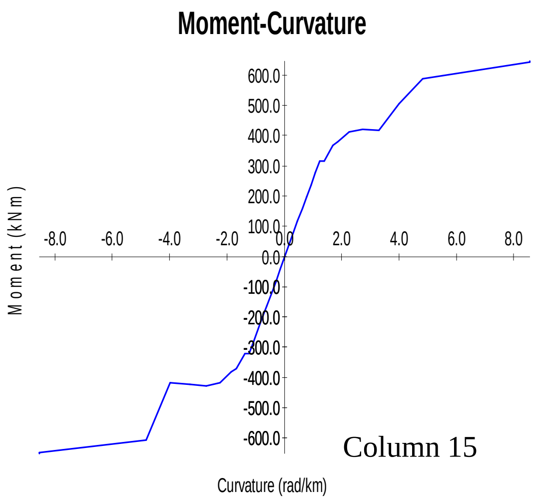

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 10

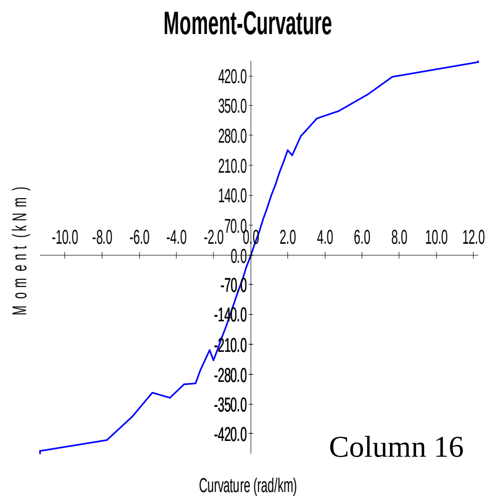

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 11

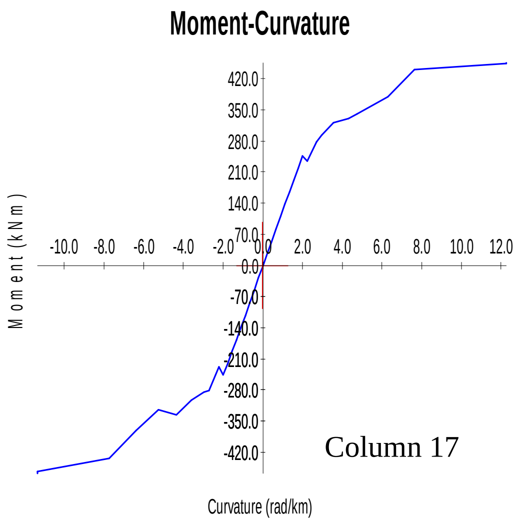

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 12

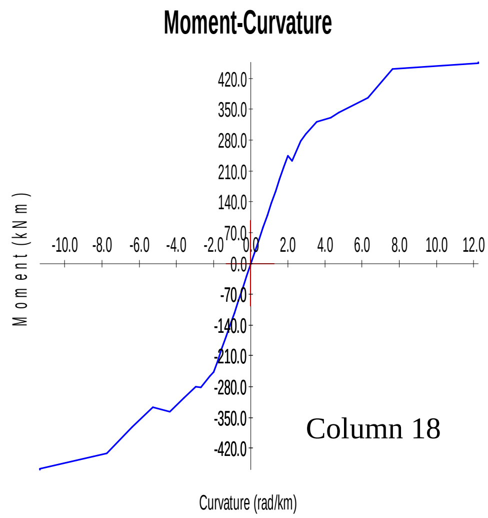

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 13

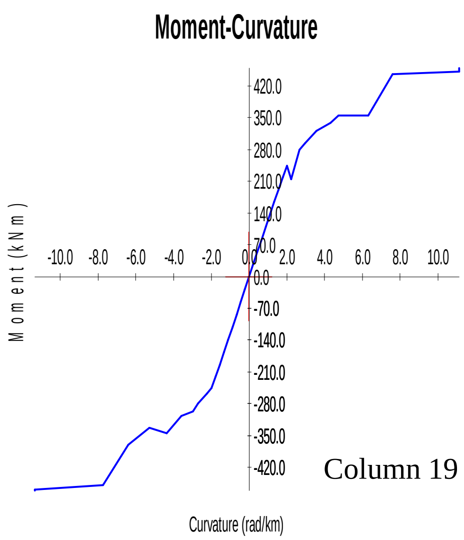

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 14

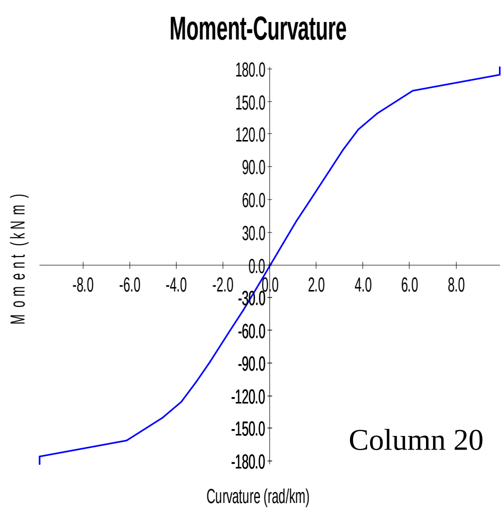

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 15

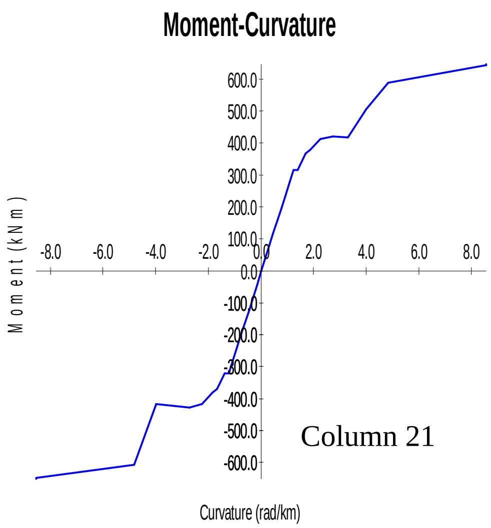

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 16

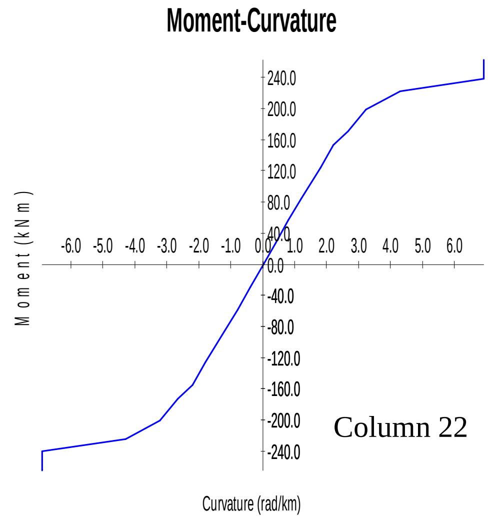

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 17

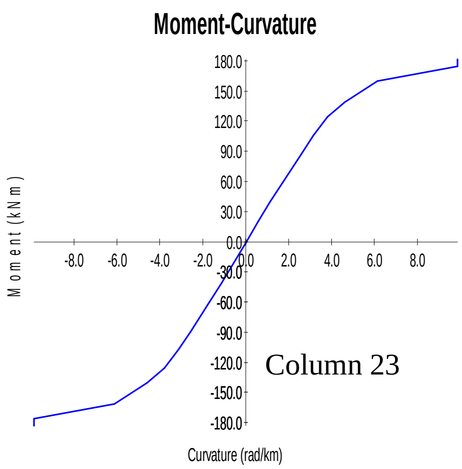

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 18

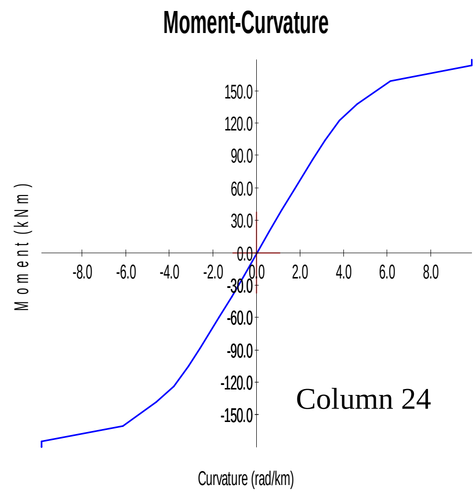

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 19

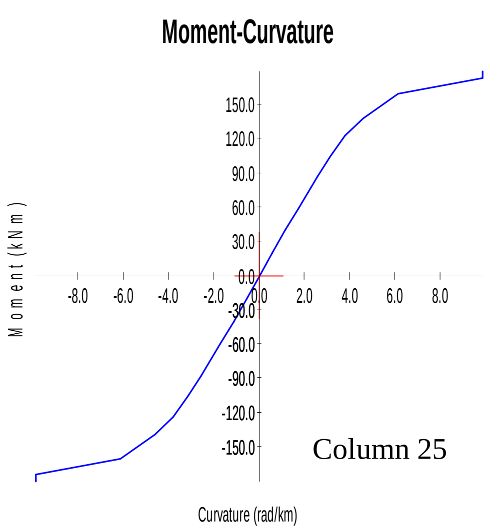

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 20

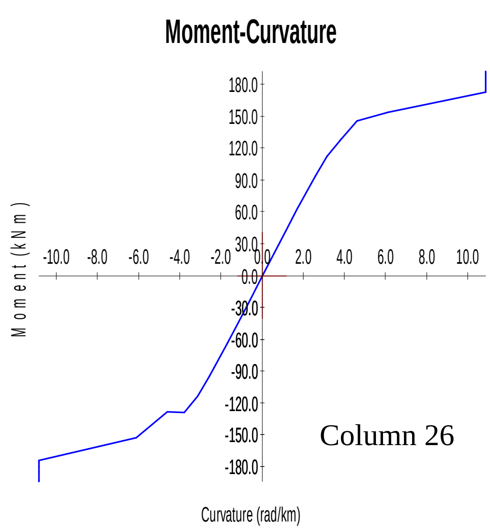

Moment-Curvature
M o m e n t (k N m)
Column 21
Curvature (rad/km)

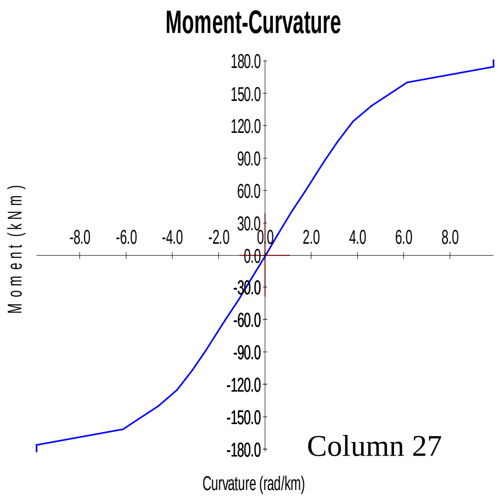

Moment-Curvature
M o m e n t (k N m)
Column 22
Curvature (rad/km)

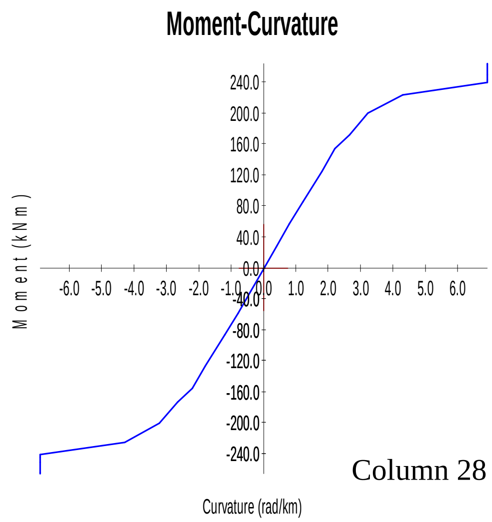

Moment-Curvature
M o m e n t (k N m)
Column 23
Curvature (rad/km)

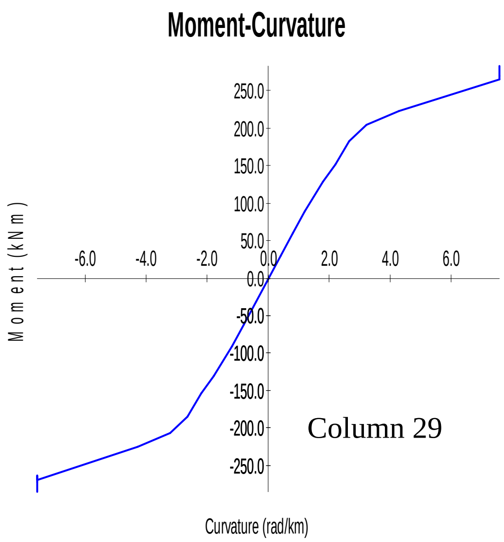

Moment-Curvature
M o m e n t (k N m)
Column 24
Curvature (rad/km)

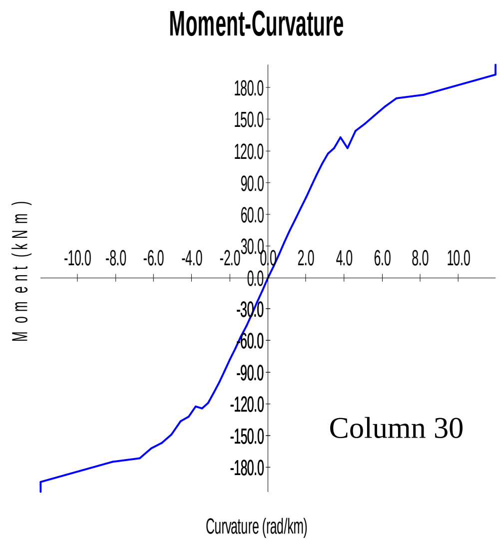

Moment-Curvature
M o m e n t (k N m)
Column 25
Curvature (rad/km)

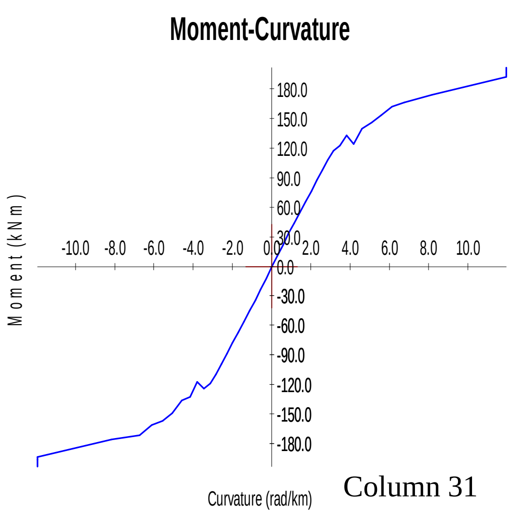

Moment-Curvature
M o m e n t (k N m)
Column 26
Curvature (rad/km)

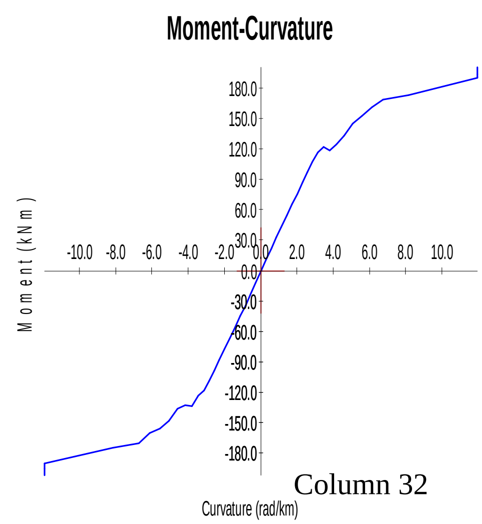

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 27

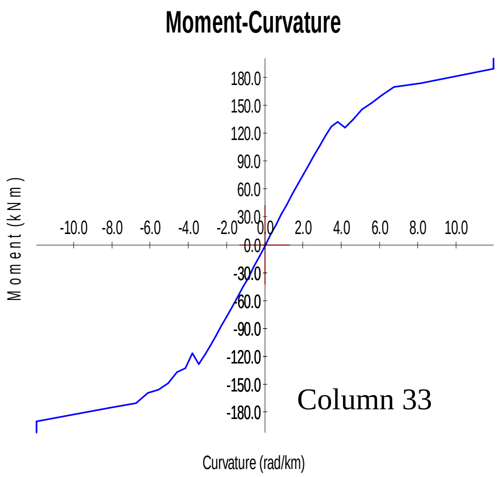

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 28

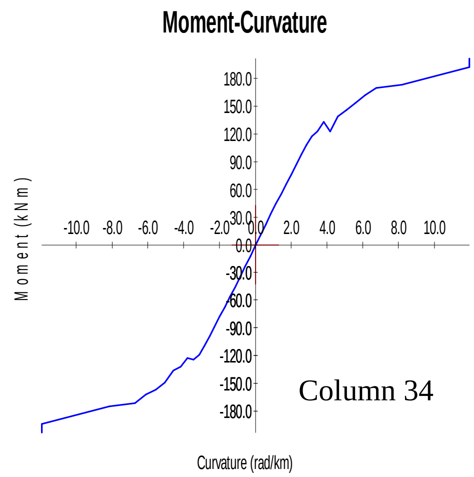

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 29

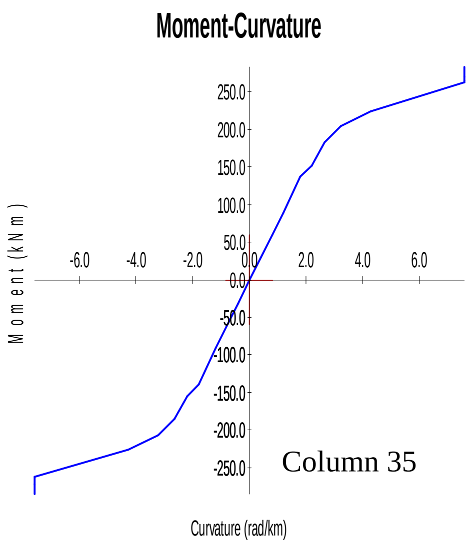

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 30

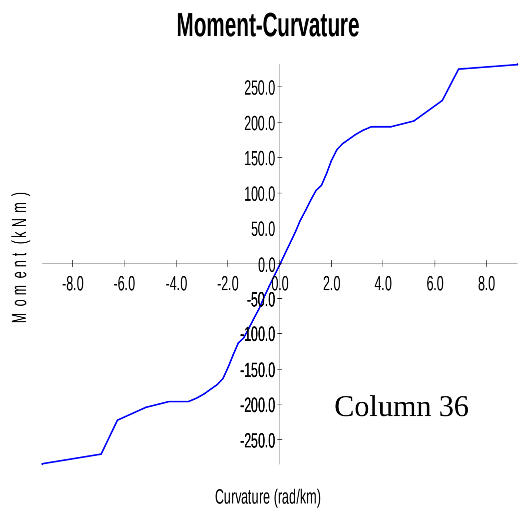

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 31

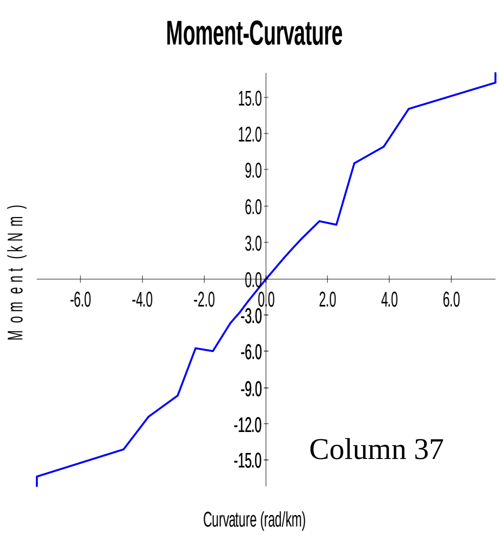

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 32

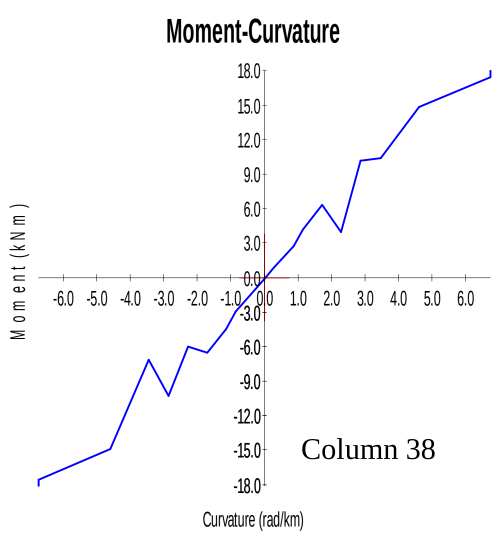

Moment-Curvature
M o m e n t (k N m)
Column 33
Curvature (rad/km)

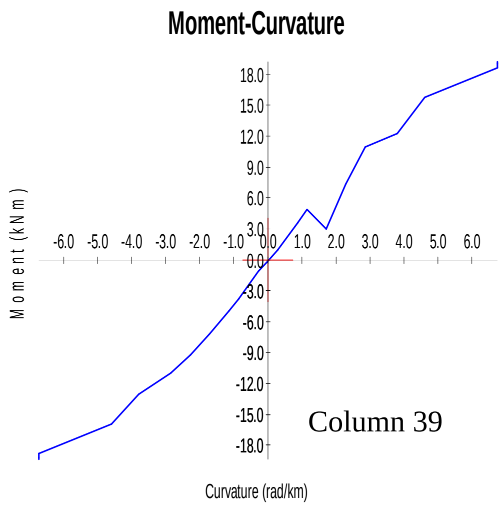

Moment-Curvature
M o m e n t (k N m)
Column 34
Curvature (rad/km)

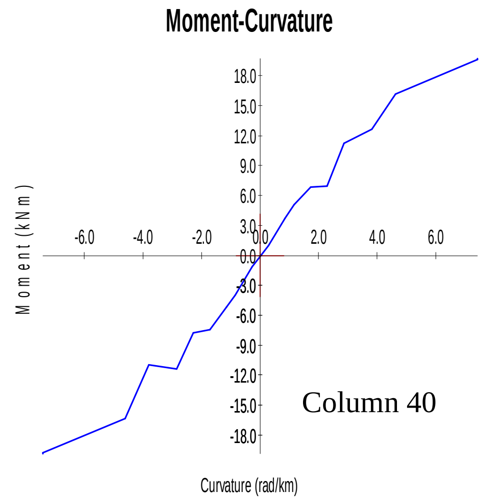

Moment-Curvature
M o m e n t (k N m)
Column 35
Curvature (rad/km)

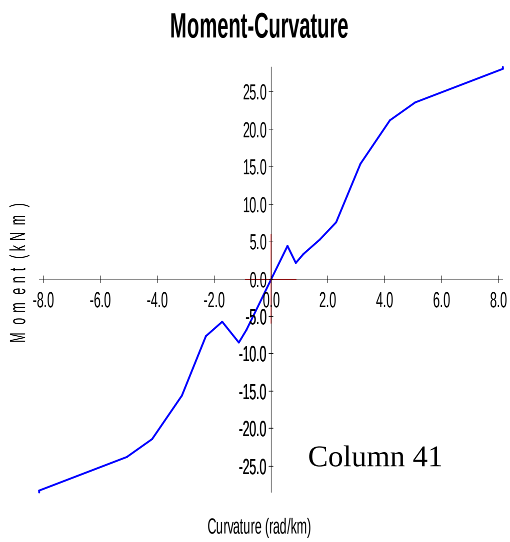

Moment-Curvature
M o m e n t (k N m)
Column 36
Curvature (rad/km)

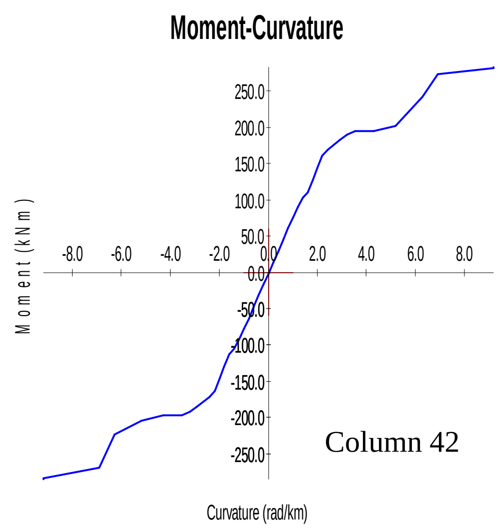

Moment-Curvature
M o m e n t (k N m)
Column 37
Curvature (rad/km)

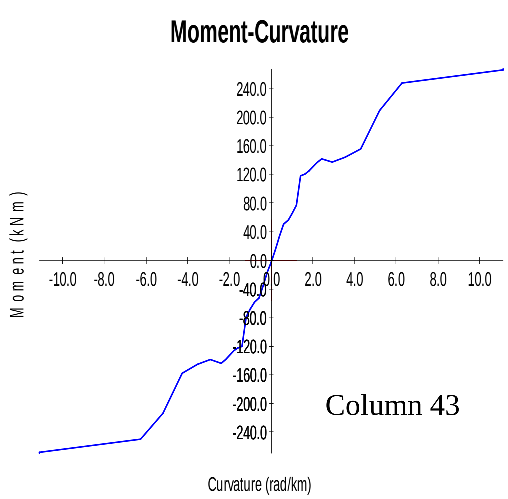

Moment-Curvature
M o m e n t (k N m)
Column 38
Curvature (rad/km)

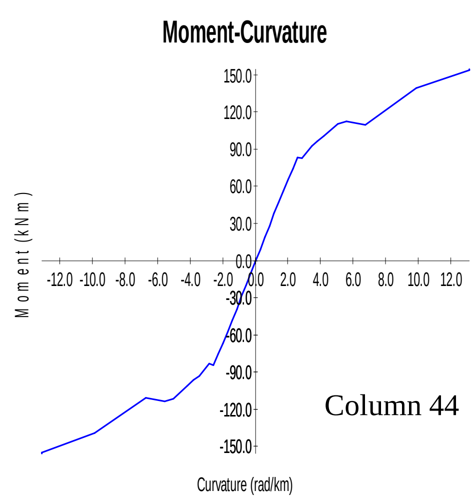

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 39

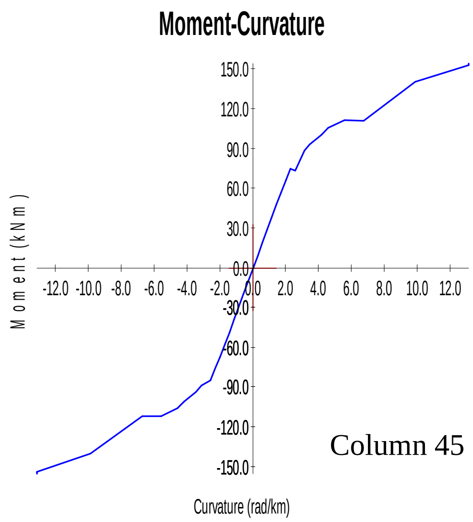

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 40

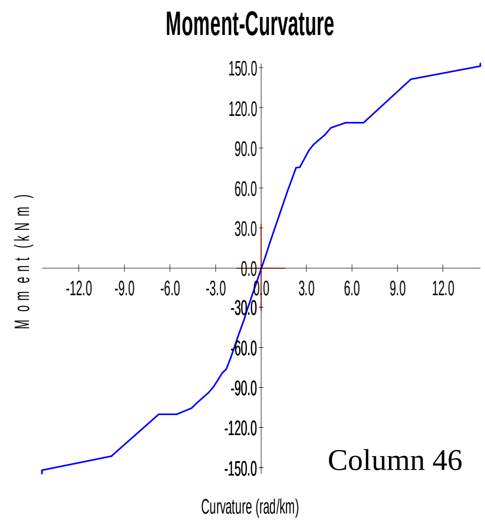

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 41

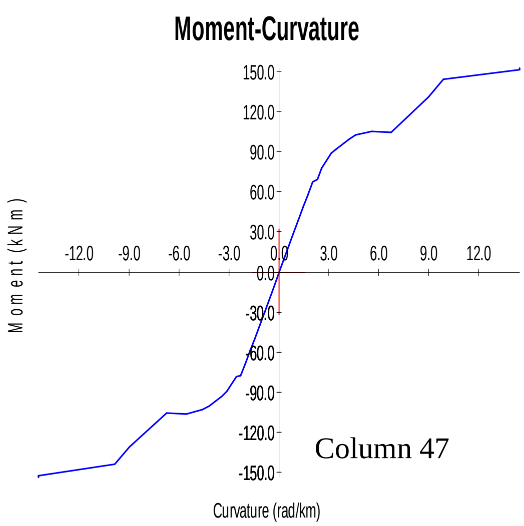

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 42

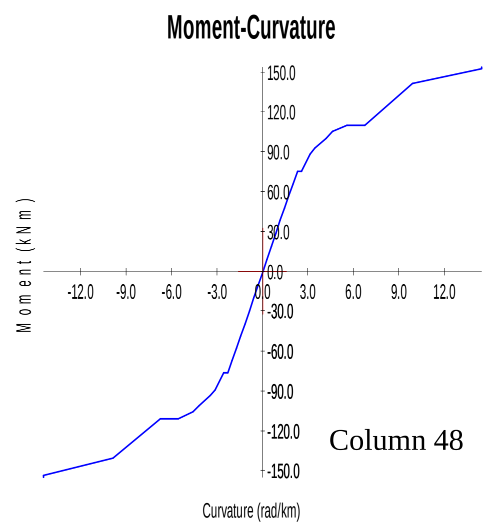

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 43

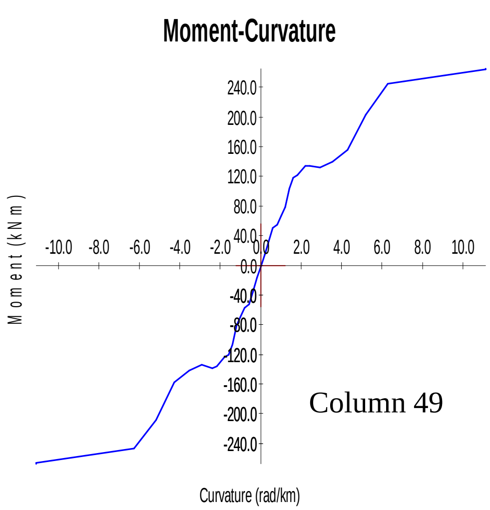

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 44

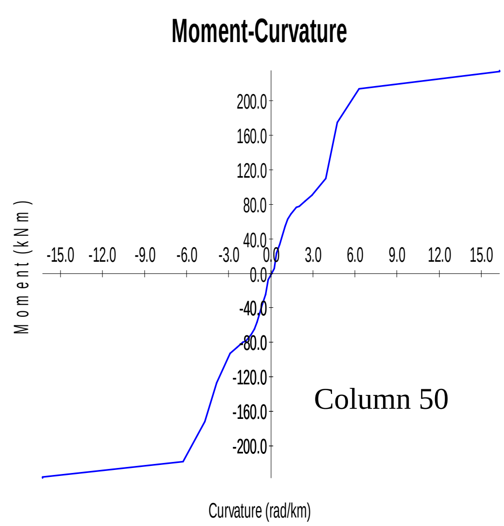

Moment-Curvature
150.0
120.0
90.0
60.0
30.0
0.0
-30.0
-60.0
-90.0
-120.0
-150.0
-12.0 -10.0 -8.0 -6.0 -4.0 -2.0 0.0 2.0 4.0 6.0 8.0 10.0 12.0
M o m e n t (k N m)
Curvature (rad/km)
Column 45

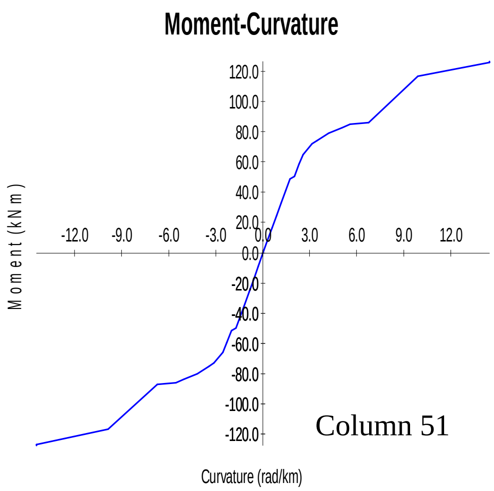

Moment-Curvature
150.0
120.0
90.0
60.0
30.0
0.0
-30.0
-60.0
-90.0
-120.0
-150.0
-12.0 -9.0 -6.0 -3.0 0.0 3.0 6.0 9.0 12.0
M o m e n t (k N m)
Curvature (rad/km)
Column 46

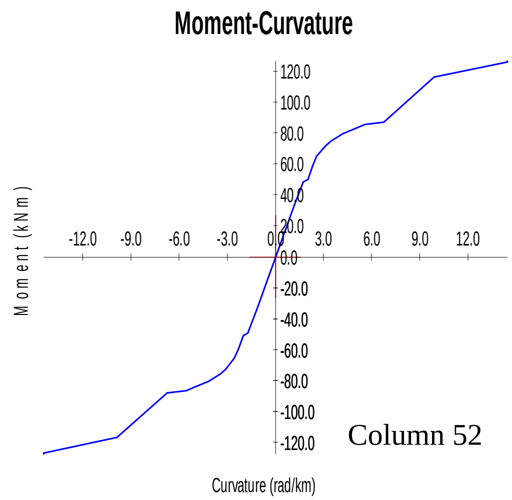

Moment-Curvature
150.0
120.0
90.0
60.0
30.0
0.0
-30.0
-60.0
-90.0
-120.0
-150.0
-12.0 -9.0 -6.0 -3.0 0.0 3.0 6.0 9.0 12.0
M o m e n t (k N m)
Curvature (rad/km)
Column 47

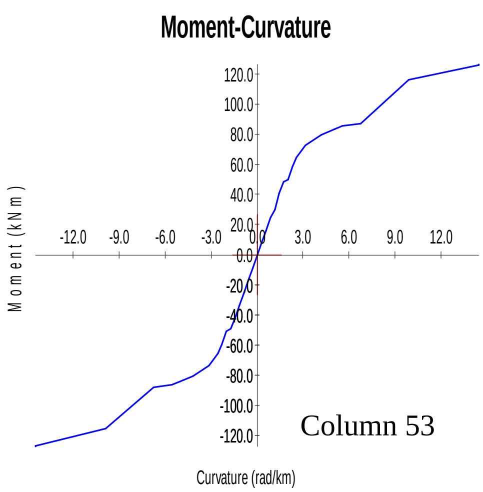

Moment-Curvature
150.0
120.0
90.0
60.0
30.0
0.0
-30.0
-60.0
-90.0
-120.0
-150.0
-12.0 -9.0 -6.0 -3.0 0.0 3.0 6.0 9.0 12.0
M o m e n t (k N m)
Curvature (rad/km)
Column 48

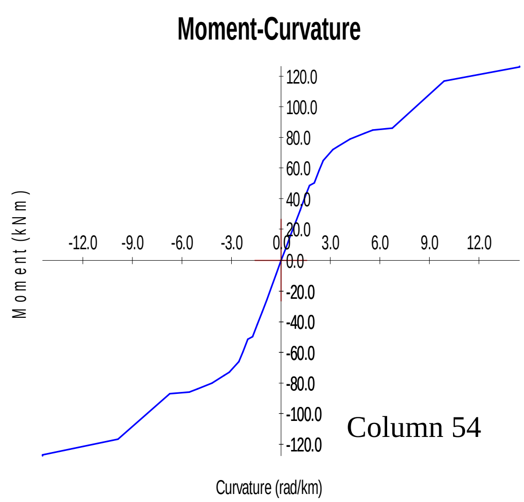

Moment-Curvature
240.0
200.0
160.0
120.0
80.0
40.0
0.0
-40.0
-80.0
-120.0
-160.0
-200.0
-240.0
-10.0 -8.0 -6.0 -4.0 -2.0 0.0 2.0 4.0 6.0 8.0 10.0
M o m e n t (k N m)
Curvature (rad/km)
Column 49

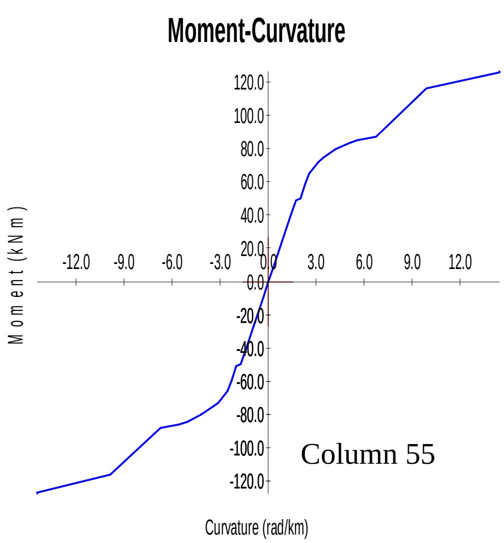

Moment-Curvature
200.0
160.0
120.0
80.0
40.0
0.0
-40.0
-80.0
-120.0
-160.0
-200.0
-15.0 -12.0 -9.0 -6.0 -3.0 0.0 3.0 6.0 9.0 12.0 15.0
M o m e n t (k N m)
Curvature (rad/km)
Column 50

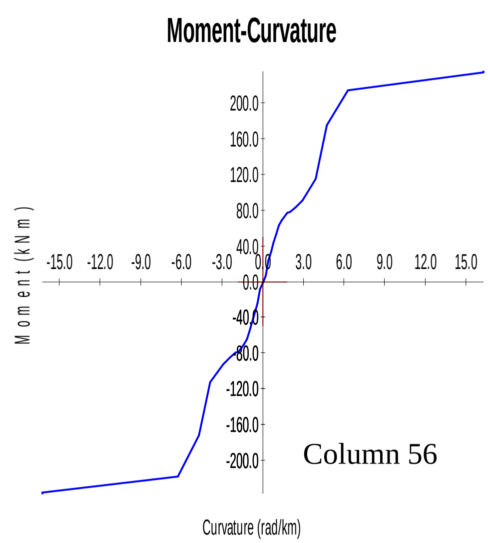

Moment-Curvature
Moment (kN m)
Curvature (rad/km)
Column 51

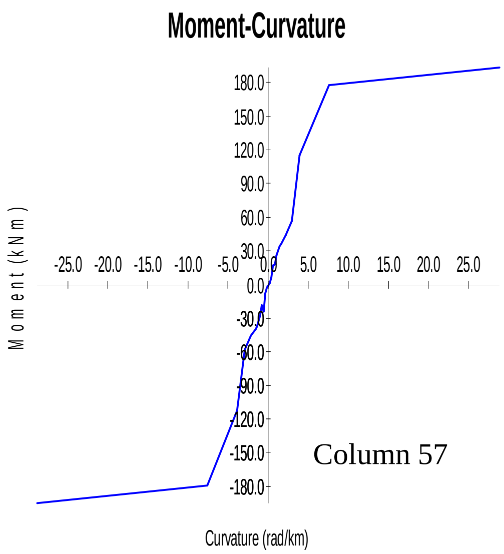

Moment-Curvature
Moment (kN m)
Curvature (rad/km)
Column 52

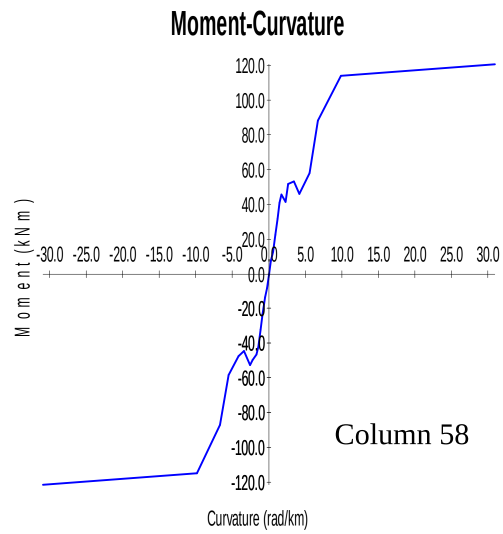

Moment-Curvature
Moment (kN m)
Curvature (rad/km)
Column 53

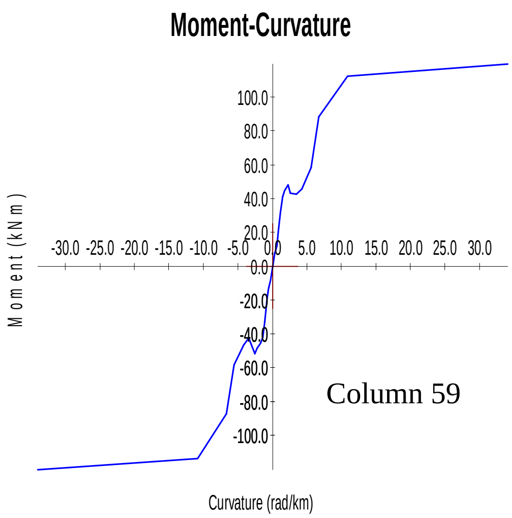

Moment-Curvature
Moment (kN m)
Curvature (rad/km)
Column 54

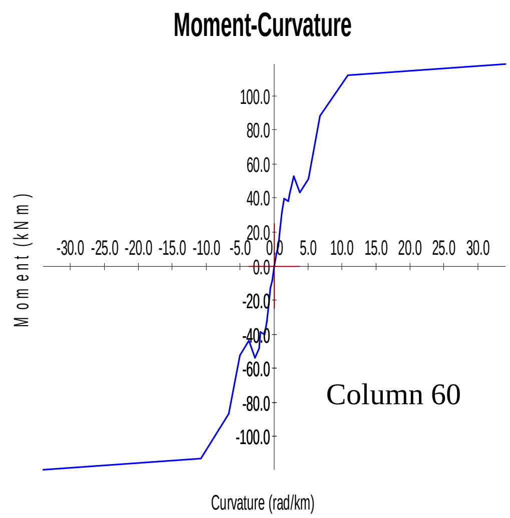

Moment-Curvature
Moment (kN m)
Curvature (rad/km)
Column 55

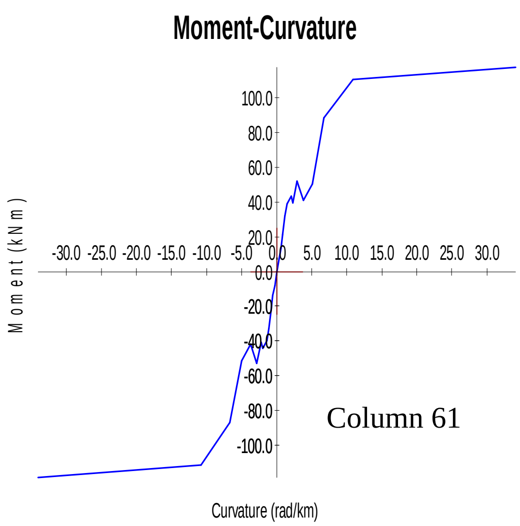

Moment-Curvature
Moment (kN m)
Curvature (rad/km)
Column 56

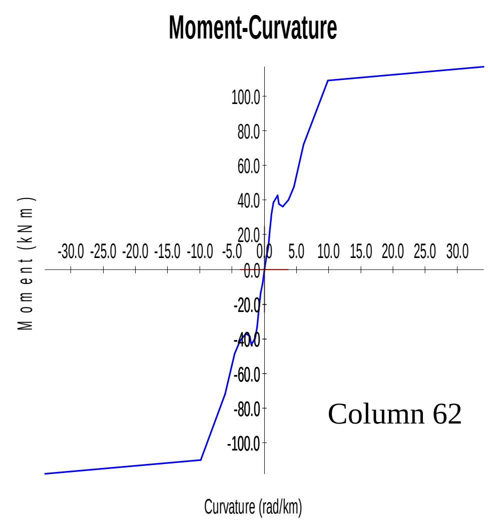

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 57

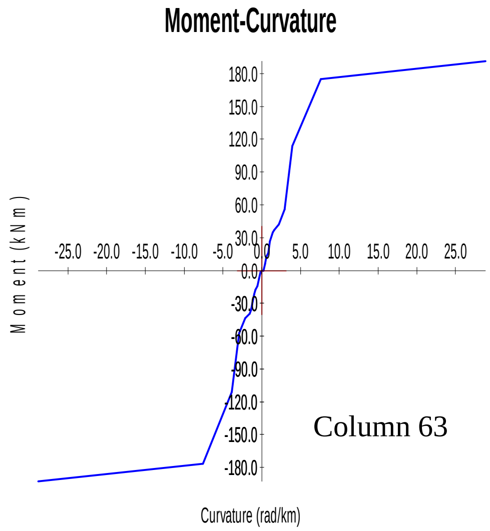

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 58

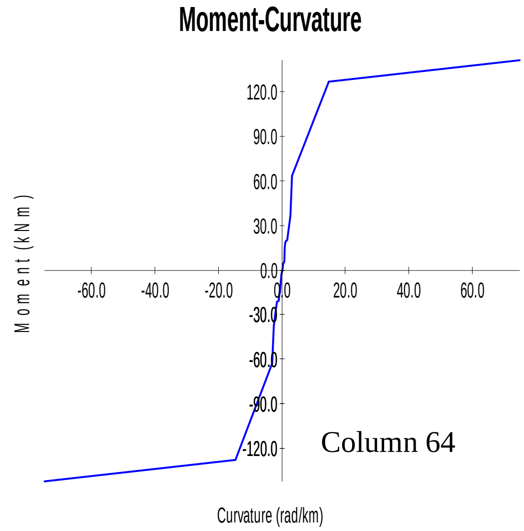

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 59

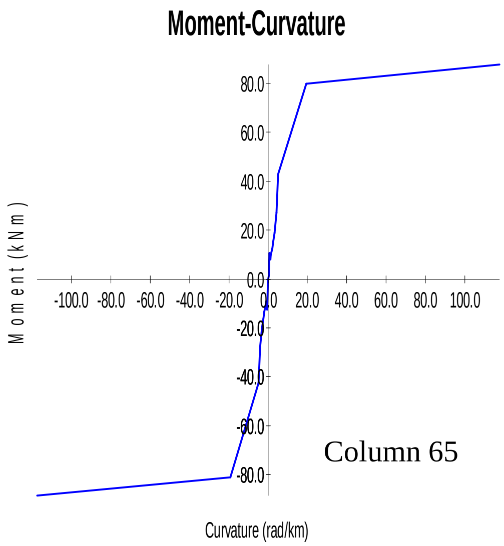

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 60

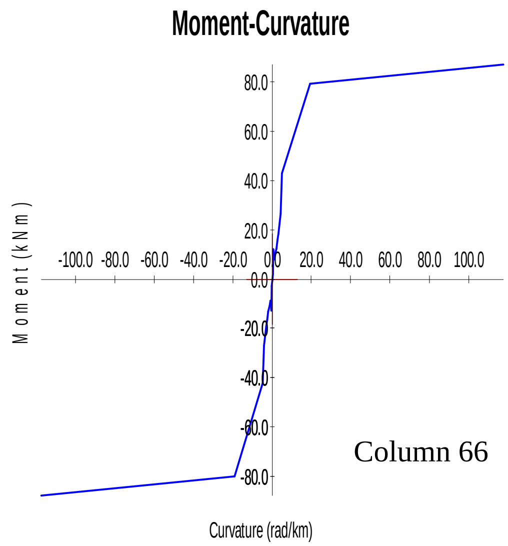

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 61

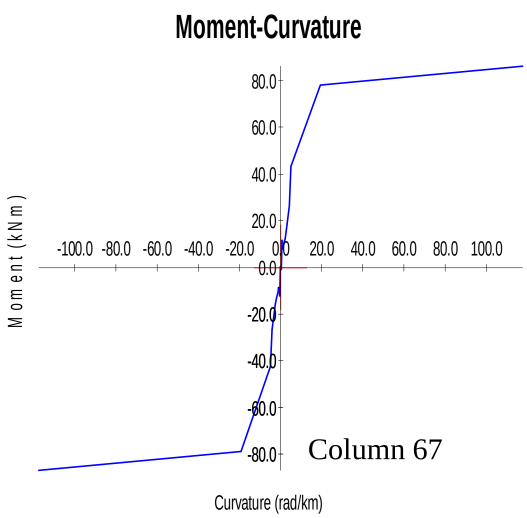

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 62

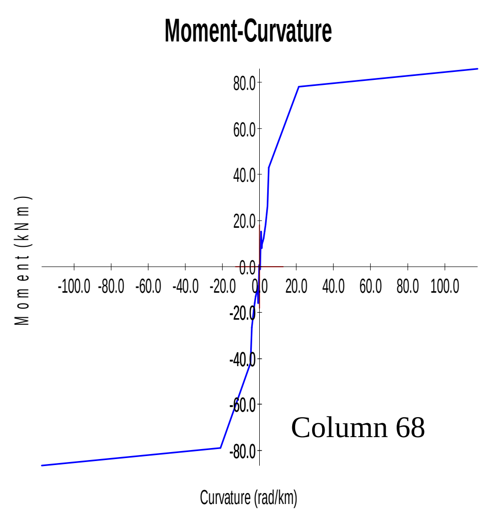

Moment-Curvature
M o m e n t (k N m)
180.0 150.0 120.0 90.0 60.0 30.0 0.0 -30.0 -60.0 -90.0 -120.0 -150.0 -180.0
-25.0 -20.0 -15.0 -10.0 -5.0 0.0 5.0 10.0 15.0 20.0 25.0
Column 63
Curvature (rad/km)

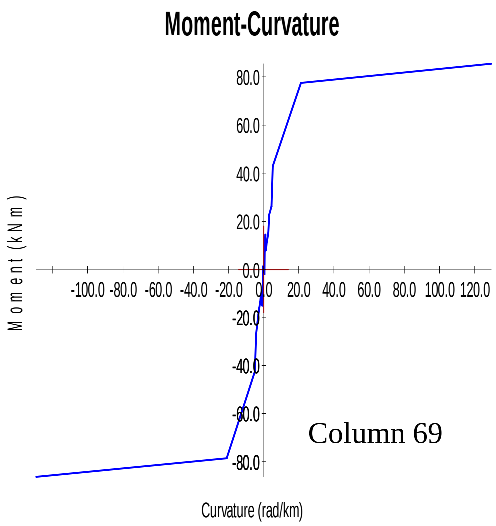

Moment-Curvature
M o m e n t (k N m)
120.0 90.0 60.0 30.0 0.0 -30.0 -60.0 -90.0 -120.0
-60.0 -40.0 -20.0 0.0 20.0 40.0 60.0
Column 64
Curvature (rad/km)

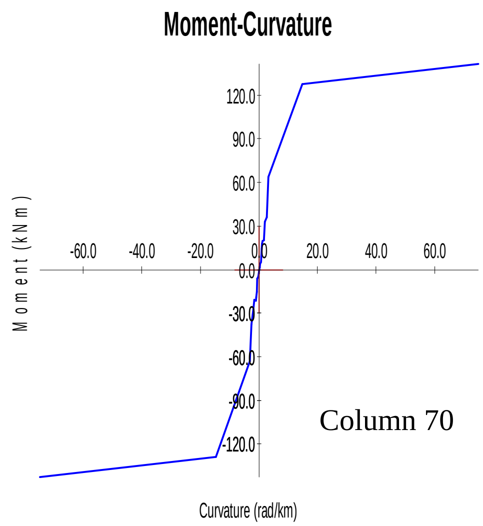

Moment-Curvature
M o m e n t (k N m)
80.0 60.0 40.0 20.0 0.0 -20.0 -40.0 -60.0 -80.0
-100.0 -80.0 -60.0 -40.0 -20.0 0.0 20.0 40.0 60.0 80.0 100.0
Column 65
Curvature (rad/km)


Moment-Curvature
M o m e n t (k N m)
80.0 60.0 40.0 20.0 0.0 -20.0 -40.0 -60.0 -80.0
-100.0 -80.0 -60.0 -40.0 -20.0 0.0 20.0 40.0 60.0 80.0 100.0
Column 66
Curvature (rad/km)

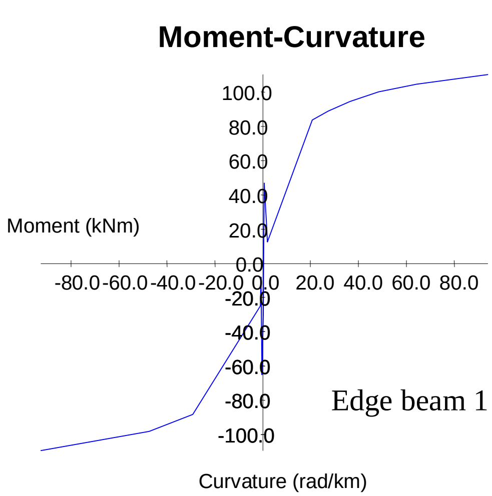

Moment-Curvature
M o m e n t (k N m)
80.0 60.0 40.0 20.0 0.0 -20.0 -40.0 -60.0 -80.0
-100.0 -80.0 -60.0 -40.0 -20.0 0.0 20.0 40.0 60.0 80.0 100.0
Column 67
Curvature (rad/km)

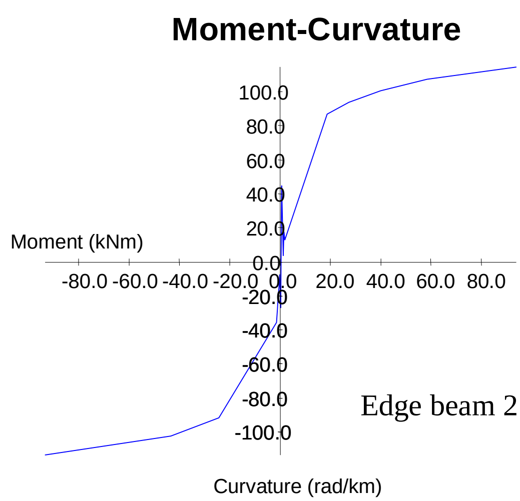

Moment-Curvature
M o m e n t (k N m)
80.0 60.0 40.0 20.0 0.0 -20.0 -40.0 -60.0 -80.0
-100.0 -80.0 -60.0 -40.0 -20.0 0.0 20.0 40.0 60.0 80.0 100.0
Column 68
Curvature (rad/km)

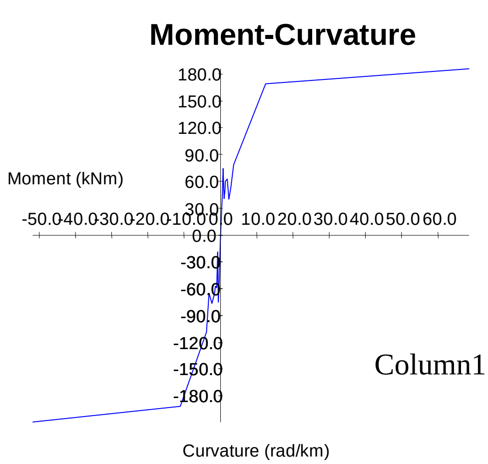


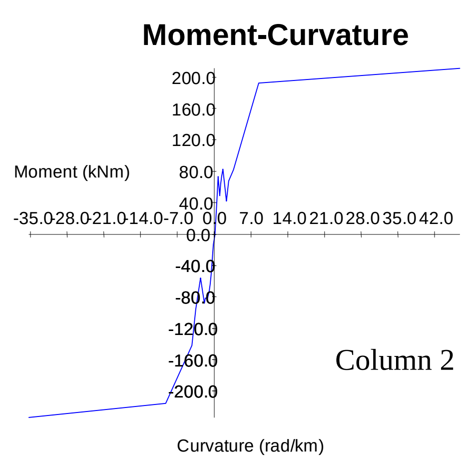


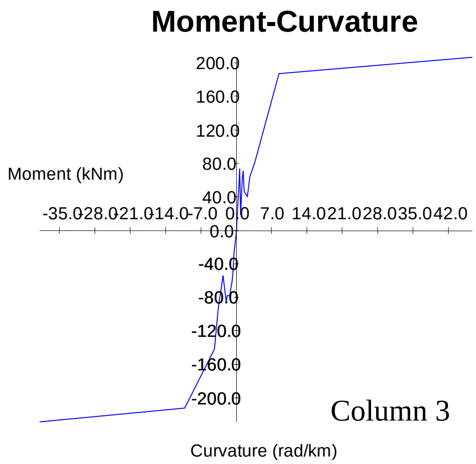
B. Moment-curvature for Y-Direction frame elements.



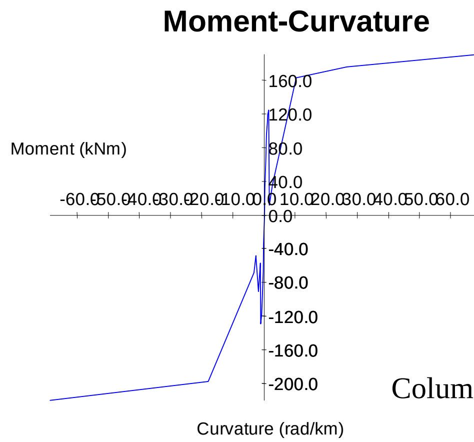




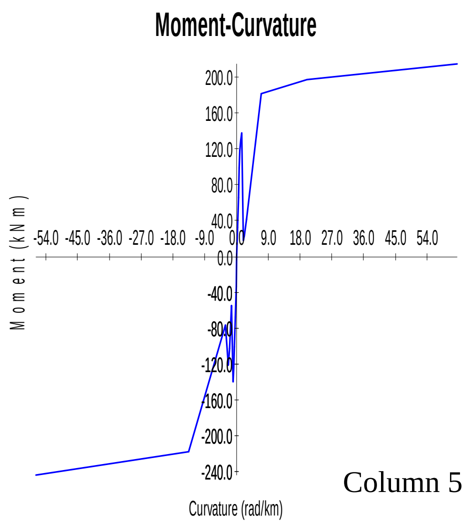




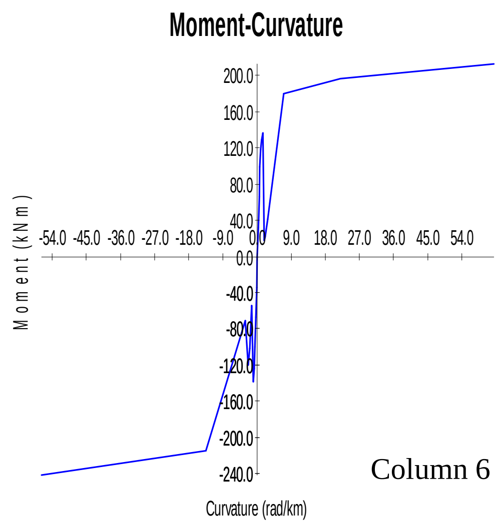
Column 3



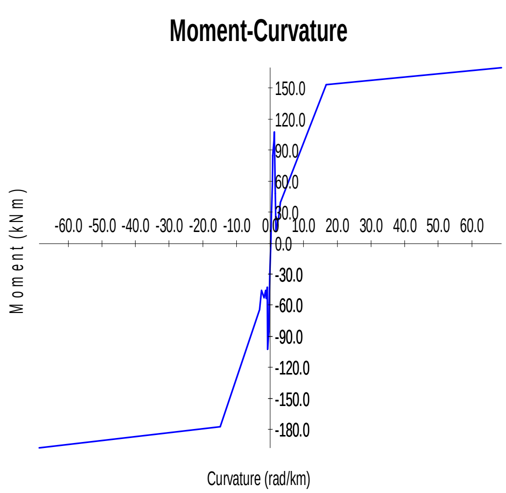
Column 4



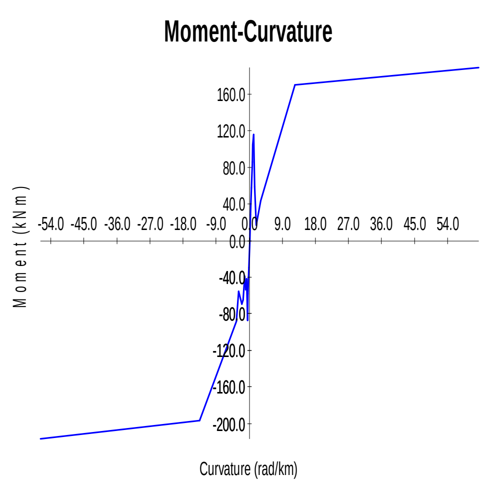
Column 5



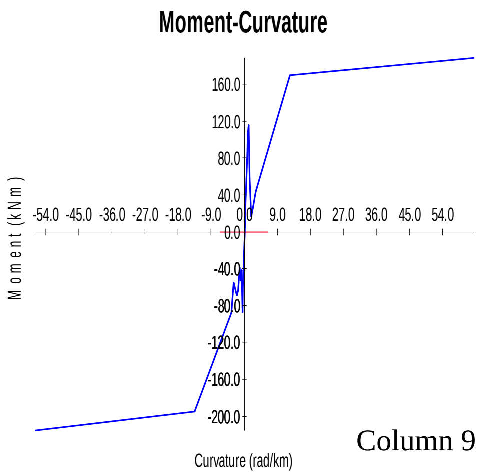
Column 6

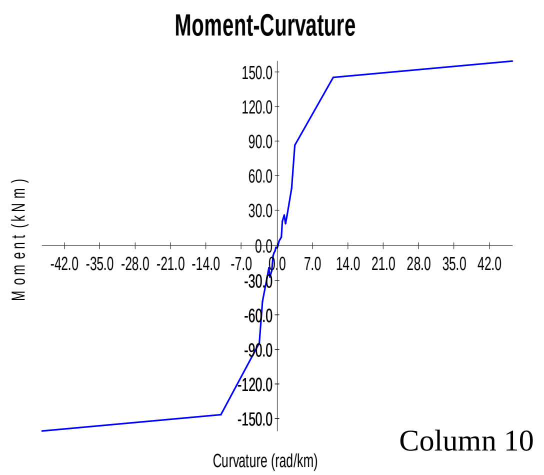


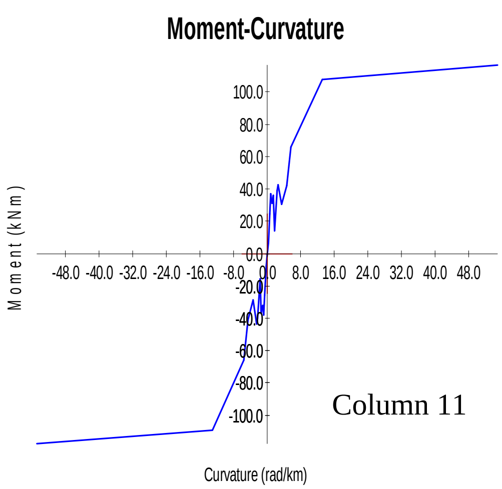
Column 7

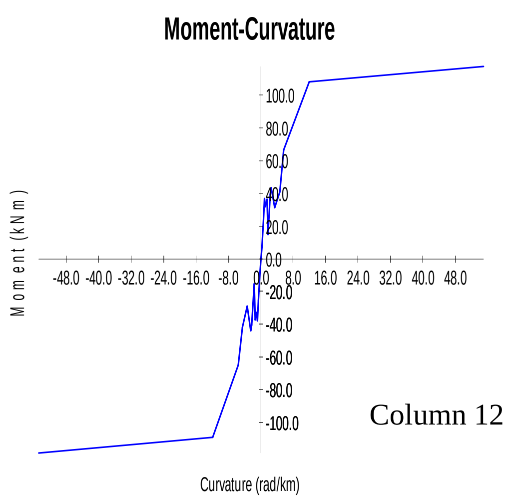


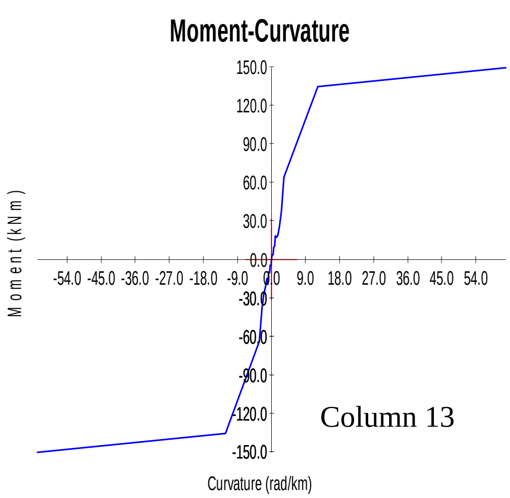
Column 8

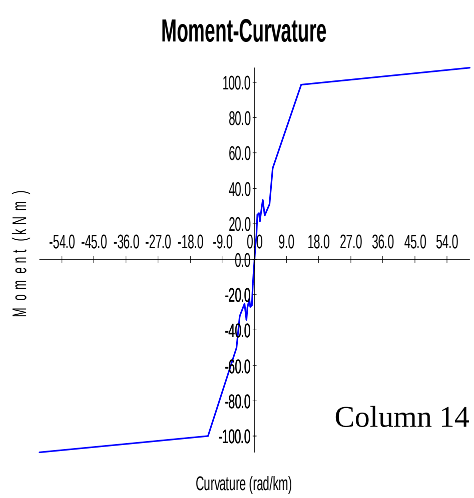

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 9

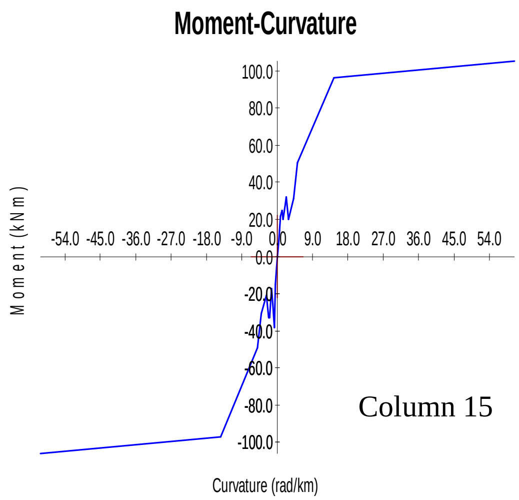

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 10

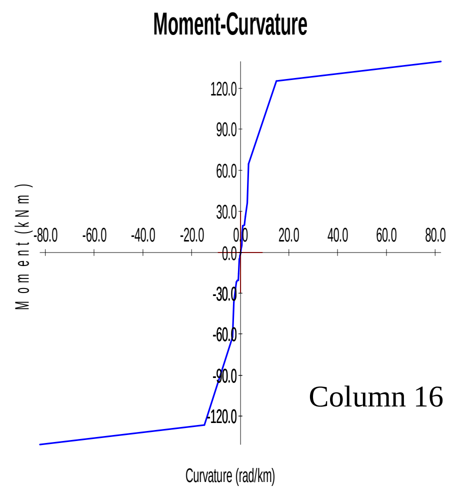

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 11

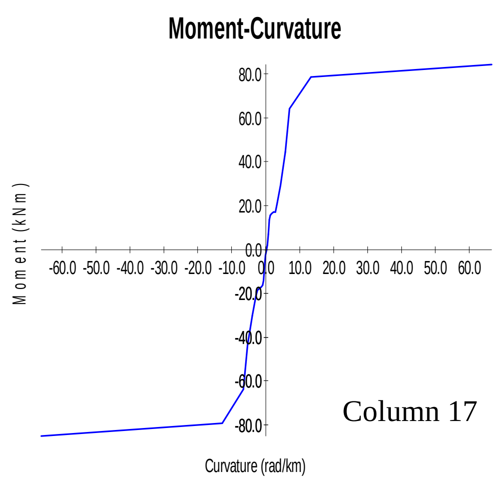

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 12

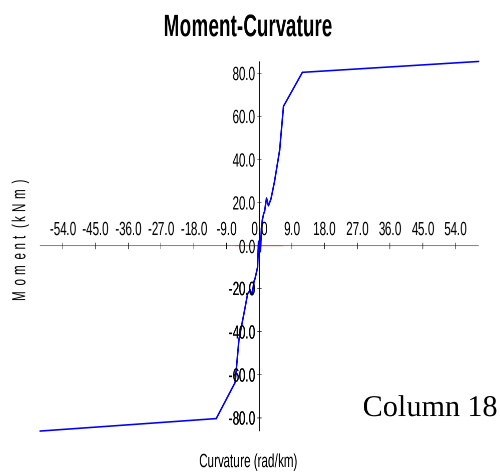

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 13

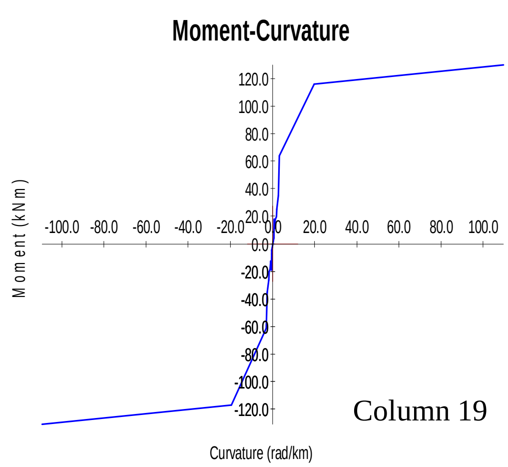

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
Column 14

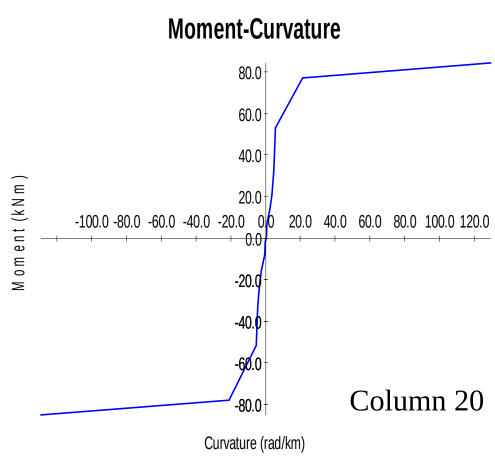

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
100.0
80.0
60.0
40.0
20.0
0.0
-20.0
-40.0
-60.0
-80.0
-100.0
-54.0 -45.0 -36.0 -27.0 -18.0 -9.0 0.0 9.0 18.0 27.0 36.0 45.0 54.0
Column 15

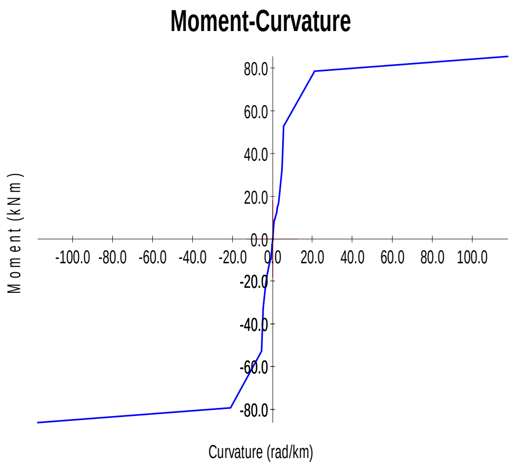

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
120.0
90.0
60.0
30.0
0.0
-30.0
-60.0
-90.0
-120.0
-80.0 -60.0 -40.0 -20.0 0.0 20.0 40.0 60.0 80.0
Column 16

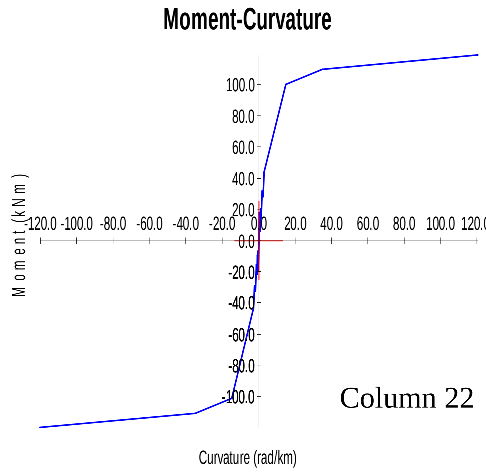

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
80.0
60.0
40.0
20.0
0.0
-20.0
-40.0
-60.0
-80.0
-60.0 -50.0 -40.0 -30.0 -20.0 -10.0 0.0 10.0 20.0 30.0 40.0 50.0 60.0
Column 17


Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
80.0
60.0
40.0
20.0
0.0
-20.0
-40.0
-60.0
-80.0
-54.0 -45.0 -36.0 -27.0 -18.0 -9.0 0.0 9.0 18.0 27.0 36.0 45.0 54.0
Column 18

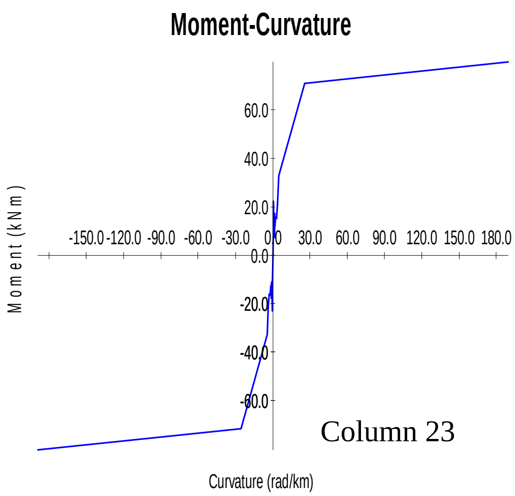

Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
120.0
100.0
80.0
60.0
40.0
20.0
0.0
-20.0
-40.0
-60.0
-80.0
-100.0
-120.0
-100.0 -80.0 -60.0 -40.0 -20.0 0.0 20.0 40.0 60.0 80.0 100.0
Column 19


Moment-Curvature
M o m e n t (k N m)
Curvature (rad/km)
80.0
60.0
40.0
20.0
0.0
-20.0
-40.0
-60.0
-80.0
-100.0 -80.0 -60.0 -40.0 -20.0 0.0 20.0 40.0 60.0 80.0 100.0 120.0
Column 20

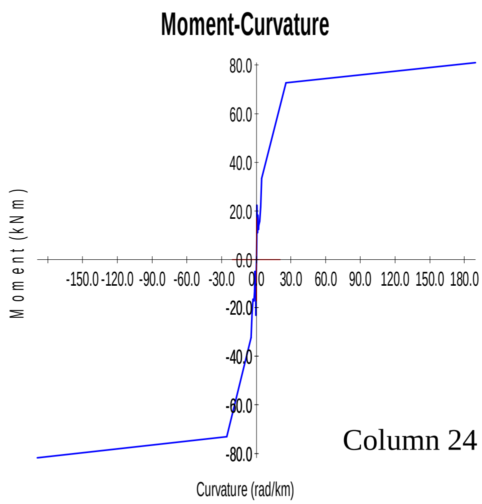




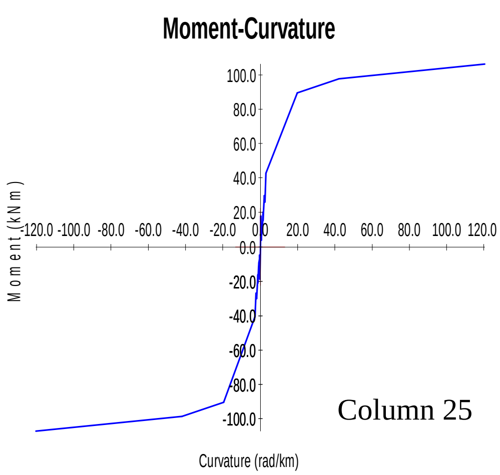
Column 22



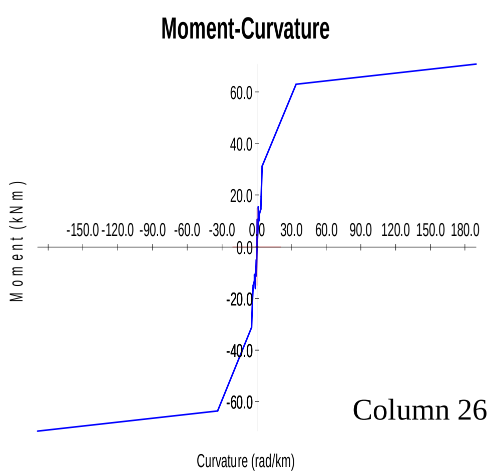
Column 23



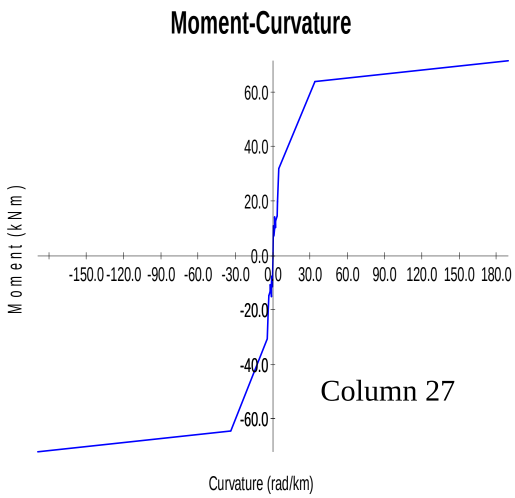
Column 24

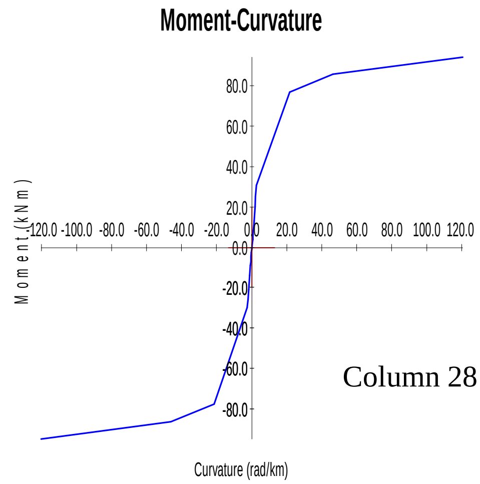


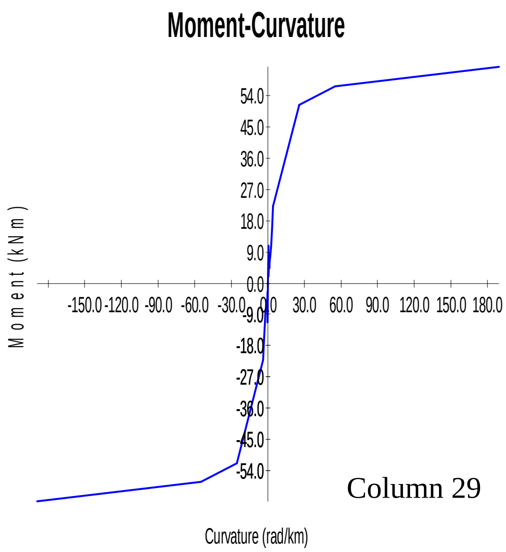
Column 25

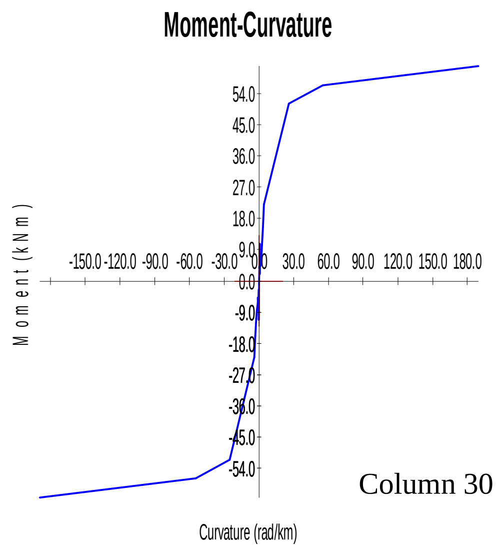


Column 26


Moment-Curvature
60.0
40.0
20.0
0.0
-20.0
-40.0
-60.0
-150.0 -120.0 -90.0 -60.0 -30.0 0.0 30.0 60.0 90.0 120.0 150.0 180.0
M o m e n t (k N m)
Curvature (rad/km)
Column 27


Moment-Curvature
80.0
60.0
40.0
20.0
0.0
-20.0
-40.0
-60.0
-80.0
-120.0 -100.0 -80.0 -60.0 -40.0 -20.0 0.0 20.0 40.0 60.0 80.0 100.0 120.0
M o m e n t (k N m)
Curvature (rad/km)
Column 28


Moment-Curvature
54.0
45.0
36.0
27.0
18.0
9.0
0.0
-9.0
-18.0
-27.0
-36.0
-45.0
-54.0
-150.0 -120.0 -90.0 -60.0 -30.0 0.0 30.0 60.0 90.0 120.0 150.0 180.0
M o m e n t (k N m)
Curvature (rad/km)
Column 29


Moment-Curvature
54.0
45.0
36.0
27.0
18.0
9.0
0.0
-9.0
-18.0
-27.0
-36.0
-45.0
-54.0
-150.0 -120.0 -90.0 -60.0 -30.0 0.0 30.0 60.0 90.0 120.0 150.0 180.0
M o m e n t (k N m)
Curvature (rad/km)
Column 30

Appendix D: FEMA440 Parameters to find Target displacements.

Table D.1: X-Direction before strengthening

Item	Value
C0	1.15
C1	1.00
C2	1.00
C3	1.00
Sa	1.10
Te	0.18
Ti	0.18
Ki	575484.90
Ke	575484.90
Alpha	0.002
R	1.15
Vy	1100.00
Dy	1.00
Weight	1145.00
Cm	1.00

Table D.2: Y-Direction before Strengthening

Item	Value
C0	1.13
C1	1.20
C2	1.05
C3	1.00
Sa	1.10
Te	0.52
Ti	0.52
Ki	6718.91
Ke	6718.91
Alpha	0.051
R	1.10
Vy	310
Dy	0.037
Weight	310.56
Cm	1.00

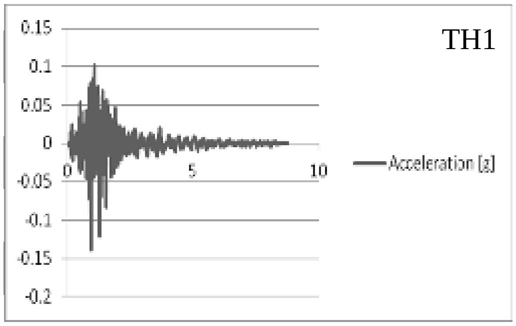
Table D.3: X-direction after strengthening

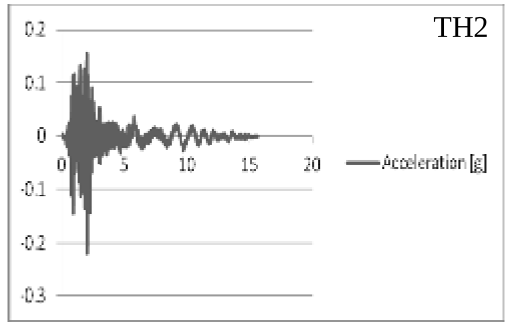
Item	Value
C0	1.35
C1	1.25
C2	1.00
C3	1.00
Sa	1.10
Te	0.15
Ti	0.15
Ki	575484.90
Ke	575484.90
Alpha	0.002
R	1.15
Vy	1100.00
Dy	1.00
Weight	1145.00
Cm	1.00

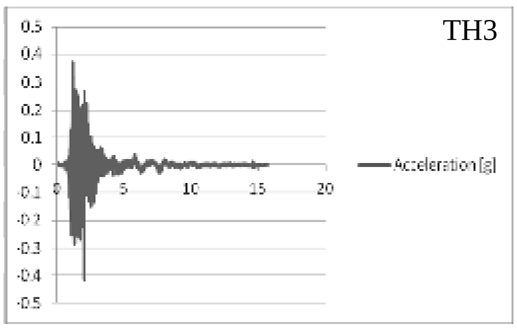
Table D.4: Y-direction after strengthening

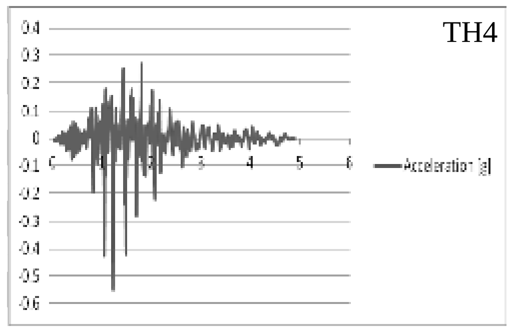
Item	Value
C0	1.08
C1	1.06
C2	1.00
C3	2.53
Sa	1.10
Te	0.27
Ti	0.27
Ki	6542.32
Ke	6542.32
Alpha	0.98
R	1.28
Vy	79.74
Dy	0.01
Weight	1001.06
Cm	1.00

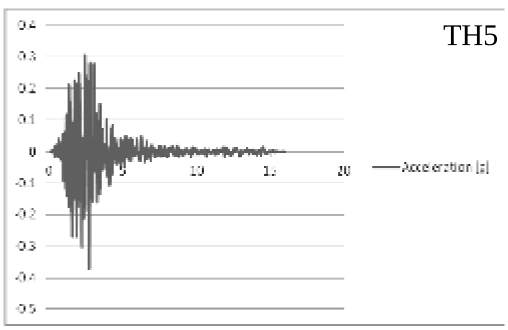
Appendix E: Time history functions of selected earthquake records.

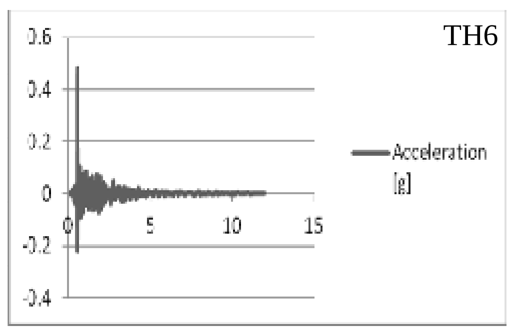


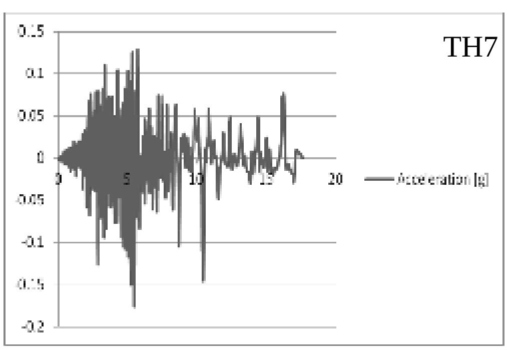


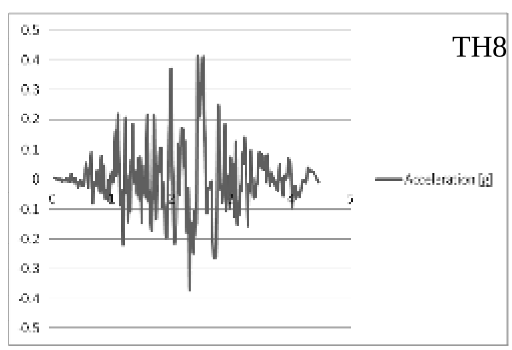


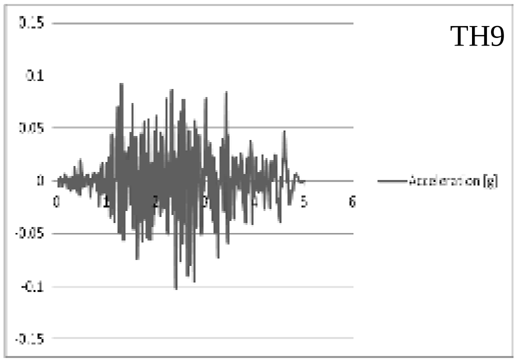


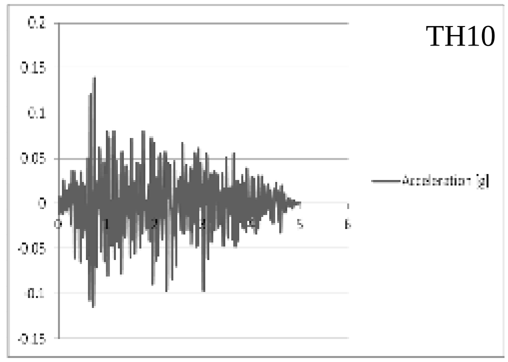


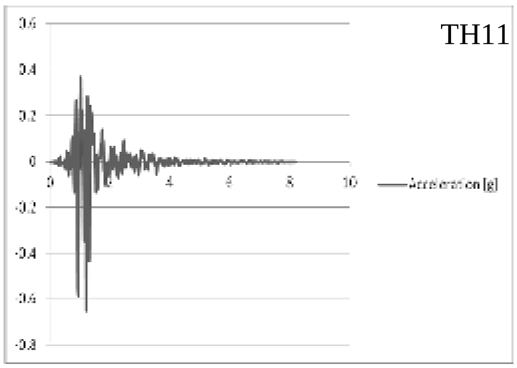


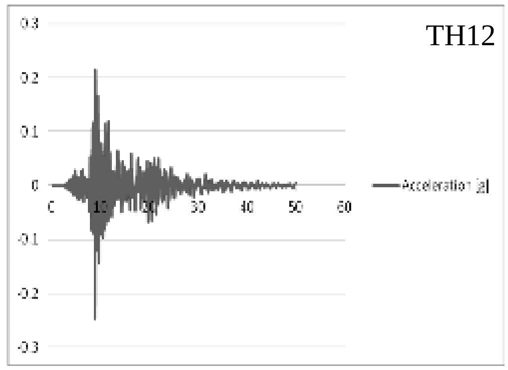


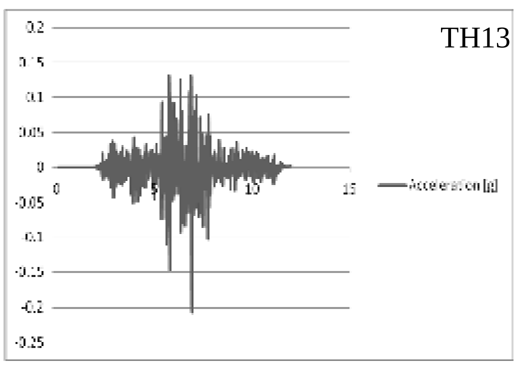


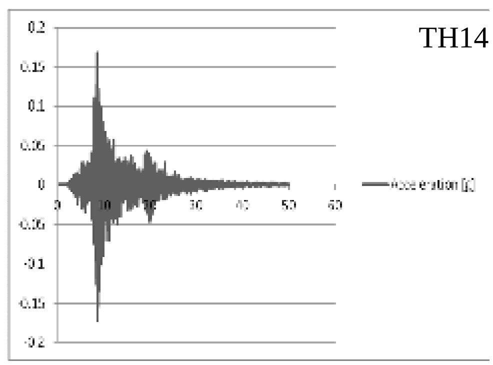

TH9
Acceleration [g]

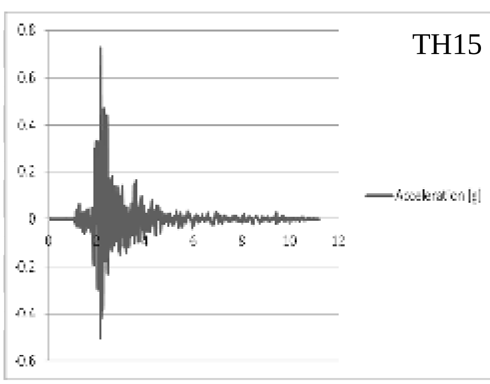

TH10
Acceleration [g]

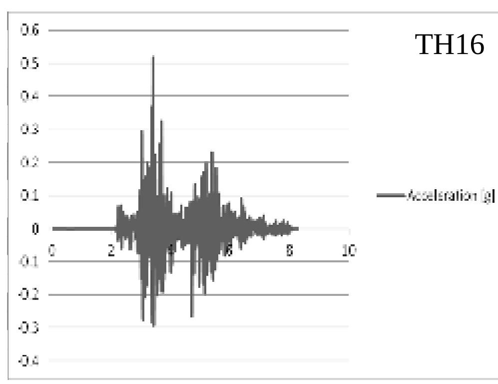

TH11
Acceleration [g]

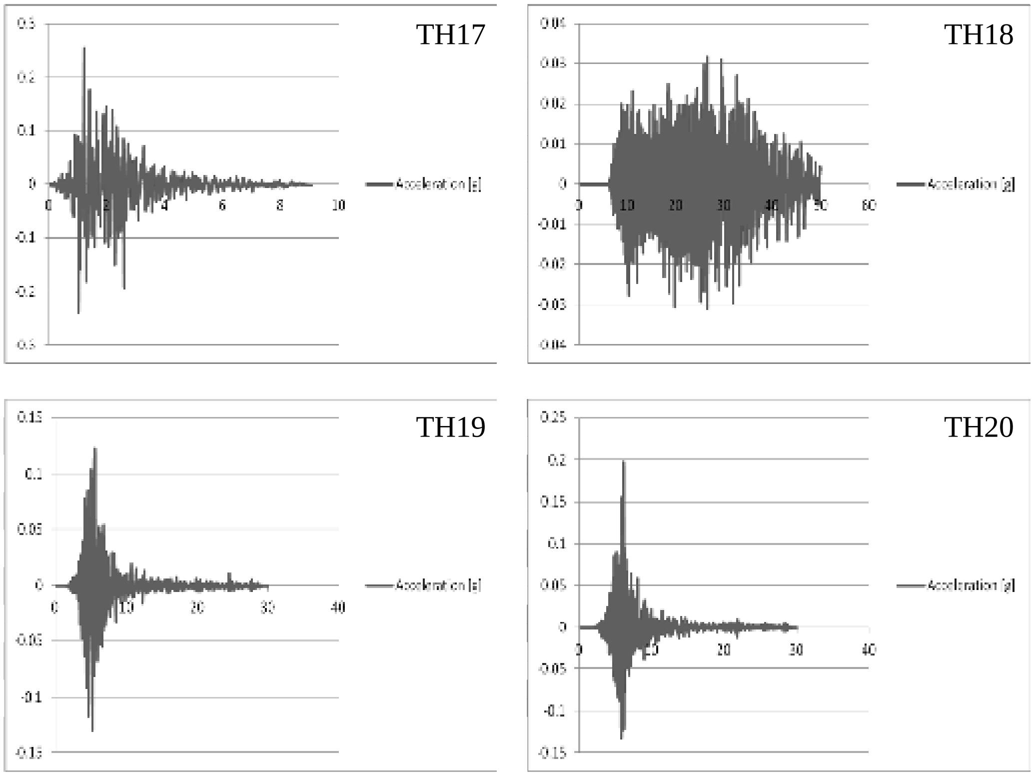

TH12
Acceleration [g]


TH13
Acceleration [g]


TH14
Acceleration [g]


TH15
Acceleration [g]


TH16
Acceleration [g]



<u>Note :</u> show the selected groun motion records for each time histories in the next page.

Name	MSE	Scale Factor	Event	Station	Year	Magnitude	Mechanisim	Fualt Distance	PGA(g)
TH1	0.3140	10.8413	Park Field	Cholame - Shandon Array #5	1966	6.19	Strike-Slip	9.6	0.6470
TH2	0.0687	2.6382	Imperial Valley-06	El Centro Array #4	1979	6.53	Strike-Slip	7	0.4741
TH3	0.0703	6.6741	Imperial Valley-06	El Centro Array #5	1979	6.53	Strike-Slip	4	0.5122
TH4	0.1677	4.7040	Victoria-Mexio	Cerro Prieto	1980	6.53	Strike-Slip	14.4	0.4950
TH5	0.0869	2.7845	Wesmorland	Westmorland Fire Sta	1981	5.90	Strike-Slip	6.5	0.4788
TH6	0.3016	16.8445	Morgan Hill	Morgan Hill	1984	6.19	Strike-Slip	13.7	0.5485
TH7	0.1464	2.6054	Superstition Hills-02	Wildlife Liquef. Array	1987	6.54	Strike-Slip	23.9	0.5476
TH8	0.1332	2.5265	Superstition Hills-02	Parachute Test Site	1987	6.54	Strike-Slip	0.9	0.5226
TH9	0.0671	5.2387	Landers	Amboy	1992	7.28	Strike-Slip	69.2	0.7231
TH10	0.0837	2.7455	Landers	Desert Hot Springs	1992	7.28	Strike-Slip	21.8	0.6087
TH11	0.1118	6.7796	Kobe-Japan	Takarazuka	1995	6.90	Strike-Slip	0.3	0.4981
TH12	0.0892	1.7984	Kocaeli-Turkey	Ambarli	1999	7.51	Strike-Slip	69.6	0.5080
TH13	0.0403	1.8289	Kocaeli-Turkey	Arcelik	1999	7.51	Strike-Slip	13.5	0.5098
TH14	0.2194	3.2088	Kocaeli-Turkey	Cekmece	1999	7.51	Strike-Slip	66.7	0.4887
TH15	0.0700	16.3662	Duzce-Turkey	Bolu	1999	7.14	Strike-Slip	12	0.6735
TH16	0.2197	7.6520	Duzce-Turkey	Lamont 375	1999	7.14	Strike-Slip	3.9	0.8761
TH17	0.4343	4.8329	Hector Mine	Hector	1999	7.13	Strike-Slip	11.7	0.6601
TH18	0.0523	1.6906	Denali-Alaska	TAPS Pump Station #11	2002	7.90	Strike-Slip	126.4	0.5539
TH19	0.0403	8.8293	Chi-Chi-Taiwan-04	CHY035	1999	6.20	Strike-Slip	25.1	0.4964
TH20	0.0564	12.0436	Chi-Chi-Taiwan-04	CHY028	1999	6.20	Strike-Slip	17.7	0.5495