SEISMIC VULNERABILITY ASSESSMENT OF RC BUILDINGS IN ISLAMABAD

By

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A Thesis submitted in partial fulfillment of
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Submitted by

M. Hamza Saeed Virk

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School of Civil & Environmental Engineering

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DEDICATED

TO

ALL MY TEACHERS AND FAMILY

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ABSTRACT

Moderate to high levels of seismic hazard are present through out the Pakistan and numerous collapsed by Oct. 8, 2005 Earthquake verified the seismic vulnerability of building type present in the country. Seismic retrofitting of existing structures is one of the most effective methods of reducing this risk. However, the seismic performance of the structure may not be improved by retrofitting or rehabilitation unless the engineer selects an appropriate intervention technique based on seismic evaluation of the structure. Current codes (BCP 2007) do not address the evaluation of seismic resistance of existing building stock, not designed in accordance with the philosophies of current seismic provisions.

The primary purpose of this work is to carry seismic evaluation of buildings in Islamabad and propose guidelines for Building Code of Pakistan. Owing to importance of the subject, various organizations in the earthquake threatened countries have come up with documents, which serve as guidelines for the assessment of the strength, expected performance and safety of existing buildings. In this study review of various documents on seismic evaluation of existing buildings from different countries is carried out to identify the most essential components of such a procedure for use in Pakistan and other developing countries, which is not only robust, reliable but also easy to use with available resources. Among these documents, ASCE 31-03 guidelines are found to be most suitable for use in our country.

In this study seismic evaluation of buildings is carried out based on ASCE 31-03 provisions in order to understand the procedure in insight. These guidelines are applied on a Case study building which is an actual building located in F-10 Markaz Islamabad. Building is analyzed for Life Safety performance level and for moderate seismicity. A general checklist is used for Tier 1 analysis. Building was found to be deficient and Tier 2 Evaluation is carried using linear analysis procedures. DCRs for some of the columns exceed the limit at Tier 2 phase so a Tier 3 Evaluation is conducted. At Tier 3 Time History analysis (due to significant higher mode effects) is carried out and building (case study 1) was found to be compliant with the ASCE 31-03 guidelines. Pushover analysis is conducted for a typical 7 story RC building (case study 2). Demand from ATC 40 procedure using BCP (2007) parameters was compared with actual site based demand spectra. ATC 40 procedure using BCP (2007) parameters was found to be conservative. Retrofitting was proposed for the deficiencies identified in case study 2 and formation of plastic hinges showed that retrofitting is adequate.

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LIST OF ABBREVIATIONS AND SYMBOLS

ABBREVIATION	DESCRIPTION
A_c	Summation of Area of all column in a story under consideration
ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
ATC	Applied Technology Council California USA
BCP	Building Code of Pakistan 2007
C	Compliant
C_p	Horizontal Force Factor
C_{vx}	Vertical distribution factor based on storey weight and height
DCR	Demand Capacity Ration
D_r	Drift Ratio
E	Modulus of Elasticity
$f^{/}c$	Specified Concrete compressive strength
f_y	yield strength
FEMA	Federal Emergency Management Authority USA
h	Story height
h_i, h_x	Height from base to floor level i & x
h_n	Height above the base to roof level
G	Shear modulus
I	Moment of inertia
IBC	International Building Code
IMRF	Intermediate Moment Resisting Frame
IO	Immediate Occupancy
J	Polar moment of Inertia
k_b	Stiffness of representative beam
k_c	Stiffness of representative column
Kip	Kilo pounds
kcf	Kip per cubic feet

LS	Life Safety
m	Component Modification Factor
MCE	Maximum considered Earthquake
MDOF	Multi degree of freedom
M_x	Overturning moment about x-axis
M_y	Overturning moment about y-axis
n	Number of stories above ground
n_c	Total number of columns
OBE	Operating Basis Earthquake
PGA	Peak Ground Acceleration
Q_{CE}	Expected Strength
Q_E	Action due to Earthquake Loads
Q_G	Action due to Gravity Loads
Q_L	Action due to effective live loads
Q_{UD}	Deformation -controlled Design Actions
Q_{UF}	Force-controlled Design Actions
RCC	Reinforced Cement Concrete
RS	Response Spectrum
S_1	Spectral acceleration at period of 1 sec
Sa	Response spectral acceleration
SDOF	Single degree of freedom
SRSS	Square root of sum of squares
Ss	Short spectral acceleration at period of 0.2 sec
S_u	Un-drained shear strength
Sv	Response spectral velocity
UBC	Uniform Building Code
V_S	Shear Velocity of rock or soil
V_x	Base shear in x-direction
V_x	Base shear in y-direction
Δx	Storey drift in x-direction
Δy	Storey drift in y-direction

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Moderate to high levels of seismic hazard are present through out the Pakistan and numerous collapsed by Oct. 8, 2005 Earthquake verified the seismic vulnerability of building type present in the country. This event made it abundantly clear that, earthquake provisions of the Pakistan Building Code 1986 (based on UBC 1973) need to be revised so that public health and safety for all communities is ensured. The Building authorities responded to the cause and successfully accomplished the task of revising the older seismic provisions and introduced Building Code of Pakistan (BCP) 2007. Modern Code, based on UBC 97, ACI 318-05 and AISC 341-05, focus on better behavior and performance to reduce and limit large losses in future. While the main focus has been on improving seismic provisions for new buildings, current codes do not address the seismic evaluation of existing building stock, not designed in accordance with the philosophies of current seismic provisions.

Islamabad is the capital city of Pakistan and lies in seismic zone 2B as per seismic map given in seismic provisions in Building Code of Pakistan (BCP) 2007. The city has a good number of moderate to high rise buildings most of which were constructed prior to 2007 when the seismic provisions of BCP were based on UBC 1973 and was even not mandatory to follow. In addition, the major earthquakes during recent years have underscored the importance of mitigation to reduce seismic risk. Seismic retrofitting of existing structures is one of the most effective methods of reducing this risk. However, the seismic performance of the structure may not be improved by retrofitting unless the engineer selects an appropriate intervention technique based on seismic evaluation of the structure.

Owing to importance of the subject, various organizations in the earthquake threatened countries have come up with documents, which serve as guidelines for the seismic vulnerability assessment of existing buildings as well as for carrying out the necessary rehabilitation, if required. These guidelines are based on the lessoned learned in the past and on latest performance based seismic analysis. In this study review of various documents on seismic evaluation of existing buildings from different countries is carried out. This comparative review of various evaluation schemes will help to identify the most essential components of such a procedure for use in Pakistan, which is not only robust, reliable but also easy to use with available resources.

ASCE 31-03 is the most updated document in USA for Seismic Evaluation of Buildings. This guideline is structured in three tiers of increasing analytical detail and decreasing conservativism towards safety. Prior to Tier 1 Evaluation, all available documents are reviewed and the level of performance desired (Life Safety or Immediate Occupancy), the region of seismicity (low, moderate, or high) and the building type is defined. Tier 1 include configuration related checks and global strength checks. The Tier 2 Evaluation Phase is intended to further investigate potential deficiencies identified in the Tier 1 Screening Phase by conducting Linear Analysis (static or dynamic). For buildings requiring further investigation, a Tier 3 Evaluation is completed. At Tier 3 detailed nonlinear static (pushover) or nonlinear dynamic (Time History) analysis is carried out. Inelastic time analysis is most accurate method; however, the use of inelastic time history analysis is limited because dynamic response is very sensitive to modeling and ground motion characteristics. Inelastic static analysis, or pushover analysis, is preferred method for seismic performance evaluation due to its simplicity. Conventional Pushover analysis is not suitable, where higher mode effects are significant. In this study for case study 1, Time History analysis is performed (as higher mode effects are significant). Pushover analysis using SAP2000 is performed for Case Study 2.

1.2 PROBLEM STAEMENT

Oct. 8, 2005 Earthquake verified the seismic vulnerability of building type present in the country. Shortly thereafter code writers revised the design provision of new buildings to provide adequate ductility to resist strong ground motion. There remain nonetheless thousands of square feet of non-ductile concrete buildings. The consequences of neglecting this general risk may be inevitably catastrophic. Seismic retrofitting of existing structures is one of the most effective methods of reducing this risk. The potential defects in these buildings are often not readily apparent. Retrofit for all the existing structures, is an unacceptable economic burden. Unfortunately guidelines to identify and to retrofit efficiently those that are vulnerable to collapse have not been included in Building Code of Pakistan. It is necessary to incorporate guidelines for seismic evaluation of buildings in our building code. Islamabad has a number of moderate to high rise buildings which were constructed before Oct. 8 earthquake. Therefore it is imperative to carry seismic evaluation of these buildings.

1.3 METHODOLOGY

a. Review of various documents on seismic evaluation of existing buildings from different countries is carried out to identify the most essential components of such a procedure for use in Pakistan, which is not only robust, reliable but also easy to use with available resources. ASCE 31-03 guidelines are selected and these guidelines are applied for evaluation of a ten storey building existing building in order to understand the procedure in depth.

b. Based on the levels of performance and seismicity, the appropriate structural, geologic and nonstructural checklists are selected. Then proceeds to fill them out using information from existing documentation, site visits and global strength checks and complete the Tier 1 Screening Phase for the case study building.

c. Carry Tier 2, linear analysis to identify any weak links in the building and perform component level checks. Performance based methodology of Pseudo lateral force is used to evaluate the building at the expected displacement of structure during the demand earthquake.

d. Perform Time History analysis for the Case Study 1 at Tier level of analysis.

e. Carry out finite element non linear static pushover analysis of a typical 7 storey reinforced concrete building. Propose a retrofit for the deficiencies identified in the building.

1.4 OBJECTIVES

The objectives of this research work can be summarized as:-

a. Comparative review of documents on seismic evaluation from various organizations.

b. Study will introduce design professional to efficient and useful procedure of ASCE 31-03 Standard for Seismic Evaluation of Existing Buildings.

c. Motivate the building authorities to add Provisions for Vulnerability assessment of existing structures in Building Code of Pakistan.

d. Use of Pushover analysis to identify deficiencies and application of retrofit technique.

CHAPTER 2

REVIEW OF DOCUMENTS ON SEISMIC EVALUATION OF EXISTING BUILDINGS

2.1 GERNAL

Structure engineers, building authorities and general public have realized that the evaluation of seismic vulnerability of the built environment is a matter of high priority. The catastrophic effects observed in recent earthquakes accelerated the awareness. The demand from the public is more rapid than the capability of the technical community to adequately manage it. This is because a lot of work has to be done towards the improvement of the codes for the design of the new structures and deal with the inherent difficulty of improvement in existing structures with procedures at the same time rigorous, general, and practically applicable**.** The lack of experimental data for the behavior and the capacity of non-seismically detailed members increases the difficulty in the evaluation of existing buildings. In view of this various organizations in the earthquake threatened countries have come up with documents, which serve as guidelines for the assessment of the expected performance and safety of existing buildings as well as for carrying out the necessary rehabilitation, if required.

2.2 REVIEW OF SEISMIC EVALUATION DOCUMENTS

Documents from USA, New Zealand, Japan and Europe have been studied and a brief summary and comparison of configuration and strength checks provided by these guidelines is presented.

2.2.1 ASCE 31-03 Seismic Evaluation of Building of Existing Buildings

ASCE 31-03 is the most advanced seismic evaluation procedure for buildings developed in USA in the recent years which improved from earlier document NEHRP Handbook for Seismic Evaluation of Existing Buildings (FEMA 178 and than FEMA 310). The evaluation procedure is based on systematic approach to determine existing structural conditions. Buildings are evaluated for certain level of structural damage (performance level) that is expected in the building when subjected to earthquake. ASCE 31-03 considers two levels of performance defined as Life Safety and Immediate Occupancy during design earthquake. ASCE 31-03 is structured in three tiers of increasing analytical detail and decreasing conservativism towards safety.

During the screening phase the design professional gets familiar with the building, its potential deficiencies and its expected behavior, so that one can quickly decide whether the building complies with this Standard. Use of Tier 1 is limited to configuration and global strength checks. Tier 1 screening provides evaluation checklists for structural, non-structural and foundation aspects for the selected performance level and given region of seismicity. After the completion of checklists, deficiencies that are found to be non-compliant are compiled and further evaluation requirements are determined.

In Tier 2 analysis and evaluation for the adequacy of the lateral-force-resisting system is performed and component level checks are performed. This analysis is limited to simplified linear analysis methods (static or dynamic).However, for unreinforced masonry bearing wall buildings with flexible diaphragms a Special Procedure is used.

A Tier 3 evaluation is performed only if one finds that Tier 1 and/or Tier 2 evaluations are too conservative. At Tier 3 phase nonlinear analysis (Pushover & Time History analysis) are performed. Expected performance of existing components can be evaluated by comparing calculated demands on the components with their capacities.

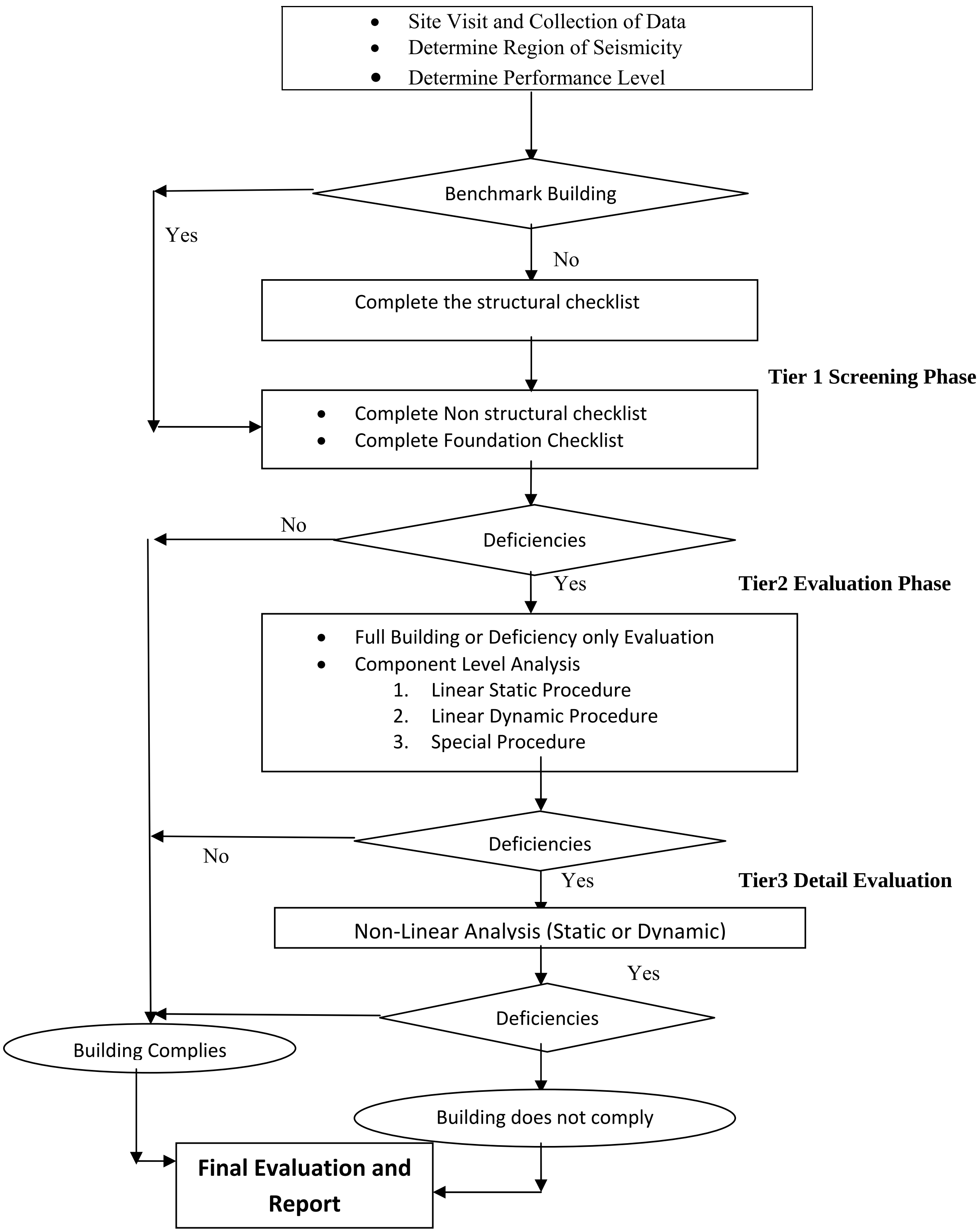

Fig 2.1 ASCE 31-03 Evaluation Procedure

2.2.2 New Zealand Draft Code (NZDC) – The Assessment and Improvement of the Structural Performance of Earthquake Risk Buildings

A draft of the document was circulated for comment in 1996 and a general draft was completed in 2002. It provides two levels of assessment An Initial Evaluation and a Detailed Assessment. This guideline only provides provisions for Life Safety performance level. In NZDC the ultimate limit state (ULS) decides acceptable and the unacceptable performance. The ULS occurs when the building or any of its components: (a) does not remain stable, (b) exceeds allowable displacement limits or (c) is strained to the accepted materials limits.

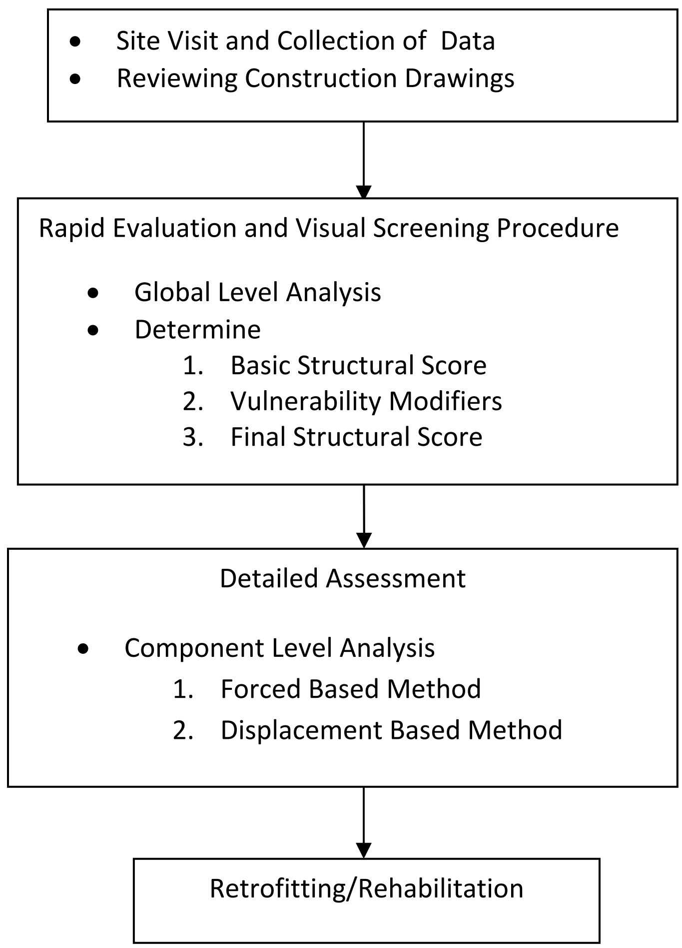

Fig 2.2 New Zealand Draft Code Evaluation Procedure

NZDC Initial Evaluation procedure is carried out from external viewing of the building and the results are presented in terms of a structural score which indicates the potential of building damage. The basic structural score which depends upon the standard used for design and earthquake damage potential of the respective building types in their location of high, moderate or low seismicity zones is modified to account for unfavorable characteristics present in the building. The structural score is further modified to account for building area.

The detailed structural evaluation is performed at the component level. A knowledge factor (K) is introduced to account for the uncertainty in material strength and condition. One of the three analysis procedures (Time history analysis, force analysis and displacement analysis) is used to evaluate the building. Time History analysis is most accurate but also most complicated. Force-based assessment determines the probable strength and ductility of the critical mechanism of post-elastic deformation. The displacement-based uses the ultimate displacement capacity of lateral force resisting elements.

2.2.3 Japanese guidelines

The standard was published in 1977 and was revised in December 1990. The standard is applicable to evaluate the seismic performance of an existing RC building whose structural system is made of moment resisting frames, with or without shear walls. Its application is limited to low-rise buildings, since the standard assumes constant acceleration response (flat response acceleration spectrum with periods) [Otani, 2000].

It provides three procedures of different accuracy and reliability. At first-level, screening procedure the buildings are classified based on their storey shear strengths, provided by either columns or / and structural walls. The buildings found to be deficient by the first-level procedure, is analyzed by the *second* level procedure, in which the deformation capacities of vertical members are also considered. The third-level procedure also includes the weak beam-strong column mechanism. The Japanese assessment compares the floor shear demand due to the earthquake action versus the floor shear capacity, for every storey and every frame of the building.

For the first-level procedure the members capacities can be computed from design values of materials strengths. For second and third level procedures, inspections are required to evaluate the member's strength and ductility. The effect of cracks and deterioration of concrete are macroscopically accounted in the evaluation of the index T, so these effects may be neglected in the calculation of structural performance of the elements. The torsional effects are considered empirically through the structural configuration factor S.

2.2.5 Euro Code 8: Design Provisions For Earthquake Resistance of Structures – Parts 1-4 General Rules for Strengthening and Repair of Buildings

This document was approved by CEN in 1995 as a prospective standard for provisional application (CEN 1995). This document provides guidelines for the evaluation of the seismic performance of existing structures and selects a suitable retrofit in case found deficient. According to Euro code, analysis and redesign of existing structures is based on modified actions and modified safety-factors. An uncertainty factor may be introduced; higher values should be used for higher damage levels.

For the calculation of actual earthquake demand, the dynamic non-linear (Time History) analysis is carried out. Eurocode suggests use of nonlinear static procedure for plain masonry. After the analysis, component level checks are performed, which is based on the verification of all cross sections. For time History analysis, the post yield deformations should be higher than the corresponding demand values. Most of the checks in Eurocode are qualitative, so it requires engineering judgment of structural experts. At the end, it gives procedure for retrofit of the deficient buildings.

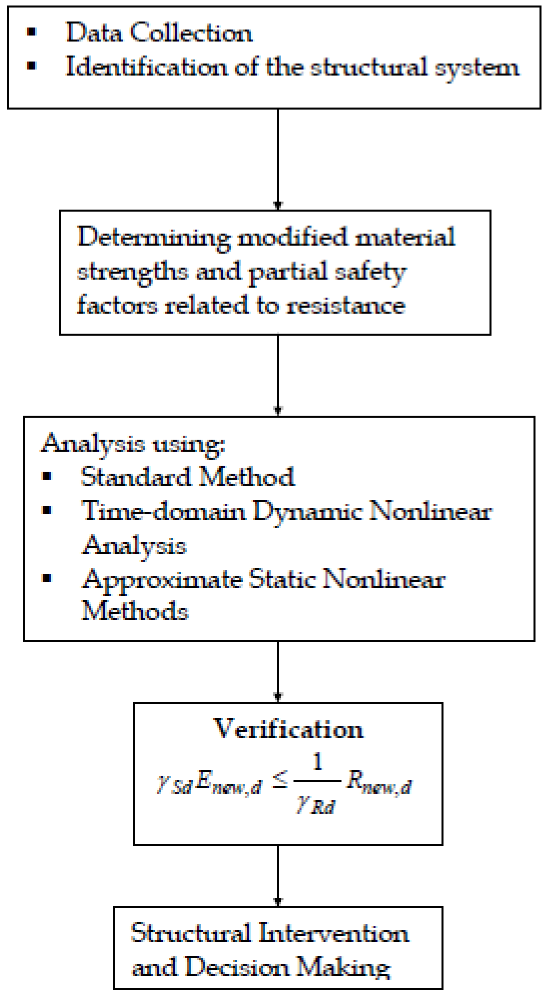

Fig 2.4 Euro Code 8 Evaluation Procedure

2.3 General Structure of Seismic Evaluation Procedure

Seismic Evaluation procedures can be broadly classified as: (a) configuration-related and (b) strength related checks. These checks are explicitly or implicitly arranged in the two tiers of assessments. A general layout of seismic evaluation procedures is shown in Fig. 2.5. The first tier is a quick assessment of the earthquake resistance of the building and its potential deficiencies. The first tier is conducted to screen out the significantly vulnerable structures for the second tier detailed analysis and evaluation. Only configuration checks and global strength checks are considered for screening phase. For Tier 2 proper force and displacement analysis (both global and/or component level) is conducted.

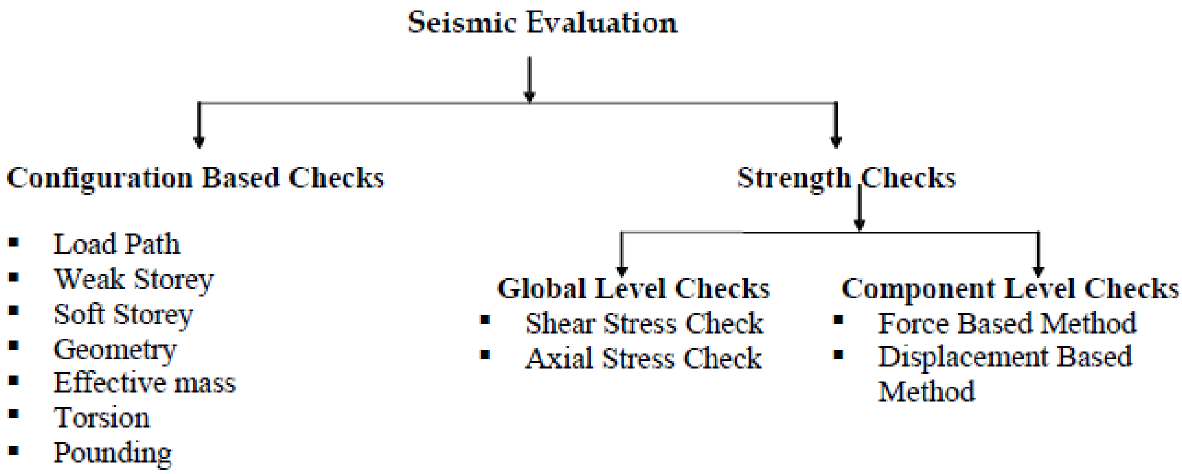

Figure2.5 General Structure of Evaluation Procedure

2.3.1 Configuration Based checks

2.3.1.1 Load Path

In a general load path, Inertial loads that develop due to accelerations of individual elements are transferred from the individual reactive elements to the diaphragms (floors and roofs), the diaphragms distribute these forces to vertical lateral-force-resisting elements (shear walls and frames); the vertical elements transfer these forces to the foundation; and the foundation transfers the forces to the underlying soil. If there is a discontinuity in the load path, the building will be unable to resist seismic forces regardless of the strength of existing elements.

ASCE 31-03 specifies that, the structure shall contain a minimum of one complete load path for Life Safety and Immediate Occupancy for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (ASCE 31-03). As per New Zealand Draft Code, the existing load paths should be identified and the effects of any past modifications, additions or alterations should be considered. Euro code 8 specifies that all lateral load resisting systems (cores, structural walls or frames) should run without interruption from their foundations to the top of the building.

2.3.1.2 Adjacent Buildings

There shall be adequate separation between buildings to prevent hammering if the floor slabs at each level do not align vertically within 50% of the height of the thicker floor slab. Historically buildings have been designed as if the adjacent buildings do not exist and buildings are often built right up to property lines in order to make maximum use of space. As a result, the buildings may impact each other, or pound, during an earthquake. Pounding may alter the dynamic response of both buildings and impart additional inertial loads on both structures.

According to ASCE 31-03, in order to avoid pounding, the building shall not be located closer than 4% of the height to an adjacent building and as per New Zealand Code separation limit is of 2% the storey height. There is no provision for pounding in Eurocode 8.

2.3.1.3 WEAK STORY

The story strength is the total strength of all the lateral-force resisting system in that particular story for the direction under consideration. It is the shear capacity of columns and/or shear walls, or the horizontal component of the capacity of diagonal braces. Generally an examination of the building elevations can determine if a weak story exists. A reduction in the number or length of lateral force- resisting elements or change in the type of lateral force- resisting system is obvious indications of a weak story. Weak story results in concentration of inelastic activity that may result in the partial or total collapse of the story.

According to ASCE 31-03 and NZDC, the strength of lateral force resisting system in any story shall not be less than 80% of the strength in an adjacent story, above or below. As per Eurocode 8, the mass of the individual stories should remain constant or reduce gradually, without abrupt changes, from the base to the top.

2.3.1.4 SOFT STORY

Soft story condition commonly occurs in buildings with open fronts at ground floor or with particularly tall first stories. Soft stories usually are revealed by an abrupt change in story drift, as shown by the damage to building. Although a comparison of the stiffnesses in adjacent stories is the direct approach, a simple first step might be to compare the interstory drifts and concentration of drift in a particular story may leads to collapse. Generally an examination of the building elevations can determine if a soil story exists. The difference between "soft" and "weak" stories is the difference between stiffness and strength.

According to ASCE 31-03 and NZDC give a quantitative check, the stiffness of lateral force resisting system in any story shall not be less than 70% of the stiffness in an adjacent story above or below, or less than 80% of the average stiffness of the three stories above or below. Eurocode 8 specifies that there should not be significant difference in the lateral stiffness of individual storey and at any storey the maximum displacement in the direction of the seismic forces should not exceed the average storey displacement by more than 20.

2.3.1.5 Geometry

Geometric irregularities are usually detected in an examination of the story-to-story variation in the dimensions of the lateral-force-resisting system. The irregularity of concern is in the dimensions of the lateral force- resisting system and not in the envelope of the building, so it may not be obvious. Geometric irregularities affect the dynamic response of the structure and may lead to unexpected higher mode effects and concentrations of demand.

ASCE 31-03 specifies that there shall be no change in the horizontal dimension of lateral force resisting system of more than 30% in a storey relative to adjacent stories. As per New Zealand Draft Code plan irregularities include irregular mass distribution, re-entrant corners and buildings with 'wings' that form an 'L', 'T' or 'E' shape. As per Eurocode 8, the building structure should be approximately symmetrical in plan with respect to two orthogonal directions.

2.3.1.6 Mass Irregularity

Mass irregularities can be detected by comparison of the story weights. The effective mass consists of the dead load of the structure tributary to each level, plus partition and permanent equipment weights at each floor. Buildings are typically designed for primary mode effects. The validity of this approximation is dependent on the vertical distribution of mass and stiffness in the building. Mass irregularities affect the dynamic response of the structure and may lead to unexpected higher mode effects and concentrations of demand.

ASCE 31-03 specifies that there shall be no change in effective mass more than 50 percent from one story to the next for Life Safety and Immediate Occupancy. As per NZDC, a significant vertical irregularity results when the mass of a storey varies 30% from those adjacent. Eurocode 8 specifies that the individual storey mass should remain constant or reduce gradually without abrupt changes.

2.3.1.7 Torsion

Torsion results in additional seismic demands imposed on the vertical elements by rotation of the diaphragm. Buildings can be designed to meet code forces including torsion, but buildings with severe torsion are less likely to perform well in earthquakes. The columns are forced to drift laterally with the diaphragm, including lateral forces and P-delta effects for which they may not have been designed. The strength of the vertical elements of the lateral-force-resisting system may not be adequate because of additional seismic demands due to torsion.

As per ASCE 31-03, distance between storey mass centre and story centre of rigidity shall be less than 20% of the building width in either plan dimension. NZDC considers the torsional deformation in evaluating the required ductility demand for critical elements. As per Eurocode 8, frame and wall systems should possess a minimum torsional rigidity.

2.3.2 Strength Checks

In addition to configuration checks, a number of checks are required to assess the load carrying capacity of the lateral Force-resisting elements, such as columns, walls, etc. The evaluation procedures provide from a simple to a much rigorous method of calculation of seismic demand at both global level and local level. Various aspects of these strength related checks are compared in the following paragraphs.

2.3.2.1 Force Levels for Strength Analysis

In ASCE 31-03, a pseudo static lateral force is applied to the structure to obtain "actual" displacements during a design earthquake. It represents the force required in a linear static analysis, to impose the expected actual deformation of the structure in its yielded state when subjected to the design earthquake motions. For ASCE 31-03, the analysis forces for evaluation are only 75% of that for design of new buildings. NZDC considers the design force level based on inelastic behavior and seismic force capacity of the structure is calculated along with the post elastic mechanism of deformation. For NZDC, the force level for evaluation of existing building is taken as 67% of that for a new building. Eurocode 8 calculates the design base shear considering a behavior factor *q* which accounts for the ductility class, the structural regularity in elevation and the prevailing failure mode in structural systems with walls. EuroCode suggest that force level for evaluation should be reduced from that of a new building but does not quantify. NZDC and Eurocode take in to account the response reduction factor (inelastic behavior) in calculating the lateral forces, whereas ASCE 31-03 considers the maximum force level with no response reduction factor (elastic Behavior). However, ASCE 31-03 allows for inelastic behavior at the component level analysis by assigning *m*-factors for the displacement-controlled ductile components.

2.3.2.2 Global Level Checks

The seismic evaluation documents specify some global level checks to quickly identify the major deficiencies. At the global level, buildings are mainly checked for shear stress and axial stress.

(a) Shear Stress Check

The shear stress check provides a quick assessment of the overall level of demand on the structure. According to ASCE 31-03the shear stress in concrete columns shall be less than 100 psi, while the shear stress in the unreinforced masonry shear walls shall be less than 30 psi. As per New Zealand draft code, a check for shear is done at the component level.

(b) Axial Stress Check

Columns that carry a substantial amount of gravity load may have limited additional capacity to resist seismic forces. When axial forces due to seismic overturning moments are added, the columns may fail in a non-ductile manner due to excessive axial compression. The alternative calculation of overturning stresses due to seismic forces alone is intended to provide a means of screening out frames with high gravity loads, but is known to have small seismic overturning forces. According to ASCE 31-03 the axial stresses due to overturning forces alone shall be less than 0.30f'c. New Zealand Draft Code provides axial stress check at component level in the displacement-based analysis.

2.3.2.3 Component Level Analysis

Component level analysis is carried to have detail assessment of the building. It helps in identifying the weak links of the building. Actions are classified as either deformation-controlled or force-controlled. A deformation-controlled action is defined as an action for which deformation is allowed to exceed the yield value. The maximum associated deformation is limited by the ductility capacity of the component. In force controlled deformation is not allowed to exceed the yield value. The actions with limited ductility are classified as force-controlled. For ASCE 31-03 acceptance criteria is based on these actions. Expected strength of is compared with the demand due to gravity and earthquake loading. Force-controlled actions are not reduced. While in deformation-controlled actions, an m-factor (depending upon the ductility of the component) is applied to the demand.

The force-based method in NZDC is based on determining the probable strength and ductility of the critical mechanism of post-elastic deformation of the lateral force-resisting elements. Displacement-based methods place a direct emphasis on establishing the ultimate displacement capacity of lateral force resisting elements. Displacement-based assessment utilizes displacement spectra, which readily represent the characteristics of real earthquakes.

The final evaluation in Eurocode 8 is based on the verification of all cross-sections comparing the capacity design as required with the design resistance values of cross-sections of the structural elements.

2.4 Comparison of guidelines

Seismic evaluation procedures for buildings are a combination of configuration-related and strength-related checks. Though there have been no significant differences in which the configuration related assessments are carried out, there is considerable degree of non-uniformity in the manner strength–related assessments are carried out. Strength checks are performed either at global or local level or at both levels.

The seismic evaluation procedure of ASCE 31-03 and New Zealand Draft, provides detailed and specific assessment techniques. Eurocode 8 describes mostly the principles of evaluation and is seriously deficient of specifics which make it difficult to use. Further, there are many parameters for which no guidance is provided and is left to the judgment of the design professional. Except for ASCE 31-03, all evaluation procedures require a building to be classified into one of the specified building category for evaluation. This becomes difficult to implement wherein the structural systems for building are vague and of mixed nature. ASCE 31-03 is preferred choice for structural systems that cannot be clearly categorized as either frames or shear walls.

All documents specify that there should be some reduction in the force level for analysis of existing building compared to new buildings. In ASCE 31-03, a reduction factor of 0.75 is applied to seismic forces in the Tier 3 evaluation; however, this reduction factor is implicitly present in the form of *m*-factors at Tier 2 analysis. New Zealand Draft Code suggests a reduction factor of 0.67. Eurocode 8 also mentions that considering the smaller remaining lifetimes, the effective peak ground acceleration should be reduced for redesign purposes; however it does not quantify the reduction factor.

ASCE 31-03 provides a more generalized approach to seismic evaluation, which is thorough and provides several levels of assessment with varying degree of complexity suitable for a large class of structures. However, it requires a higher degree of understanding on the part of design professionals as it is uses a different approach than that of traditional design codes. NZDC is transparent and uses familiar basic principles as applicable to design of new buildings, though its approach is considerably non-generalized (applicable to specific building types given in the Code). Eurocode 8 lack specific steps of assessment and leave a lot to the judgment of the design professional. It appears that ASCE 31-03 and NZDC approaches can be suitably combined to develop a transparent, reasonably rigorous and generalized procedure for seismic evaluation of buildings in developing countries such as Pakistan. However considering the generalized and very specific approach provided by ASCE 31-03, it is the most suitable for use in Pakistan.

CHAPTER 3

SCREENING PHASE (TIER 1)

3.1 EVALUATION REQUIREMENTS

All available documents and drawings are reviewed before Tier 1 analysis. The information collected should be sufficient to define the level of performance desired, the region of seismicity and the building type and to complete the Tier 1 Checklists. The two performance levels for both structural and nonstructural components are Life Safety (LS) and Immediate Occupancy (IO). The region of seismicity of the building shall be defined as low, moderate, or high in accordance with Table 3.1

Table 3.1 Levels of Seismicity Definitions

Level Of Seismicity	S_{DS}	S_{D1}
Low	<0.167g	<0.067g
Moderate	>0.167g <0.500g	>0.067g <0.200g
High	>0.500g	>0.200g

S_{DS} = Design short-period spectral response acceleration parameter

S_{D1} = Design spectral response acceleration at a one-second period

3.2 BENCHMARK BUILDINGS

Initially, the design professional determines whether the building meets the benchmark building criteria of this handbook. A structural seismic evaluation using this Handbook is not required for buildings designed and constructed in accordance with the benchmark documents listed in Table 3-2; an evaluation for foundations and nonstructural elements remains applicable. If the seismicity of a region has changed since the benchmark dates than structural checklist is compeleted.

Table 3.2 Benchmark Buildings

Building Type [1]	Model Building Seismic Design Provisions				FEMA 178[1s]	CBC[io]
	BOCA[1s]	SBCC[1s]	UBC[1s]	NEHRP[1s]		
Wood Frame, Wood Shear Panels (Type W1 & W2)[2]	1992	1993	1976	1985	*	1973
Wood Frame, Wood Shear Panels (Type W1A)	1992	1993	1976	1985	*	1973
Steel Moment Resisting Frame (Type S1 & S1A)	**	**	1994[4]	**	*	1995
Steel Braced Frame (Type S2 & S2A)	1992	1993	1988	1991	1992	1973
Light Metal Frame (Type S3)	*	*	*	*	1992	1973
Steel Frame w/ Concrete Shear Walls (Type S4)	1992	1993	1976	1985	1992	1973
Reinforced Concrete Moment Resisting Frame (Type C1)[3]	1992	1993	1976	1985	*	1973
Reinforced Concrete Shear Walls (Type C2 & C2A)	1992	1993	1976	1985	*	1973
Steel Frame with URM Infill (Type S5, S5A)	*	*	*	*	*	*
Concrete Frame with URM Infill (Type C3 & C3A)	*	*	*	*	*	*
Tilt-up Concrete (Type PC1 & PC1A)	*	*	1997	*	*	*
Precast Concrete Frame (Type PC2 & PC2A)	*	*	*	*	1992	1973
Reinforced Masonry (Type RM1)	*	*	1997	*	*	*
Reinforced Masonry (Type RM2)	1992	1993	1976	1985	*	*
Unreinforced Masonry (Type URM)[5]	*	*	1991[6]	*	1992	*
Unreinforced Masonry (Type URMA)	*	*	*	*	*	*

3.3 SELECTION AND USE OF CHECKLISTS

Table 3-3 provides checklists, as a function of region of seismicity and level of performance. Each of the evaluation statements on the checklists is marked "compliant" (C), "noncompliant" (NC), or "not applicable" (N/A). Compliant statements identify issues that are acceptable according to the criteria of this Handbook, while non-compliant statements identify issues that require further investigation. Different checklists are provided for the each building types defined in this handbook. The General Structural Checklists is used for buildings that cannot be classified as one of the Common Building Types. For a building with a different lateral-force-resisting system in each principal direction two sets of structural checklists are used (one for each direction).

TABLE 3.3 Checklists Required for Tier 1 Analysis

Region of Seismicity	Level of Performance[2]	Required Checklists[1]					
		Region of Low Seismicity (Sec. 3.6)	Basic Structural (Sec. 3.7)	Supplemental Structural (Sec. 3.7)	Geologic Site Hazard and Foundation (Sec. 3.8)	Basic Nonstructural (Sec. 3.9.1)	Supplemental Nonstructural (Sec. 3.9.2)
Low	LS	√					
	IO		√		√	√	
Moderate	LS		√		√	√	
	IO		√	√	√	√	√
High	LS		√	√	√	√	
	IO		√	√	√	√	√

3.4 SEISMIC SHEAR FORCES

3.4.1Pseudo Lateral Force

The pseudo lateral force, in a given horizontal direction of a building, is calculated in accordance with Equations (3-1) or (3-2).

$$V = C \times Sa \times W \qquad (3\text{-}1)$$

Where:

V = Pseudo lateral force;

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

Sa = Response spectral acceleration at the fundamental period of the building in the direction under consideration.

W = Total dead load and anticipated live load as follows: In storage and warehouse occupancies, a minimum of 25% of the floor live load;

For buildings with shallow foundations and without basements being evaluated for the Life Safety Performance Level, Equation (3-2) may be used to compute the pseudo lateral force.

$$V = 0.75W \qquad (3\text{-}2)$$

Code Based procedures of seismic design of new buildings (UBC, IBC and etc.) account for nonlinear seismic response in a linear static analysis procedure by including a response modification factor, R. These procedures reduce equivalent base shear to produce a rough approximation of the internal forces during a design earthquake. In other words, the base shear is equivalent to what the building is expected to resist strength-wise, but the building displacement using this base shear are significantly less than the displacements the building will actually experience during a design earthquake.

The linear static analysis procedure in this Handbook, as well as in FEMA 356, takes a different approach to account for the nonlinear seismic response. Pseudo static lateral forces are applied to the structure to obtain "actual" displacements during a design earthquake. The pseudo lateral force of Equation (3-1) represents the force required, in a linear static analysis, to impose the expected actual deformation of the structure in its yielded state when subjected to the design earthquake motions In short, this procedure is based on equivalent displacement and Pseudo lateral forces.

3.4.2 Story Shear Forces

The pseudo lateral force computed in accordance with procedure described above are distributed vertically in accordance with Equation (3-3).

$$F_x = (w_x h_x^k / \sum w_i h_i^k) \text{ x } V \qquad (3\text{-}3a)$$

$$V_j = \sum Fx \qquad (3\text{-}3b)$$

where:

Vj = Story shear at story level j,

n = Total number of stories above ground level,

j = Number of story level under consideration,

W = Total seismic weight as defined above.

V = Pseudo lateral force from Equation (3-1) or (3-2).

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

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

h_i = Height (ft) from base to floor level i.

h_x = Height (ft) from base to floor level x.

k = 1 for T= 0.5 seconds

= 2 for T > 2.5 seconds

For the building with flexible diaphragms, story shear shall be calculated separately for each line of lateral resistance.

3.4.3 Spectral Acceleration

Spectral acceleration is based on mapped spectral accelerations, defined below for the site of the building being evaluated. Alternatively, a site specific response spectrum may be developed.

3.4.3.1 Mapped Spectral Acceleration

The mapped spectral acceleration, Sa, is computed in accordance with Equation (3-4).

$$Sa = S_{D1}/T \qquad (3\text{-}4)$$

Sa shall not exceed S_{DS};

where:

$$SD_1 = 2/3\ FvS_1 \qquad (3\text{-}5)$$

$$SD_S = 2/3\ FaS_s \qquad (3\text{-}6)$$

- S_s and S_1 are short period response acceleration and spectral response acceleration at a one second period, for the Maximum Considered Earthquake (MCE).
- T = Fundamental period of vibration of the building in seconds.
- F_v and F_a are site coefficients and shall be determined from Tables 3-5 and 3-6, respectively, based on the site class and the values of the response acceleration parameters Ss and S1.

The site class of the building shall be defined as one of the following:

Class A: Hard rock with measured shear wave velocity, v_s > 5,000 ft/sec;

Class B: Rock with 2,500 ft/sec < v_s < 5,000ft/sec.

Class C: Very dense soil and soft rock with1, 200 ft/sec < v_s< 2,500 ft/sec or with either standard blow count N > 50 or un-drained shear strength s_u > 2,000 psf.

Class D: Stiff soil, 600 ft/sec < v_s <1,200 ft/sec or with 15 < N < 50

Class E: Any profile with more than 10 feet of soft clay defined as soil with plasticity index PI>20, or water content w > 40 percent, v_s < 600ft/sec.

Class F: Soils requiring a site-specific geotechnical investigation and dynamic site response analyses:

- Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, highly-sensitive clays, collapsible weakly-cemented soils;
- Peats and/or highly organic clays (H>1 feet of peat and/or highly organic clay;
- Very high plasticity clays (H > 25 feet with PI > 75 percent);
- Very thick soft/medium stiff clays (H >120 feet).

Table 3.4 Values of F_v

Site Class	Mapped Spectral Acceleration at one-second Period				
	$S_1 < 0.1$	**$S_1 = 0.2$**	**$S_1 = 0.3$**	**$S_1 = 0.4$**	**$S_1 < 0.5$**
A	0.8	0.8	0.8	0.8	0.8
B	1	1	1	1	1
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	*	*	*	*	*

*Site-specific geotechnical investigation and dynamic response analysis required.

Table 3.5Values of F_a

Site Class	Mapped Spectral Acceleration at one-second Period				
	$S_1 <0.1$	**$S_1 = 0.2$**	**$S_1 = 0.3$**	**$S_1 = 0.4$**	**$S_1 < 0.5$**
A	0.8	0.8	0.8	0.8	0.8
B	1	1	1	1	1
C	1.2	1.2	1.1	1	1
D	1.6	1.4	1.2	1.1	1
E	2.5	1.7	1.2	0.9	0.9
F	*	*	*	*	*

*Site-specific geotechnical investigation and dynamic response analysis required.

3.4.3.2 Site-Specific Spectral Acceleration

The site-specific response spectra are based on the geologic, seismological, and soil characteristics associated with the specific site of the building being evaluated. Site-specific response spectra are based on input ground motions with a 2% probability of exceedance in 50 years (2500 year return interval) and developed for an equivalent viscous damping ratio of 5%.

3.4.4 Period

The fundamental period of a building, in the direction under consideration, is calculated in accordance with Equation (3-7).

$$T = C_t h_n^{3/4} \qquad (3\text{-}7)$$

Where:

h_n = height (in feet) above the base to the roof level

C_t = 0.030 for moment-resisting frames of reinforced concrete (Building Type C1)

Alternatively, for steel or reinforced-concrete moment frames of 12 stories or less the fundamental period of the building may be calculated as follows:

$$T=0.10N \qquad (3\text{-}8)$$

Where:

N = number of stories above the base.

3.5 QUICK CHECKS FOR STRENGTH AND STIFFNESS

Quick Checks are used to compute the stiffness and strength of building components. Quick Checks are triggered by evaluation statements in the Checklists and are required to determine the compliance of certain building components.

3.5.1 Story Drift for Moment Frames

Equation (3-9) is used to calculate the drift ratios of regular, multistory, multi-bay moment frames with columns continuous above and below the story under consideration. The drift ratio is based on the deflection due to flexural displacement of a representative column, including the effect of end rotation due to bending of the representative girder.

$$DR = (K_b+K_c)/K_bK_c\,(h/12E)\ Vc \qquad (3\text{-}9)$$

Where:

DR= Drift Ratio = Inter-story displacement divided by story height,

K_b = I/L for the representative beam,

K_c = I/h for the representative column,

h = Story height (in.),

I = Moment of inertia (in4),

L = Center to center length of columns (in.),

E = Modulus of elasticity (ksi),

V_c = Shear in the column (kips).

The column shear forces are taken as a portion of the story shear forces, computed in accordance with procedure defined above. For reinforced concrete frames, an equivalent cracked section moment of inertia equal to one half of gross value shall be used.

Equation (3-9) may also be used for the first floor of the frame if columns are fixed against rotation at the bottom. However, if columns are pinned at the bottom, the drift ratio is multiplied by two.

3.5.2 Shear Stress in Concrete Frame Columns

The average shear stress, V_{avg}, in the columns of concrete frames is computed in accordance with Equation (3-10) and should be less than 100 psi.

$$V_{avg} = 1/m\,[n_c / (n_c\text{-}n_f)]\,(V_j/A_c) \qquad (3\text{-}10)$$

Where:

n_c = Total number of columns;

n_f = Total number of frames in the direction of loading;

Ac = Summation of the cross sectional area of all columns in the story under consideration;

Vj = Story shear computed in accordance with Section 2.2.2.3.

m = component modification factor; m shall be taken equal to 2.0 for buildings being evaluated to the Life Safety Performance Level and 1.3 for buildings being evaluated to the Immediate Occupancy Performance Level.

Equation (3-10) assumes that all of the columns in the frame have similar stiffness.

3.5.3 Shear Stress in Shear Walls

The average shear stress in shear walls, V_{avg}, is calculated using Equation (3-11) and should be less than 100 psi.

$$V_{avg} = 1/m\ (Vj/Aw) \qquad (3\text{-}11)$$

Where:

Vj = Story shear at level j.

Aw = Summation of the horizontal cross sectional area of all shear walls in the direction of loading. Openings shall be taken into consideration when computing Aw. For masonry walls, the net area shall be used.

m = component modification facto

3.5.4 Axial Stress Due to Overturning

The axial stress of columns subjected to overturning forces, pot, is calculated in accordance with Equation (2-12) and it should not exceed 0.3 $f_{c.}$

$$P_{ot} = 1/m\ (2/3)\ (Vh_n/\ Ln_f)\ x\ (1/A_{col}) \qquad (2\text{-}12)$$

where:

n_f = Total number of frames in the direction of loading;

V = Pseudo lateral force;

h_n = height (in feet) above the base to the roof level.

L = Total length of the frame (in feet);

m = Component modification factor taken equal to 2.0 for buildings being evaluated to the Life Safety Performance Level and 1.3 for buildings being evaluated to Immediate Occupancy Performance Level.

A_{col} = Area of end column of frame

3.6 CASE STUDY 1

3.6.1 Description of Building & Building Site:

The building is a 10 storey (plus a basement) RC Frame structure with a shear wall at one end (Lift Well) constructed in 1991 as per BCP 1985. A 3D Model and planes at different levels are shown in the appendix. The building considered has a raft foundation on a site with bearing capacity of 1 tsf and site class D. The Building Site is in Seismic Zone 2B as per BPC 2007. As per seismic hazard carried by Zameer, Spectral acceleration at 1 sec, S_1 = 0.15g & Short period spectral acceleration, S_S = 0.82g. According to Table 3.1 building site is classified as of moderate seismicity level. The tower considered is surrounded by another tower of similar story heights. Partition walls are of Brick masonry & Block masonry. Building has some minor cracks. Currently the building is vacant due to seismic risk. Material strength for analysis are taken as, f_c = 3 ksi, & f_y = 60 ksi. Structural Drawing of the building was available and it is believed that construction is as per drawing. Adjacent Building is of same story heights and no. of stories. Non Structural checklists are not considered in this study.

3.6.2 BASIC STRUCTURAL GERNAL CHECKLIST (FOR COCRETE FRAME AND SHEAR WALL SYSTEM)

BUILDING SYSTEM

General

C NC N/A LOAD PATH: The structure shall contain one complete load path for Life Safety& Immediate Occupancy for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation.

C NC N/A ADJACENT BUILDINGS: An adjacent building shall not be located next to the structure being evaluated closer than 4% of the height for Life Safety and Immediate Occupancy.
Adjacent building is of same no of stories and height.

C NC N/A MEZZANINES: Interior mezzanine levels shall be braced independently from the main structure, or shall be anchored to the lateral-force-resisting elements of the main structure.

Configuration

C NC N/A WEAK STORY: The strength of the lateral-force-resisting system in any story shall not be less than 80% of the strength in an adjacent story above or below for Life-Safety and Immediate Occupancy.

Lateral force resisting system reduces to half from story 7 to 8

C NC N/A SOFT STORY: The stiffness of the lateral-force-resisting system in any story shall not be less than 70% of the stiffness in an adjacent story above or below or less than 80% of the average stiffness of the three stories above or below for Life-Safety and Immediate Occupancy.

Lateral force resisting system reduces to half from story 7 to 8

C NC N/A GEOMETRY: There shall be no changes in horizontal dimension of the lateral-force-resisting system of more than 30% in a story relative to adjacent stories for Life Safety and Immediate Occupancy, excluding one-story penthouses.

There is about 50% difference of horizontal dimension b /w story 7 &8.

C NC N/A VERTICAL DISCONTINUITIES: All vertical elements in the lateral-force-resisting system shall be continuous to the foundation.

All columns and shear walls are continuous to the foundation.

C **NC** **N/A** MASS: There shall be no change in effective mass more than 50% from one story to the next for Life Safety and Immediate Occupancy.

There is about 50% difference of mass b /w story 7 &8.

C **NC** **N/A** TORSION: The distance between the story center of mass and the story center of rigidity shall be less than 20% of the building width in either plan dimension for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.6)

Center of mass & centre of rigidity differ by more than 20%. The difference is shown in the table given in the appendix c.

Condition of Material

C **NC** **N/A** DETERIORATION OF CONCRETE: There shall be no visible deterioration of concrete or reinforcing steel in any of the vertical- or lateral-force-resisting elements.

On the site visit, visible cracks were observed in the building.

C **NC** **N/A** POST-TENSIONING ANCHORS: There shall be no evidence of corrosion or spelling in the vicinity of post-tensioning or end fittings. Coil anchors shall not have been used.

LATERAL FORCE RESISTING SYSTEM

General

C NC N/A REDUNDANCY: The number of lines of moment frames in each principal direction shall be greater than or equal to 2 for Life Safety and Immediate Occupancy. The number of bays of moment frames in each line shall be greater than or equal to 2 for Life Safety and 3 for Immediate Occupancy.

Moment Frames With Infill walls

C NC N/A INTERFERING WALLS: All infill walls placed in moment frames shall be isolated from structural elements.

Concrete Moment Frames

C NC N/A SHEAR STRESS CHECK: The shear stress in the concrete columns, calculated using the Quick Check procedure of Section 3.5.3.2, shall be less than 100 psi or 2 for Life Safety and Immediate Occupancy.

Calculation of Average stress is shown in appendix B; the average shear stress is exceeding the limit in all the stories.

C NC N/A AXIAL STRESS CHECK: The axial stress due to gravity loads in columns subjected to overturning forces shall be less than $0.10f'c$ for Life Safety and Immediate Occupancy. Alternatively, the axial stresses due to overturning forces alone, calculated using the Quick Check Procedure of Section 3.5.3.6, shall be less than 0.30$f'c$ for Life Safety and Immediate Occupancy.

In appendix axial stress due to overturning force verified to be in the limit.

Concrete Shear Walls

C NC N/A SHEAR STRESS CHECK : The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 3.5.3.3, shall be less than 100 psi for Life Safety and Immediate Occupancy.

Shear stress is greater than 100 psi.

C NC N/A REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area shall be greater than 0.0015 in the vertical direction and 0.0025 in the horizontal direction for Life Safety and Immediate Occupancy. The spacing of reinforcing steel shall be equal to or less than 18" for Life Safety and Immediate Occupancy.

Structure drawing shows that these limits are fulfilled.

CONNECTIONS

C NC N/A CONCRETE COLUMNS: All concrete columns shall be doweled into the foundation for Life Safety and the dowels shall be able to develop the tensile capacity of the column for Immediate Occupancy.

C NC N/A TRANSFER TO SHEAR WALLS: Diaphragms shall be reinforced and connected for transfer of loads to the shear walls for Life Safety and the connections shall be able to develop the shear strength of the walls for Immediate Occupancy.

C NC N/A WALL REINFORCING: Walls shall be doweled into the foundation for Life Safety and the dowels shall be able to develop the strength of the walls for Immediate Occupancy.

C NC N/A SHEAR-WALL-BOUNDARY COLUMNS: The shear wall boundary columns shall be anchored to the building foundation for Life Safety and the anchorage shall be able to develop the tensile capacity of the column for Immediate Occupancy.

Geologic Site Hazards

C NC N/A LIQUEFACTION: Liquefaction susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.7.1.1)

C NC N/A SLOPE FAILURE: The building site shall be sufficiently remote from potential earthquake-induced slope failures or rock falls to be unaffected by such failures or shall be capable of accommodating any predicted movements without failure. (Tier 2: Sec. 4.7.1.2)

C NC N/A SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Tier 2: Sec. 4.7.1.3)

Condition of Foundations

C **NC** **N/A** FOUNDATION PERFORMANCE: There shall be no evidence of excessive foundation movement such as settlement or heave that would affect the integrity or strength of the structure. (Tier 2: Sec. 4.7.2.1)

Capacity of Foundations

C **NC** **N/A** POLE FOUNDATIONS: Pole foundations shall have a minimum embedment depth of 4 ft. for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.7.3.1)

3.6.3 Conclusion of Tier 1 Analysis of the case study

Configuration of the building is irregular so detailed analysis of the building should carried out considering 3D model of the building as per Tier 2 provisions. Shear stress in columns and shear wall exceeds the limit in the global strength checks. Shear stresses at component level should be verified.

Chapter 4

Evaluation Phase (Tier 2)

4.1 General

A Full-Building Tier 2 analysis and evaluation of the adequacy of the lateral-force-resisting system is performed for all buildings designated as "T2" in the table given in this handbook. For all other buildings, the design professional may choose to perform a deficiency-only Tier 2 evaluation. A Tier 2 Evaluation includes an analysis using one of the linear method: Linear Static Procedure, Linear Dynamic Procedure, or Special Procedure. If deficiencies are identified in a Tier 2 Evaluation, the design professional may perform a Tier 3 Evaluation. Alternatively, the design professional may choose to end the investigation and report the deficiencies. This standard uses a displacement based lateral force procedure (Pseudo Lateral Force) and m-factor on an element-by-element basis. It represents the most direct method for considering nonconforming systems.

4.2 Tier 2 Analysis Procedures

Four analysis procedures are provided in this section:

- Linear Static Procedure (LSP),
- Linear Dynamic Procedure (LDP),
- Special Procedure
- Procedure for Nonstructural Components.

All building structures are evaluated by either the Linear Static Procedure (LSP) or the Linear Dynamic Procedure (LDP) discussed in coming paragraphs. The acceptability criteria for both the LSP and LDP are provided below. If original design calculations are available, the results may be used; an appropriate scaling factor, (to relate the original design base shear to the Pseudo lateral force of this Handbook) is applied. Unreinforced masonry (URM) bearing wall buildings with flexible diaphragms are evaluated in accordance with the requirements of the Special Procedure.

4.2. 1Analysis Procedures for LSP & LDP

The Linear Static or Linear Dynamic Procedure is performed as required by the Procedures given in ASCE 31-03 handbook. The Linear static procedure is applicable for all buildings unless a Linear Dynamic Procedure or Special Procedure is required.

The Linear Dynamic Procedure shall be used for:

- Building taller than 100 feet, or
- Buildings with mass, stiffness, or geometric irregularities.

4.2.1.1 Linear Static Procedure (LSP)

The Linear Static Procedure shall be performed as follows:

1. A mathematical building model shall be developed in accordance with Section 4.2.2.
2. The pseudo lateral force shall be calculated and shall be distributed vertically as per procedure given below.
3. The building or component forces and displacements using linear, elastic analysis methods shall be calculated.
4. Diaphragm forces shall be calculated according to procedure described below.
5. The component actions shall be compared with the acceptance criteria of Section 4.2.4.5

In the Linear Static Procedure, the building is modeled with linearly-elastic stiffness and equivalent viscous damping that approximate values expected for loading to near the yield point. Design earthquake demands for the Linear Static Procedure are represented by static lateral forces whose sum is equal to the pseudo lateral force defined by Equation (3-1). The magnitude of the pseudo lateral force has been selected with the intention that when it is applied to the linearly elastic model of the building it results in design displacement amplitudes approximating maximum displacements that are expected during the design earthquake. If the building responds inelastically to the design earthquake, as will commonly be the case, the calculated internal forces will exceed those that would develop in the yielding building. The component forces in yielding structures calculated from linear analysis represent the total (linear and nonlinear) deformation of the component. The acceptability criteria reconciles the calculated forces with component capacities using component ductility related factors, m.

4.2.1.1.1 Pseudo Lateral Force and Story Shear Forces

Pseudo Lateral force and its distribution as Story Shear Forces are computed according to procedure described in chapter 3.The fundamental period of vibration of the building for use in calculation of Pseudo Lateral Force is determined using one of following calculation.

- For a one-story building with a single span flexible diaphragm, T is calculated in accordance with Eq. (4-1).

$$T = (0.1\Delta_w + 0.078\Delta_d)^{0.5} \qquad (4\text{-}1)$$

Where:

Δ_w and Δ_d are in-plane wall and diaphragm displacements in inches due to a lateral force equal to the weight tributary to diaphragm in the direction under consideration.

- For multiple-span diaphragms, a lateral force equal to the weight tributary to the diaphragm span under consideration shall be applied to each span of the diaphragm to calculate a separate period for each diaphragm span. The period that maximizes the pseudo lateral force shall be used for design of all walls and diaphragm spans in the building.

- Based on an eigenvalue (dynamic) analysis of the mathematical model of the building.

Equation (4.1) is derived from an assumed first-mode shape for the building for single span flexible diaphragms. For multiple spans with widely varying aspect ratios, this approach may be conservative. It is recommended a dynamic analysis be performed for such cases to determine the period. In this study time period of the building calculated through modal analysis.

4.2.1.1.2 Diaphragms

Diaphragms are designed to resist combined effects of inertial forces, calculated in accordance with Equation (4-2), developed at the level under consideration and horizontal forces resulting from offsets in, or changes in stiffness of, the vertical lateral-force-resisting elements above and below the diaphragm. Forces resulting from offsets in, or changes in stiffness of, the vertical lateral-force-resisting elements are taken as the forces due to the pseudo lateral force of Equation (3.1) without reduction, unless smaller forces can be justified by rational analysis and are directly added to the diaphragm inertial forces.

$$F_{px} = 1/C\ (\sum F_i / \sum w_i)\ w_x \qquad (4\text{-}2)$$

Where:

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

F_i = Lateral load applied at floor level i defined

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

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

C = Modification Factor defined in chapter 3.

4.2.1.1.3 Determination of Deformations

Structural deformations and story drifts shall be calculated using lateral forces described above.

4.2.1.2 Linear Dynamic Procedure (LDP)

The Linear Dynamic Procedure shall be performed as follows:

1. Develop a mathematical building model in accordance with Section 4.2.2;
2. Develop a response spectrum for the site;
3. Perform a response spectrum analysis of the building;
4. Modify the actions and deformations in accordance with Section 4.2.1.2.3;
5. Compute diaphragm forces in accordance with Section 4.2.1.2.4, if required;
6. Compute the component actions in accordance with Section 4.2.3.1;
7. Compare the component actions with the acceptance criteria of Section 4.2.4.5.

4.2.1.2.1 Modal Responses

Modal responses are combined using the SRSS (square root sum of the squares) or CQC (complete quadratic combination) method to estimate the response quantities. The CQC is used when modal periods associated with motion in a given direction are within 25%. The number of modes considered in the response spectrum analysis should be sufficient to capture at least 90% of the participating mass of the building in each of the building's principal horizontal axes. The SRSS method may be used to combine multidirectional effects. The CQC method is not used for combination of multidirectional effects.

4.2.1.2.2 Ground Motion Characterization

The seismic ground motions are characterized for use in the LDP by developing:

- A mapped response spectrum
- A site-specific response spectrum

4.2.1.2.3 Modification of Demands

With the exception of diaphragm actions and deformations, all actions and deformations calculated using the Linear Dynamic Procedure are multiplied by the modification factor, C, defined in Table 3-4.

4.2.1.2.4 Diaphragms

Floor diaphragms are analyzed for (1) the seismic forces calculated by dynamic analysis, but not be less than 85% of the forces calculated using Equation (4-2); and (2) the horizontal forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements above and below the diaphragm. Forces resulting from offsets in, or changes in stiffness of, the vertical lateral-force-resisting elements are taken to be equal to the elastic forces without reduction, unless smaller forces can be justified by rational analysis.

4.2.2 Mathematical Model for LSP & LDP

4.2.2.1Basic Assumptions

Buildings with stiff or rigid diaphragms may be modeled two-dimensionally if torsional effects are sufficiently small or indirectly captured; otherwise a three dimensional model should be developed. Lateral-force-resisting frames in buildings with flexible diaphragms are modeled as two dimensional assemblies of components; alternatively, a three-dimensional model shall be used with the diaphragms modeled as flexible elements.

4.2.2.2Horizontal Torsion

The effects of horizontal torsion are considered in a Tier 2 analysis. The total torsional moment at a given floor level shall be equal to the sum of the following two torsional moments:

1. Actual torsion resulting from the eccentricity between the centers of mass and the centers of rigidity.
2. Accidental torsion produced by horizontal offset in the centers of mass.

A building is considered torsionally irregular if the building has stiff or rigid diaphragms and the ratio $\eta = \delta_{max}/\delta_{avg}$, due to total torsional moment exceeds 1.2. In torsionally irregular buildings, the effect of accidental torsion are amplified by the factor, Ax, given in Equation (4-3).

$$A_x = [\delta_{max}/1.2\ \delta_{avg}]^2 \qquad (4\text{-}3)$$

Where:

δ_{max} = The maximum at any point of Diaphragm at level x.

δ_{avg} = The algebraic average of displacement at the extreme points of diaphragm at level x.

A_x = Shall be greater than 1 and need not to be greater than 3.

If the ratio, η, of the maximum displacement at any point on any floor diaphragm, to the average displacement, exceeds 1.50, a three-dimensional model shall be developed for a Tier 2 analysis. When $\eta < 1.5$, the forces and displacements calculated using two-dimensional models shall be increased by the maximum value of h calculated for the building. For this study a 3D model of the building is analyzed.

4.2.2.3Primary and Secondary Components

A primary element is an element that is considered to resist seismic forces in order for the structure to achieve selected performance level. A secondary component is an element that may attract seismic forces but is not required to resist seismic forces in order for the structure to achieve selected performance level. Only primary component need to be included in mathematical model for lateral force analysis unless the interaction of secondary component may result in less desirable seismic performance.

4.2.2.4. Diaphragms

Mathematical models of buildings with stiff diaphragms should include diaphragm flexibility. Mathematical models of buildings with rigid diaphragms should account for the rigidity of the diaphragms. The in-plane deflection of the diaphragm is calculated for an in-plane distribution of lateral force consistent with the distribution of mass, as well as all in-plane lateral forces associated with offsets in the vertical seismic framing.

4.2.2.5 Multidirectional Excitation Effects

Buildings are analyzed for seismic forces in any horizontal direction. Seismic displacements and forces are assumed to act non-concurrently in the direction of each principal axis of a building, unless the building is torsionally irregular or one or more components form part of two or more intersecting elements, in that case multidirectional excitation effects are considered. Multidirectional excitation is evaluated by applying 100% of the seismic forces in one horizontal direction plus 30% of the seismic forces in the perpendicular horizontal direction.

4.2.2.6 Vertical Acceleration

The effects of vertical excitation on horizontal cantilevers and pre-stressed elements are considered using static or dynamic analysis methods. Vertical earthquake motions are characterized by a spectrum with ordinates equal to 67% of those of the horizontal spectrum. Alternatively, vertical response spectra are developed using site-specific analysis.

4.2.3 Acceptance Criteria for LSP & LDP

Component actions are calculated considering effects of gravity loads as well as seismic forces. Component strengths are computed and are compared with strengths then shall be compared with the acceptance criteria given below.

4.2.3.1 Component Actions

Actions are classified as either deformation controlled or force-controlled. A deformation controlled action is defined as an action that has an associated deformation that is allowed to exceed the yield value and the maximum associated deformation is limited by the ductility capacity of the component. A force-controlled action is defined as an action that has an associated deformation that is not allowed to exceed the yield value.

4.2.3.1.1 Deformation Controlled Actions

Deformation-controlled design actions, QUD, are calculated according to Equation (4-4).

$$QUD = QG \pm QE \qquad (4\text{-}4)$$

where:

QUD = Action due to gravity loads and earthquake forces,

QG = Action due to gravity force

QE = Action due to earthquake forces calculated using LSP or LDP described in above sections.

4.2.3.1.2 Forced Controlled Actions

There are three methods for determining force controlled actions:

Method1: Force-controlled actions, QUF are calculated as the sum of forces due to gravity and the maximum force that can be delivered by deformation-controlled actions.

Method 2 Force-controlled actions are calculated according to Equation (4-5)

$$Q_{UF} = Q_G \pm Q_E/CJ \qquad (4\text{-}5)$$

Where:

QUF= Actions due to gravity loads and earthquake forces,

C = Modification Factor defined in Table 3-4,

J = a force-delivery reduction factor, taken as smallest DCR of the components in the load path or it is taken equal to 2.5 in the level of high seismicity, 2 for moderate seismicity, and 1.5 for low seismicity.

Method 3 For the evaluation of buildings analyzed using pseudo lateral force of (3-2), Equation (4-6), with C=1.0, shall be used.

4.2.3.2 Component Strength

Component strength is taken as the Expected strength, Q_{CE}, for deformation controlled actions, and as Nominal strength, Q_{CN}, for force controlled actions. The expected strength is assumed equal to the nominal strength multiplied by 1.25.

4.2.3.3 Acceptance Criteria for the LSP & LDP

4.2.3.3.1 Deformation-Controlled Actions

The acceptability of deformation-controlled primary and secondary components are determined in accordance with Equation (4-8).

$$Q_{CE} \geq Q_{UD}/m \qquad (4\text{-}8)$$

Where:

Q_{UD} = Action due to gravity and earthquake loading calculated as described above.

m = Component demand modifier to account for the expected ductility of the component; the *m*-factor is taken from Tables 4.1(taken from the handbook) based on the level of performance and component characteristics.

Q_{CE} = Expected strength of the component at the deformation level under consideration.

Table 4.1 m Factor for Concrete Component

Component/Conditions	Primary		Secondary	
	LS	IO	LS	IO
Beams, flexure				
Ductile[1]				
$v \leq 3\sqrt{f'_c}$	8	3	8	3
$v \geq 6\sqrt{f'_c}$	4	2.5	4	2.5
Non-Ductile	2.5	1.5	3	1.5
Columns, flexure				
Ductile[1]				
$\frac{P}{A_g f'_c} \leq 0.1$	5	3	5	3
$\frac{P}{A_g f'_c} \geq 0.4$	2	1.5	2	1.5
Non-Ductile	2.5	1.5	3	2
$\frac{P}{A_g f'_c} \leq 0.1$	1.5	1.5	1.5	1.5
$\frac{P}{A_g f'_c} \geq 0.4$				
Beams controlled by shear	2	1.5	3.5	2.5
Beam-Column Joints	(2)	(2)	(2)	(2)
Slab-Column Systems[5]				
$\frac{V_g}{V_c} \leq 0.1$	3	3	3	3
$\frac{V_g}{V_c} \geq 0.4$	1.5	1.5	1.5	1.5
Infilled Frame Columns Modeled as Chords				
Confined along entire length	4	1.5	5	1.5
Not confined	1.5	1.5	1.5	1.5
Shear Walls Controlled by Flexure				
With confined boundary				
$a \leq 0.1$[3]	5	3	6	3
$a \geq 0.25$	3	1.5	4	1.5
Without confined boundary				
$a \leq 0.1$	3	2	4	2
$a \geq 0.25$	2	1.5	2.5	1.5
Coupling Beams	2.5	1.5	4	2
Shear Walls Controlled by Shear	2.5	1.5	3	2

4.2.3.5.2 Force-Controlled Actions

The acceptability of force-controlled primary and secondary components shall be determined in accordance with Equation (4-9).

$$Q_{CN} \geq Q_{UF} \qquad (4\text{-}9)$$

Where:

Q_{UF}= Action due to gravity and earthquake loading.

Q_{CN}= Nominal strength of the component at the deformation level under considering all co-existing actions due to gravity and earthquake loads.

4.3 CHECKING OF RESULTS

A 3D model of the building is developed on Etabs (V9.6) and Linear Static & Linear Dynamic Analysis keeping in view the modeling consideration discussed above. The effective stiffness is taken as 50% for beams and 70 % for columns. All the regarding the building model is given in appendix A. Analysis results are given in Appendix C. Acceptance criteria as explained in Sec 4.2.3 is utilized to verify the adequacy of members. For columns displacement controlled action "m" is taken as 1.5. For Beam moments m value of is taken as 4. For Forced controlled actions 2^{nd} method from the methods described above is taken and value of J is 1.5 (for moderate seismicity).

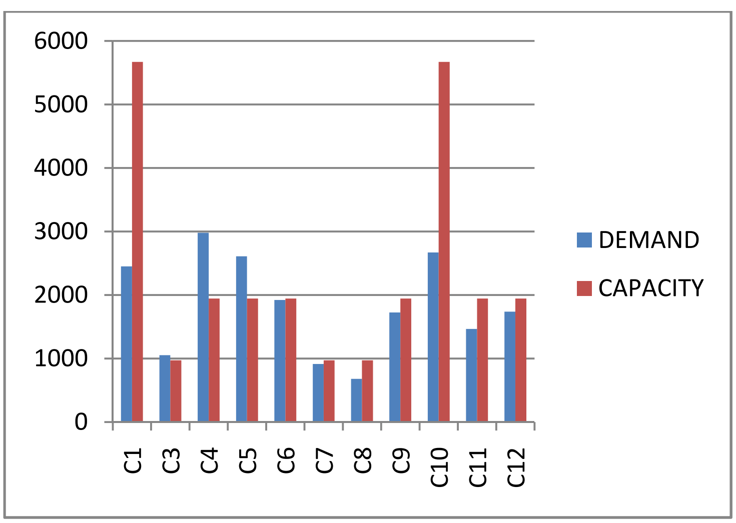

Fig 4.1 Axial Load Demand and Capacity of selected columns

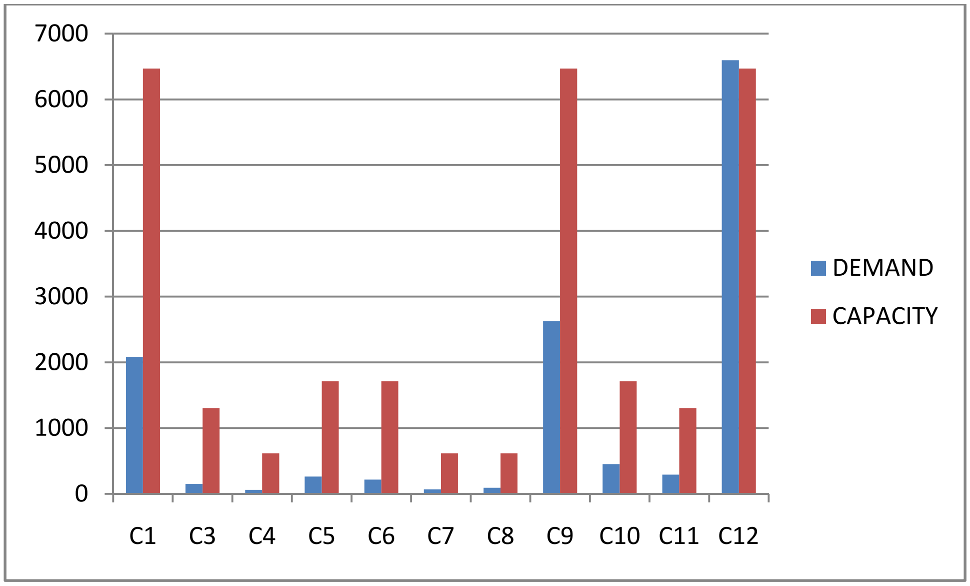

Fig 4.2 Moment M33 Demand and Capacity of Selected Columns

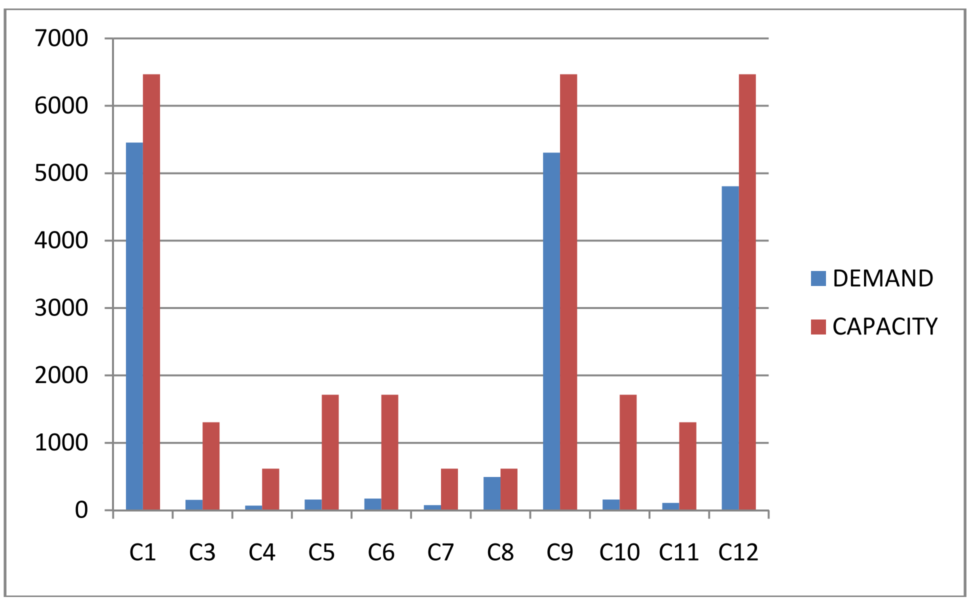

Fig 4.3Moment M22 Demand and Capacity of selected columns

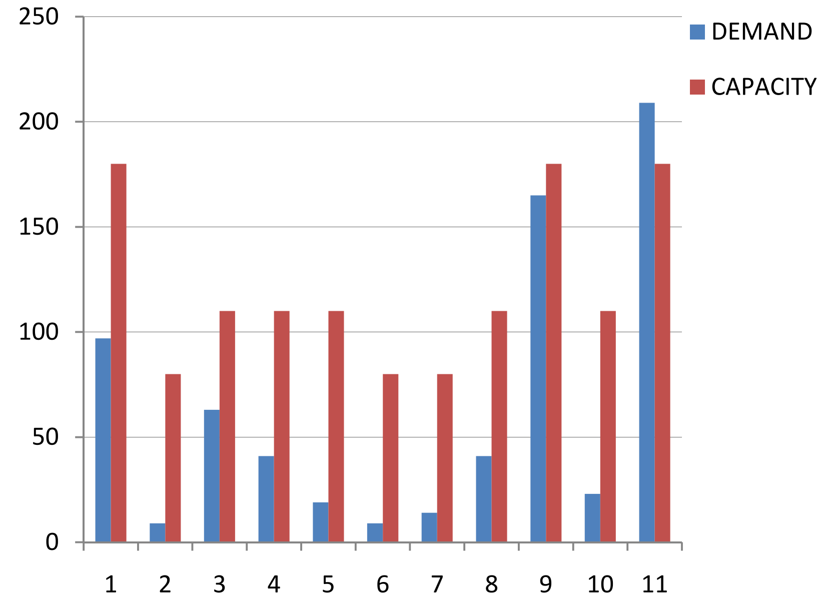

Fig 4.4Shear V2 Demand and Capacity of selected columns

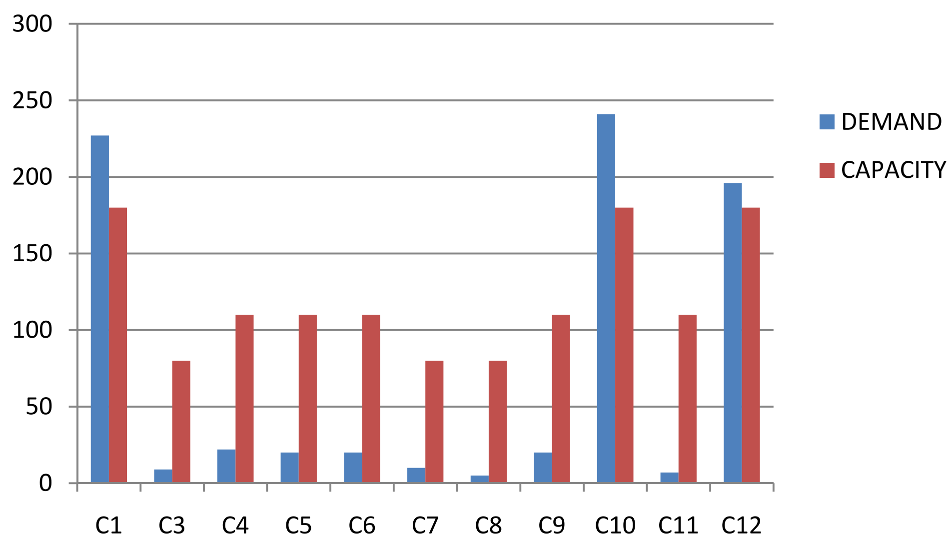

Fig 4.5 Shear V3 Demand and Capacity of selected columns

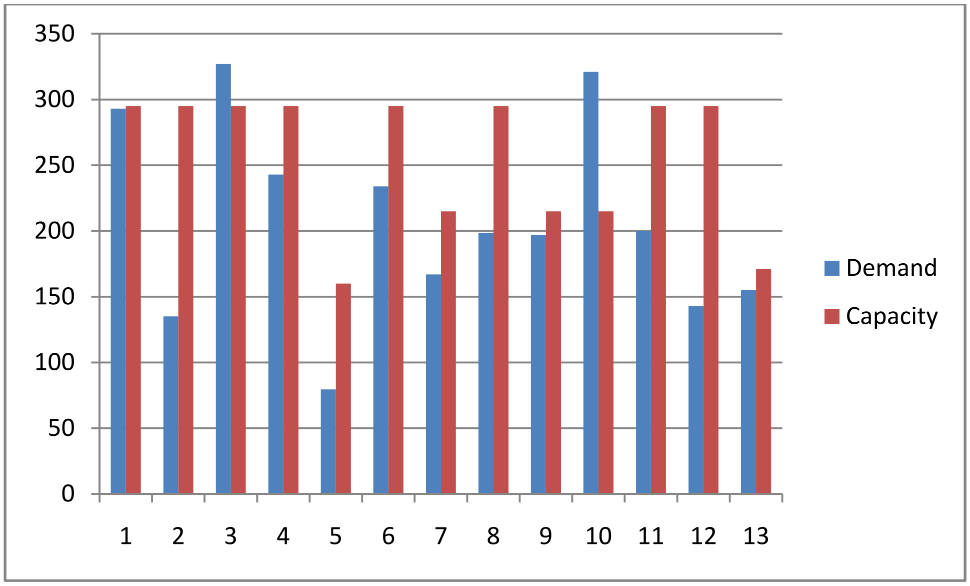

Fig4.6 Moment Demand and Capacity of 7th Storey Beams

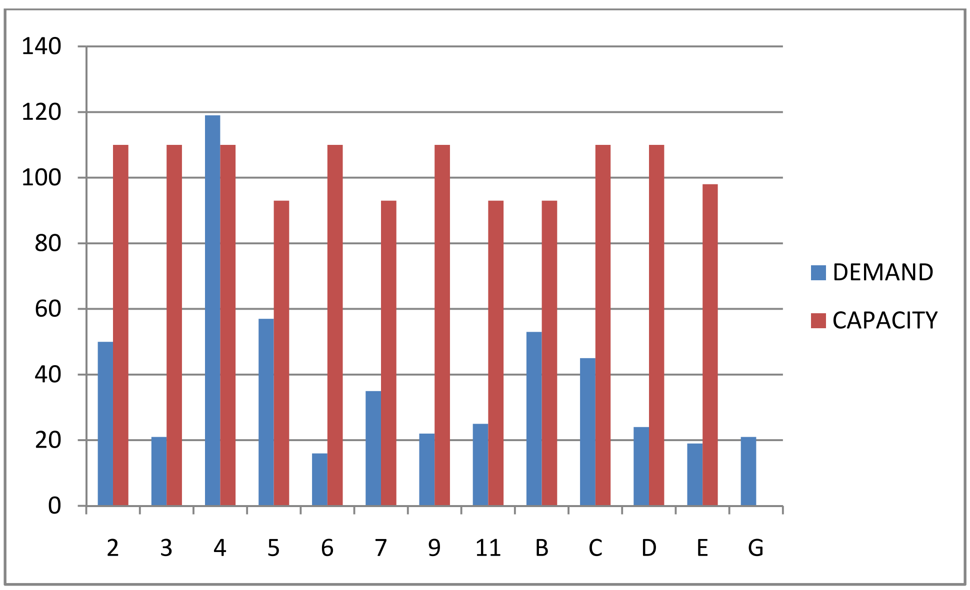

Fig4.7 Shear demand and capacity of & 7th Storey Beams

4.3.1Conclusion of Tier 2 Results

1. The modal analysis results shown in the appendix C showed that there is a significant participation of higher effects. Also there are pronounced torsional effects in 1st and 3rd mode.
2. Story drifts are with in the allowable limits
3. Maximum Roof displacement in X direction is 8.11 inches and 7.36 inches in Y direction.
4. Shear demand in few of the basement columns exceed the shear capacity.
5. Moment Demand to Capacity Ratio is high in few of the columns.
6. Axial load demand to capacity of column exceeds 0.1 f'_c, but as it was verified that axial stress overturning was within allowable limits so that is not a problem.
7. Shear Demand in beams is less than shear capacity accept one Beam 4.
8. Flexural Capacity of beams is adequate

To verify the defects determined in Tier 2 Analysis, Further analysis should be carried. For Tier analysis Time History analysis should be as it is observed in modal analysis that significant higher mode effect is present.

CHAPTER 5

TIER 3 ANALYSES

5.1 General

For buildings requiring further investigation, a Tier 3 Evaluation is carried out either for entire building or for deficiencies identified in initial Tiers. Tier 1 and Tier 2 evaluations are conservative because of the simplifying assumptions involved in their application. More detailed and more accurate evaluations may employ less conservatism and may therefore reveal that building components found deficient at Tier 1 and or Tier 2 evaluations are satisfactory to resist seismic forces. No evaluation procedures more detailed than the Tier 1 and Tier 2 are presently available. Therefore, in order to make more detailed evaluations, design procedures are adopted. Provisions intended for design are used for evaluation by inserting existing conditions in the analysis procedures intended for design. Expected performance of existing components can be evaluated by comparing calculated demands with the capacities.

5.2 Available Procedures

A Tier 3 Evaluation is performed using one of the two following procedures:

5.2.1 Provisions for Seismic Rehabilitation Design

A component-based evaluation procedure developed for seismic rehabilitation of existing buildings is used for a Tier 3 Evaluation. The analysis procedures for such a detailed evaluation include linear and nonlinear methods for static or dynamic analysis of buildings. Seismic Rehabilitation guidelines include ATC 40, FEMA 356 and such like others. FEMA 356 is the recommended design procedure for adaption to evaluation. The linear analysis methods should implicitly or explicitly recognize nonlinear response. Force levels used for analysis in provisions for seismic rehabilitation of existing buildings are reduced to 75% for Tier 3 Evaluation as these procedures are intended for design. In Tier 1 and 2 Evaluations, reduction factor is taken into account in various factors including material strength and m factor. The use of reduction factor is justified by following factors (ASCE 31-03).

1. The reduction factor is intended to reduce earthquake motion from conservative level used in design to more appropriate for evaluating existing building.
2. The actual strength of component will be greater than used in evaluation.
3. An existing building does not need to have the same level of safety as new building since the remaining useful life of an existing building is less than a new building.

A building fulfilling all provisions for the seismic evaluation of existing buildings is considered as compliant with Tier 3 Evaluation.

5.2 2 Provisions for Design of New Buildings

Well-established provisions for the design of new buildings can be used for Tier 3 Evaluation. Acceptable provisions for such a detailed evaluation include ASCE 7-02, the International Building Code (IBC 2006) and UBC etc. Demand levels used for analysis in provisions for seismic design of new buildings are reduced by 75% when used in a Tier 3 Evaluation Linear analysis method should implicitly or explicitly recognize nonlinear response. A building meeting all provisions for the design of the new buildings is considered as compliant.

However Provisions for design of new buildings may not be suitable for evaluation of existing buildings because they are based on construction detail and the building configuration that meet specific standard. These standards may not describe the construction details and configurations or the archaic materials of construction frequently found in existing buildings.

5.3 Methods of Analysis

A structural analysis of the mathematical model of the structure is required to determine force and displacement demands in various components of the structure and compared with capacities to evaluate seismic performance. Several analysis methods, elastic and inelastic, linear and dynamic are available to predict the seismic performance of the structures.

5.3.1 Elastic Methods of Analysis

Elastic methods are force based methods and the structures are assumed to respond elastically. These methods include code static lateral force procedure, code dynamic procedure and elastic procedure using demand-capacity ratios (Pseudo Lateral Force Procedure). These methods are force-based procedures and assume that structures respond elastically to earthquakes.

In code static lateral force procedure, (UBC & IBC etc.) a static analysis is performed by subjecting the structure to reduced lateral forces obtained by scaling by a reduction factor, "R" which depends on the structural system. It is assumed that the actual strength of structure is higher than the design strength and the structure is able to dissipate energy through yielding.

In code dynamic procedure, demands on various components are determined by an elastic dynamic analysis. The dynamic analysis may be either a response spectrum analysis or an elastic time history analysis. Sufficient number of modes must be considered to have a mass participation of at least 90% for response spectrum analysis.

In (DCR) procedure (Pseudo Lateral Force Procedure), the force actions are compared to corresponding capacities. Demands for DCR calculations must include gravity effects. While code static lateral force and code dynamic procedures reduce the full earthquake demand by an R-factor, the DCR approach takes the full earthquake demand without reduction and adds it to the gravity demands. Demand is reduced at component level by a factor “m” accounts for the ductility for particular action associated with that component. This approach is more realistic. DCRs approaching 1.0 (or higher) may indicate potential deficiencies. This approach is utilized in Tier 2 Analysis of this study.

Although engineers are more familiar with force-based procedures but they have certain drawbacks. Structural components are evaluated in the elastic range and post-elastic behavior of structures could not be identified by an elastic analysis. However, post-elastic behavior should be considered as almost all structures are expected to deform in inelastic range during a strong earthquake. The seismic force reduction factor "R" is utilized to account for inelastic behavior indirectly at global level. These methods predict elastic capacity of structure and indicate where the first yielding will occur and don’t account for the redistribution of forces and predict failure mechanisms. Real deficiencies present in the structure could be missed. Force-based methods primarily provide life safety but they can’t provide damage limitation and easy repair.

5.3.2 Inelastic Methods of Analysis

Structures behave inelastically under a strong earthquake so inelastic analytical procedures are required for accurate analysis. Inelastic analytical procedures help to understand the actual behavior of structures by identifying failure modes and the potential for progressive collapse. Inelastic analysis procedures basically include inelastic time history analysis and inelastic static analysis (pushover analysis).

The inelastic time history analysis is the most accurate method. However, the use of inelastic time history analysis is limited because dynamic response is very sensitive to modeling and ground motion characteristics. It requires proper modeling of cyclic load deformation characteristics and availability of a set of representative ground motion records. Also the computation time for input preparation and interpreting voluminous output make the use of inelastic time history analysis impractical for seismic performance evaluation.

Inelastic static analysis, or pushover analysis, has been the preferred method for seismic performance evaluation due to its simplicity. It is a static analysis that directly incorporates nonlinear material characteristics. Inelastic static analysis procedures include Capacity Spectrum Method (ATC 40), Displacement Coefficient Method (FEMA 273) and the Secant Method

Displacement-based procedures provide a more rational approach to these issues compared to force-based procedures by considering inelastic deformations rather than elastic forces. The analytical tool for evaluation process should also be relatively simple which can capture critical response parameters that significantly affect the evaluation process.

5.4 Selection of Detailed Procedures

An analysis of the building shall be performed using one of the procedures described above keeping in view the limitations discussed below. Linear procedures are appropriate when the expected level of nonlinearity is low. This is measured by component demand to capacity ratios (DCRs) (< 2.0) calculated from linear analysis.

The procedures that explicitly recognize the nonlinear response of building components in earthquakes gives most accurate results, nonlinear analysis methods should be selected for complex or irregular buildings and for higher performance levels. For the Buildings with one or more of the following characteristics are evaluated using nonlinear procedures (ASCE 31-03).

- The fundamental period of the building, $T \geq 3.5 \times S_{D1}/ S_{DS}$
- The ratio of the building's horizontal dimension at any story exceeds 1.4 times the horizontal dimension at an adjacent story (excluding penthouses & mezzanines).
- The building has a torsional stiffness irregularity in any storey. A torsional stiffness irregularity exist in any story if the diaphragm above the story under consideration is not flexible and the results of the analysis indicate that drift along any side of the structure is more than 150 percent of the average story drift.
- The building has a vertical mass or stiffness irregularity. A vertical mass or stiffness irregularity exists when the average drift in any storey exceeds that of the storey above or below by more than 150 percent.
- The lateral-force-resisting system is non-orthogonal.

Static procedures are appropriate when higher mode effects are not significant. Static analysis is applicable for short and regular buildings. Dynamic procedures are required for tall buildings, buildings with torsional irregularities, or non-orthogonal systems.

5.5 Nonlinear Dynamic Procedure

In the Nonlinear Dynamic Procedure (NDP) seismic analysis of the building, a mathematical model directly incorporating the nonlinear load deformation characteristics of individual components and elements of the building is subjected to earthquake shaking represented by ground motion time histories to obtain forces and displacements. For the NDP, earthquake shaking is characterized by discretized recorded or synthetic earthquake records.

With the NDP, the design displacements are not established using a target displacements as in Pushover analysis, but instead are determined directly through dynamic analysis using ground motion time histories. Calculated response can be highly sensitive to characteristics of individual ground motions; therefore, the analysis should be carried out with a minimum 0f three ground motion records. Because the numerical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake. Calculated demand displacements and internal forces are compared directly with the capacity.

5.6 Tier 3 Analysis of Case Study 1

As Case study 1 was found to be deficient in Tier 2 so more detailed, Tier 3 analysis is carried out. As it was identified at Tier 2 analysis that the building has considerable higher mode effects so it should be evaluated using dynamic analysis (i.e Time Histories Analysis). Time History analysis is performed for Tier3 evaluation of the case study 1.

Time History records are taken from Zameer study (MS Thesis 2010). Three time history load cases were defined as EQ1, EQ 2 and EQ3. Each load case comprised of two orthogonal horizontal components, EQ11 in x-direction and EQ 12 in Y-direction and so on. Both horizontal components of earthquake were applied simultaneously. A scaling factor of 386.4 was kept to convert from g units to inch units. Output time step was kept as 0.1 second which is approximately one tenth of time period of structure; hence response at 10 discrete time points will be captured in a single oscillation. Output steps were kept as 1.5 times duration of excitation divided by output time step .This was done so that complete time history response is captured till it damps out.

5.6.1 Results of Time History Analysis

Results of critical Basement columns are shown below. The results show that all columns have adequate shear and flexural capacity. So finally it is concluded that Case Study 1 Building is compliant with the specification of ASCE 31-03 standard.

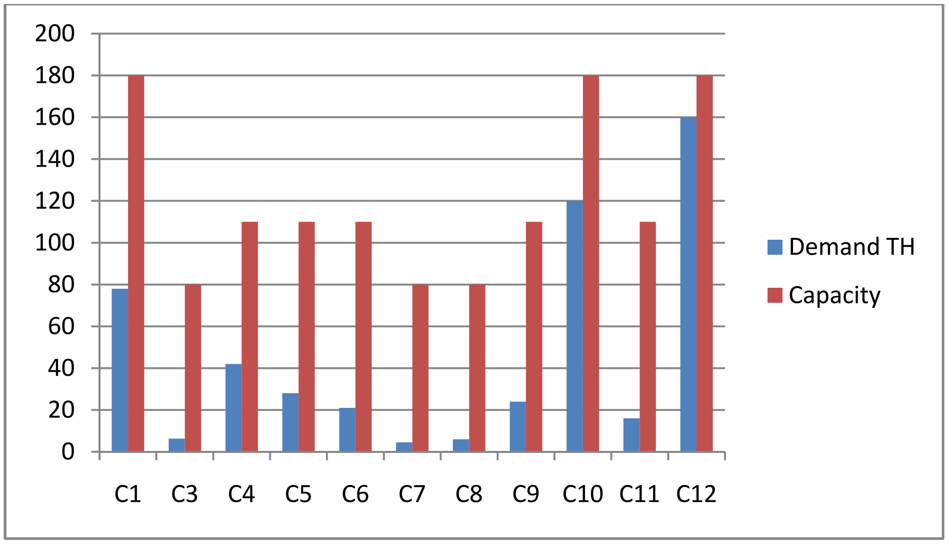

Fig 5.1 Shear V2 Demand TH and Capacity

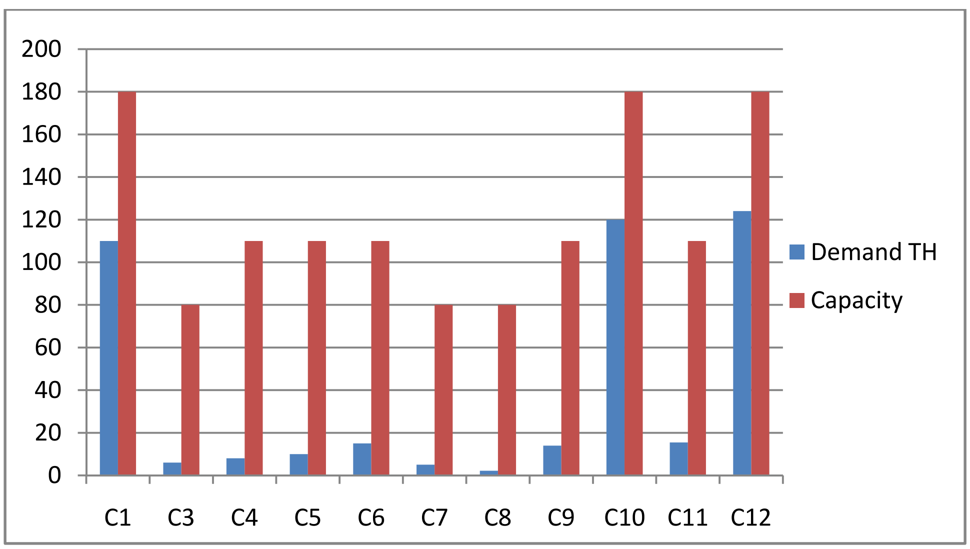

Fig 5.2 Shear V3 Demand and capacity

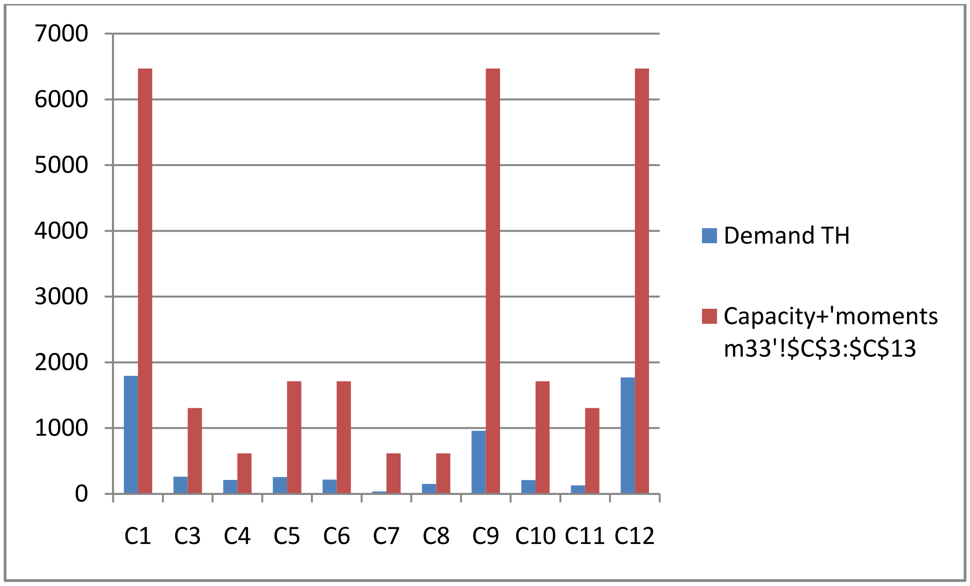

Fig 5.3 M33 Demand TH and Capacity

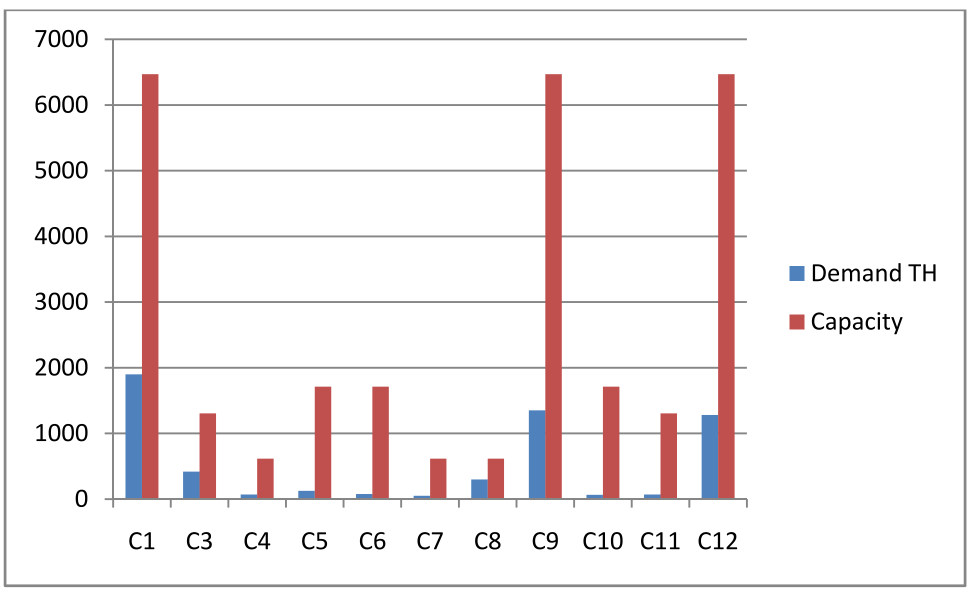

Fig 5.4 M33 Demand TH and Capacity

5.7 Description of Pushover Analysis

Pushover analysis is an approximate nonlinear analysis method in which the structure is subjected to monotonically increasing lateral loads until a target displacement is reached. A mathematical model of the building which includes load-deformation diagrams of all lateral force resisting elements is generated and gravity loads are applied initially. A predefined lateral load pattern which is distributed along the building height is then applied. The lateral forces are increased until some members yield and the structural model is modified to account for the reduced stiffness of yielded members. The lateral forces are again increased until additional members yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with base shear to get the global capacity curve. Pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines because it is conceptually and computationally simple. Pushover analysis determines the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure. Pushover analysis determines the force demand for brittle members and deformation demand for ductile members. It identifies the weak links in the structure and is therefore very useful for efficient retrofitting. Pushover analysis exposes the design weaknesses that may remain hidden in an elastic analysis. These are story mechanisms, excessive deformation demands, strength irregularities.

Pushover analysis can be performed as force-controlled or displacement controlled. Force-controlled is used when the load is known (such as gravity loading). In displacement-controlled procedure is used where the magnitude of applied load is not known in advance. The load is increased until the control displacement reaches a specified value. Generally, roof displacement at the center of mass of structure is chosen as the control displacement. The internal forces and deformations at the target displacement give inelastic strength and deformation demands which is compared with available capacities to find a performance point. In this study displacement based procedure is used for Seismic Loads and forced based procedure for gravity loads.

Available simplified conventional nonlinear static procedures are as follow.

- **Capacity Spectrum Method**, that uses intersection of capacity (pushover) curve and a reduced response spectrum in spectral coordinates (Acceleration Displacement Response Spectrum Format) to find a performance point. The specifications of this method are covered in ATC40.
- **Displacement Coefficient Method** described in FEMA-356 [20] is a non-iterative approximate procedure based on displacement modification factors. The expected maximum inelastic displacement of nonlinear MDOF system is obtained by modifying the elastic spectral displacement of an equivalent SDOF system with a series of coefficients. The procedure proposed by Newmark and Hall is based on the estimation of inelastic response spectra from elastic response spectra while displacement modification factor varies depending on the spectral region.

5.7.1 PUSHOVER ANALYSIS WITH SAP2000

Nonlinear static pushover analysis is a very powerful feature offered in the Non-linear version of SAP2000. Pushover analysis can be performed on both two and three dimensional structural models and as a force controlled or deformation controlled.

Geometric nonlinearity can be considered through P-delta effects or P-delta effects plus large displacements. In Sap 2000 Nonlinearity of the frame element is represented by specified hinges and a capacity drop occurs for a hinge when the hinge reaches a negative-sloped portion of its force-displacement curve during pushover analysis (Fig 5.5).

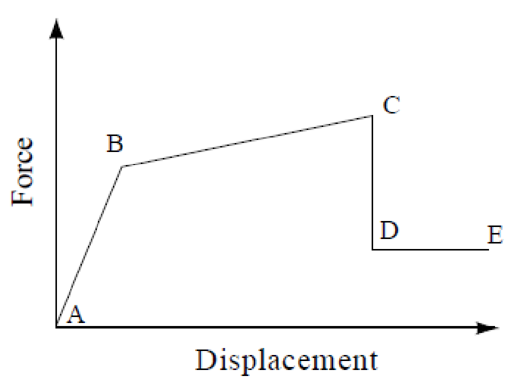

Figure 5.5 Generalized Force Displacement Characteristic of Frame element

Three different member unloading methods are provided in SAP 2000 to remove the load that the hinge was carrying and redistribute it to the rest of the structure. For "Unload Entire Structure" option, when the hinge reaches point C on its force displacement curve (Figure 5.5), try to increase the base shear. If the lateral deformation increases the analysis proceed otherwise base shear is reduced by reversing the lateral load on the whole structure until the force in that hinge is consistent with the value at point D on its force-displacement curve (Figure 5.5).As unloading completes, base shear is again increased, and other elements of the structure pick up the load that was removed from the unloaded hinge. For "Apply Local Redistribution" option, only the element in which hinge has formed is unloaded instead of unloading the entire structure. In the "Restart Using Secant Stiffness" whenever any hinge reaches point C (Fig 5.5) on force-displacement curve, all hinges that have become nonlinear are reformed using secant stiffness properties and the analysis is restarted.

In Fig 5.5, Point A corresponds to unloaded condition and point B represents yielding of the element. Point C corresponds to nominal strength and deformation at which significant strength degradation begins. The drop from C to D represents the initial failure of the element and resistance to lateral loads beyond point C is unreliable. The resistance from D to E allows the frame elements to resist gravity loads. Beyond point E, the maximum deformation capacity, gravity load can no longer be sustained. Hinges can be assigned at any location of potential yielding. Uncoupled moment (M2 and M3), torsion (T), axial force (P) and shear (V2 and V3) force-displacement relations can be defined. As the column axial load changes under lateral loading, there is also a coupled P-M2-M3 (PMM) hinge which yields based on the interaction of axial force and bending moments at the hinge location. More than one type of hinge can be assigned at the same location of a frame element.

SAP2000 considers three types of hinge properties. These are default hinge properties, user-defined hinge properties and generated hinge properties. Default hinges and user-defined hinges can be assigned to frame elements. When these hinge properties (default or user-defined) are assigned to a frame element, the program automatically creates generated hinge property. Default hinge properties could not be modified and are section dependent. The built-in default hinge properties for steel and concrete members are based on ATC-40 and FEMA-273 criteria. User-defined hinge properties can be based on default properties or they can be fully user defined.

5.7.2 Limitations of Pushover Analysis

Although pushover analysis has advantages over elastic analysis procedures, however the assumptions for pushover analysis and limitations of current pushover procedures must be identified. The estimate of target displacement, selection of lateral load patterns and identification of failure mechanisms due to higher modes of vibration are important issues that affect the accuracy of pushover results.

In pushover analysis, the target displacement of a MDOF system is estimated as the displacement demand for the corresponding equivalent SDOF system. A shape vector representing the deflected shape of the MDOF system is used to obtain the properties of an equivalent SDOF system. A fixed shape vector, elastic first mode, is used for simplicity without considering the higher mode effects by conventional approaches.

The distribution of inertia forces vary with the severity of earthquake and with time during earthquake since however, in pushover analysis, generally an invariant lateral load pattern is used. The lateral load patterns used in pushover analysis are proportional to product of story mass and displacement associated with a shape vector at the story under consideration. Commonly used lateral force patterns are uniform, elastic first mode, “code" distributions and a single concentrated horizontal force at the top of structure. The invariant lateral load patterns could not predict potential failure modes due to middle or upper story mechanisms caused by higher mode effects. Invariant load patterns can provide adequate predictions if higher mode effects are not significant. These limitations have led many researchers to propose adaptive load patterns which consider the changes in inertia forces with the level of inelasticity. Although some improved predictions have been obtained from adaptive load patterns, they make pushover analysis computationally demanding and conceptually complicated

This study is restricted only to the application of conventional pushover analysis. Due to limitation of conventional pushover analysis to incorporate higher mode effects, Case Study 1 cannot be accurately assessed. As Case Study 1 there are considerable higher modes effect due irregularities. So it should be analyzed with Time History analysis. Other possible ways are use adaptive or modal pushover analysis technique which are beyond the scope of this study. In order to illustrate the application of pushover analysis a regular 7 storey building is considered.

5.8 Pushover Analysis of Case Study 2

In this study Push over Analysis is performed on a typical regular 7 storey RC Moment Frame building. The details of building layout and member sizes are given in appendix A. Pushover analysis is performed using SAP 2000 (V14). Auto Hinges provided in the software are assigned to frame elements. Gravity Load Case is defined as force controlled while Earthquake Loading are defined as displacement controlled with a target displacement of 12 inches.. Demand is calculated based on F-10 (Islamabad) Site Response spectra. Demand from ATC 40 criteria using BCP (2006) seismic parameters for zone 2B is also plotted against with the capacity curve. The results obtained from the pushover analysis are shown below.

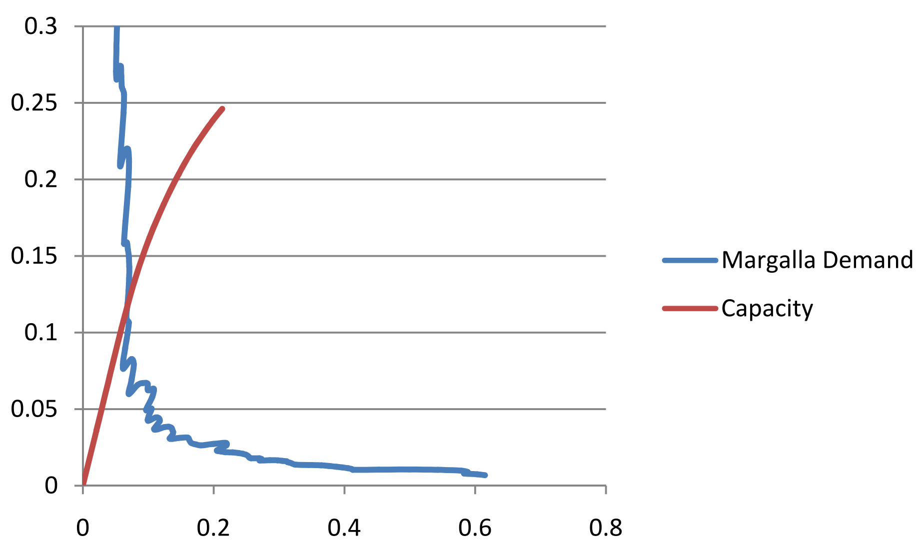

Fig 5.6 Capacity and Margalla Demand in X direction

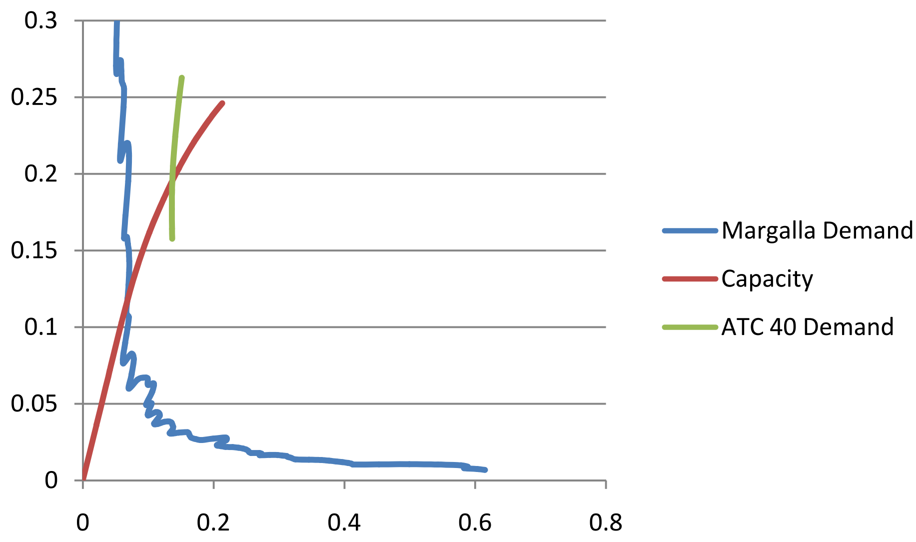

Fig 5.7 Comparison of ATC 40 & F-10 Site RS Performance Point in X direction

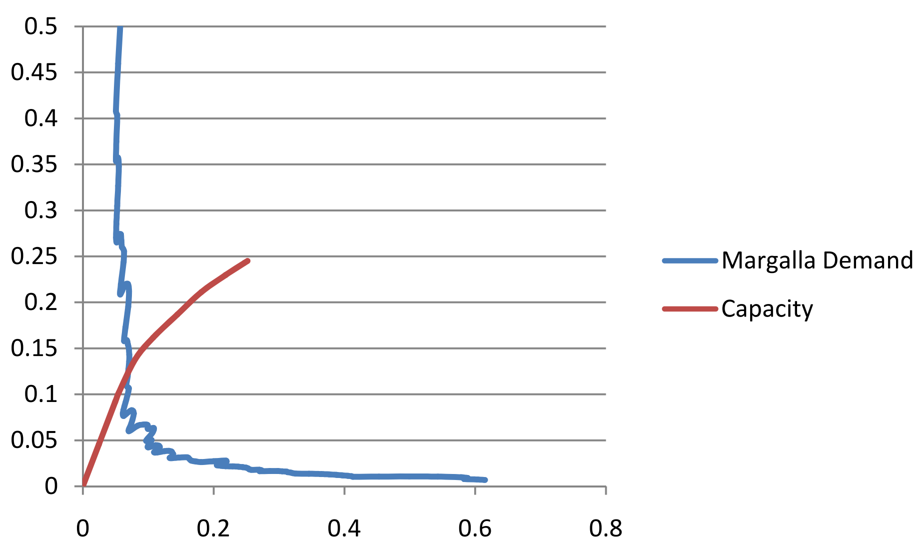

Fig 5.8 Capacity and Margalla Demand in Y direction

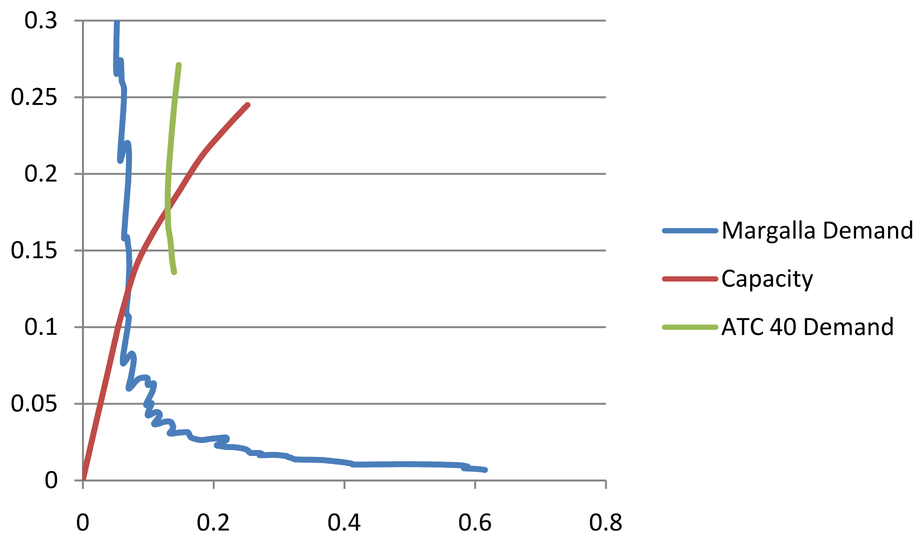

Fig 5.9 Comparison of ATC 40 & F-10 Site RS Performance point in Y direction

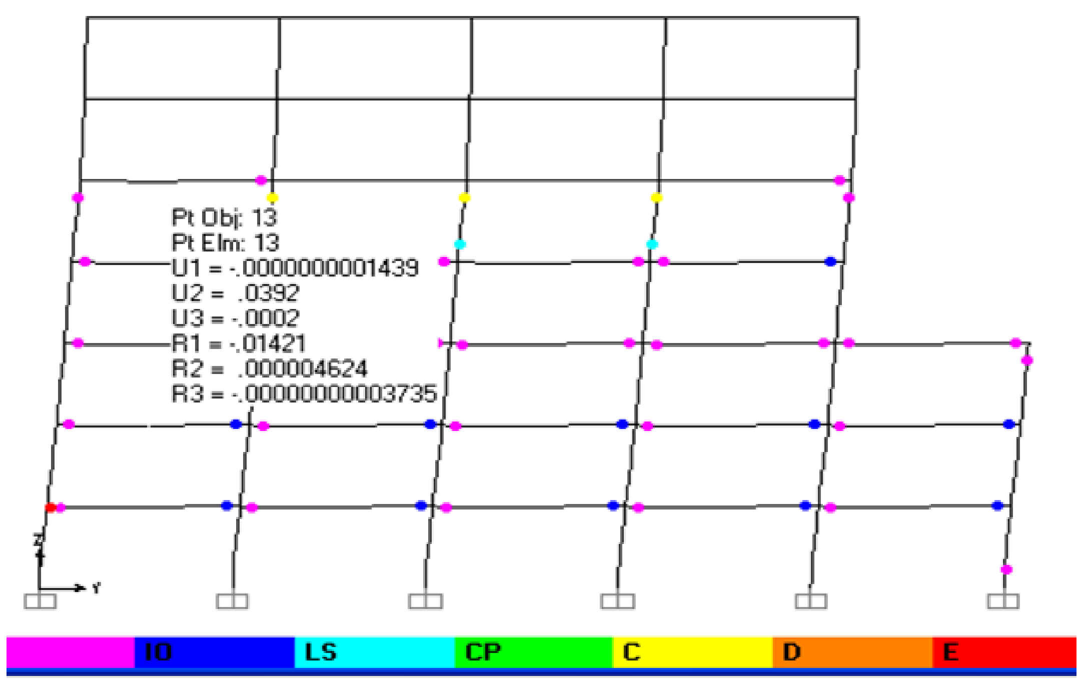

Figure 5.10 Formation of Hinges for PO2 load case

The Demand & Capacity in Acceleration displacement Response format showing the performance point of the building in X and Y direction is obtained with site specific RS based demand. Performance point for ATC 40 and F-10 site Response spectra is compared. Site Based Response spectra shows lesser demand than ATC 40. It means that ATC 40 with use of BCP parameters gives conservative results. Formation of hinges is shown in the results. The results show that collapse hinges are developing in columns at storey level 4, which is undesirable. All other hinges are developing in the beams, so 4th story column should be strengthened.

Then the building under consideration is strengthened by column jacketing of the weak columns identified in 4th story and by placing steel bracing. Strengthening layout is shown in appendix A. Strengthened Column X-Sections are defined using Section Designer in Sap 2000. User Defined Hinges based on PM interaction diagram (from Sap 2000 section designer) are defined for strengthened columns and steel braces. Pushover analysis is run using same load cases and results are viewed. This time Hinges are developed in the beams, which is desirable.

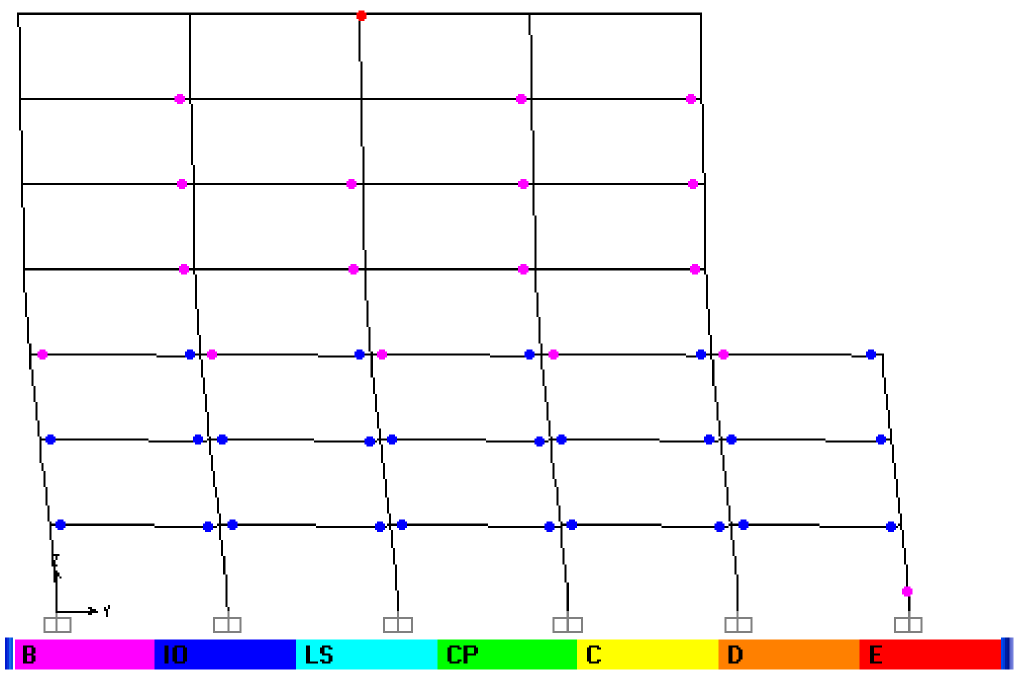

Figure 5.11 Plastic Hinge Formation of Retrofitted Structure

CHAPTER 5

TIER 3 ANALYSES

5.1 General

For buildings requiring further investigation, a Tier 3 Evaluation is carried out either for entire building or for deficiencies identified in initial Tiers. Tier 1 and Tier 2 evaluations are conservative because of the simplifying assumptions involved in their application. More detailed and more accurate evaluations may employ less conservatism and may therefore reveal that building components found deficient at Tier 1 and or Tier 2 evaluations are satisfactory to resist seismic forces. No evaluation procedures more detailed than the Tier 1 and Tier 2 are presently available. Therefore, in order to make more detailed evaluations, design procedures are adopted. Provisions intended for design are used for evaluation by inserting existing conditions in the analysis procedures intended for design. Expected performance of existing components can be evaluated by comparing calculated demands with the capacities.

5.2 Available Procedures

A Tier 3 Evaluation is performed using one of the two following procedures:

5.2.1 Provisions for Seismic Rehabilitation Design

A component-based evaluation procedure developed for seismic rehabilitation of existing buildings is used for a Tier 3 Evaluation. The analysis procedures for such a detailed evaluation include linear and nonlinear methods for static or dynamic analysis of buildings. Seismic Rehabilitation guidelines include ATC 40, FEMA 356 and such like others. FEMA 356 is the recommended design procedure for adaption to evaluation. The linear analysis methods should implicitly or explicitly recognize nonlinear response. Force levels used for analysis in provisions for seismic rehabilitation of existing buildings are reduced to 75% for Tier 3 Evaluation as these procedures are intended for design. In Tier 1 and 2 Evaluations, reduction factor is taken into account in various factors including material strength and m factor. The use of reduction factor is justified by following factors (ASCE 31-03).

1. The reduction factor is intended to reduce earthquake motion from conservative level used in design to more appropriate for evaluating existing building.
2. The actual strength of component will be greater than used in evaluation.
3. An existing building does not need to have the same level of safety as new building since the remaining useful life of an existing building is less than a new building.

A building fulfilling all provisions for the seismic evaluation of existing buildings is considered as compliant with Tier 3 Evaluation.

5.2 2 Provisions for Design of New Buildings

Well-established provisions for the design of new buildings can be used for Tier 3 Evaluation. Acceptable provisions for such a detailed evaluation include ASCE 7-02, the International Building Code (IBC 2006) and UBC etc. Demand levels used for analysis in provisions for seismic design of new buildings are reduced by 75% when used in a Tier 3 Evaluation Linear analysis method should implicitly or explicitly recognize nonlinear response. A building meeting all provisions for the design of the new buildings is considered as compliant.

However Provisions for design of new buildings may not be suitable for evaluation of existing buildings because they are based on construction detail and the building configuration that meet specific standard. These standards may not describe the construction details and configurations or the archaic materials of construction frequently found in existing buildings.

5.3 Methods of Analysis

A structural analysis of the mathematical model of the structure is required to determine force and displacement demands in various components of the structure and compared with capacities to evaluate seismic performance. Several analysis methods, elastic and inelastic, linear and dynamic are available to predict the seismic performance of the structures.

5.3.1 Elastic Methods of Analysis

Elastic methods are force based methods and the structures are assumed to respond elastically. These methods include code static lateral force procedure, code dynamic procedure and elastic procedure using demand-capacity ratios (Pseudo Lateral Force Procedure). These methods are force-based procedures and assume that structures respond elastically to earthquakes.

In code static lateral force procedure, (UBC & IBC etc.) a static analysis is performed by subjecting the structure to reduced lateral forces obtained by scaling by a reduction factor, "R" which depends on the structural system. It is assumed that the actual strength of structure is higher than the design strength and the structure is able to dissipate energy through yielding.

In code dynamic procedure, demands on various components are determined by an elastic dynamic analysis. The dynamic analysis may be either a response spectrum analysis or an elastic time history analysis. Sufficient number of modes must be considered to have a mass participation of at least 90% for response spectrum analysis.

In (DCR) procedure (Pseudo Lateral Force Procedure), the force actions are compared to corresponding capacities. Demands for DCR calculations must include gravity effects. While code static lateral force and code dynamic procedures reduce the full earthquake demand by an R-factor, the DCR approach takes the full earthquake demand without reduction and adds it to the gravity demands. Demand is reduced at component level by a factor “m” accounts for the ductility for particular action associated with that component. This approach is more realistic. DCRs approaching 1.0 (or higher) may indicate potential deficiencies. This approach is utilized in Tier 2 Analysis of this study.

Although engineers are more familiar with force-based procedures but they have certain drawbacks. Structural components are evaluated in the elastic range and post-elastic behavior of structures could not be identified by an elastic analysis. However, post-elastic behavior should be considered as almost all structures are expected to deform in inelastic range during a strong earthquake. The seismic force reduction factor "R" is utilized to account for inelastic behavior indirectly at global level. These methods predict elastic capacity of structure and indicate where the first yielding will occur and don’t account for the redistribution of forces and predict failure mechanisms. Real deficiencies present in the structure could be missed. Force-based methods primarily provide life safety but they can’t provide damage limitation and easy repair.

5.3.2 Inelastic Methods of Analysis

Structures behave inelastically under a strong earthquake so inelastic analytical procedures are required for accurate analysis. Inelastic analytical procedures help to understand the actual behavior of structures by identifying failure modes and the potential for progressive collapse. Inelastic analysis procedures basically include inelastic time history analysis and inelastic static analysis (pushover analysis).

The inelastic time history analysis is the most accurate method. However, the use of inelastic time history analysis is limited because dynamic response is very sensitive to modeling and ground motion characteristics. It requires proper modeling of cyclic load deformation characteristics and availability of a set of representative ground motion records. Also the computation time for input preparation and interpreting voluminous output make the use of inelastic time history analysis impractical for seismic performance evaluation.

Inelastic static analysis, or pushover analysis, has been the preferred method for seismic performance evaluation due to its simplicity. It is a static analysis that directly incorporates nonlinear material characteristics. Inelastic static analysis procedures include Capacity Spectrum Method (ATC 40), Displacement Coefficient Method (FEMA 273) and the Secant Method

Displacement-based procedures provide a more rational approach to these issues compared to force-based procedures by considering inelastic deformations rather than elastic forces. The analytical tool for evaluation process should also be relatively simple which can capture critical response parameters that significantly affect the evaluation process.

5.4 Selection of Detailed Procedures

An analysis of the building shall be performed using one of the procedures described above keeping in view the limitations discussed below. Linear procedures are appropriate when the expected level of nonlinearity is low. This is measured by component demand to capacity ratios (DCRs) (< 2.0) calculated from linear analysis.

The procedures that explicitly recognize the nonlinear response of building components in earthquakes gives most accurate results, nonlinear analysis methods should be selected for complex or irregular buildings and for higher performance levels. For the Buildings with one or more of the following characteristics are evaluated using nonlinear procedures (ASCE 31-03).

- The fundamental period of the building, $T \geq 3.5 \times S_{D1} / S_{DS}$
- The ratio of the building's horizontal dimension at any story exceeds 1.4 times the horizontal dimension at an adjacent story (excluding penthouses & mezzanines).
- The building has a torsional stiffness irregularity in any storey. A torsional stiffness irregularity exist in any story if the diaphragm above the story under consideration is not flexible and the results of the analysis indicate that drift along any side of the structure is more than 150 percent of the average story drift.
- The building has a vertical mass or stiffness irregularity. A vertical mass or stiffness irregularity exists when the average drift in any storey exceeds that of the storey above or below by more than 150 percent.
- The lateral-force-resisting system is non-orthogonal.

Static procedures are appropriate when higher mode effects are not significant. Static analysis is applicable for short and regular buildings. Dynamic procedures are required for tall buildings, buildings with torsional irregularities, or non-orthogonal systems.

5.5 Nonlinear Dynamic Procedure

In the Nonlinear Dynamic Procedure (NDP) seismic analysis of the building, a mathematical model directly incorporating the nonlinear load deformation characteristics of individual components and elements of the building is subjected to earthquake shaking represented by ground motion time histories to obtain forces and displacements. For the NDP, earthquake shaking is characterized by discretized recorded or synthetic earthquake records.

With the NDP, the design displacements are not established using a target displacements as in Pushover analysis, but instead are determined directly through dynamic analysis using ground motion time histories. Calculated response can be highly sensitive to characteristics of individual ground motions; therefore, the analysis should be carried out with a minimum 0f three ground motion records. Because the numerical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake. Calculated demand displacements and internal forces are compared directly with the capacity.

5.6 Tier 3 Analysis of Case Study 1

As Case study 1 was found to be deficient in Tier 2 so more detailed, Tier 3 analysis is carried out. As it was identified at Tier 2 analysis that the building has considerable higher mode effects so it should be evaluated using dynamic analysis (i.e Time Histories Analysis). Time History analysis is performed for Tier3 evaluation of the case study 1.

Time History records are taken from Zameer study (MS Thesis 2010). Three time history load cases were defined as EQ1, EQ 2 and EQ3. Each load case comprised of two orthogonal horizontal components, EQ11 in x-direction and EQ 12 in Y-direction and so on. Both horizontal components of earthquake were applied simultaneously. A scaling factor of 386.4 was kept to convert from g units to inch units. Output time step was kept as 0.1 second which is approximately one tenth of time period of structure; hence response at 10 discrete time points will be captured in a single oscillation. Output steps were kept as 1.5 times duration of excitation divided by output time step .This was done so that complete time history response is captured till it damps out.

5.6.1 Results of Time History Analysis

Results of critical Basement columns are shown below. The results show that all columns have adequate shear and flexural capacity. So finally it is concluded that Case Study 1 Building is compliant with the specification of ASCE 31-03 standard.

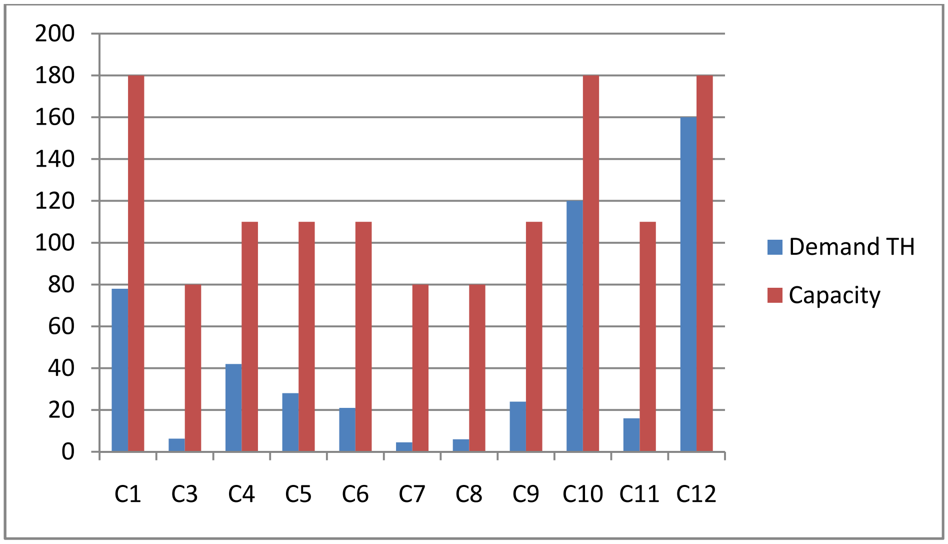

Fig 5.1 Shear V2 Demand TH and Capacity

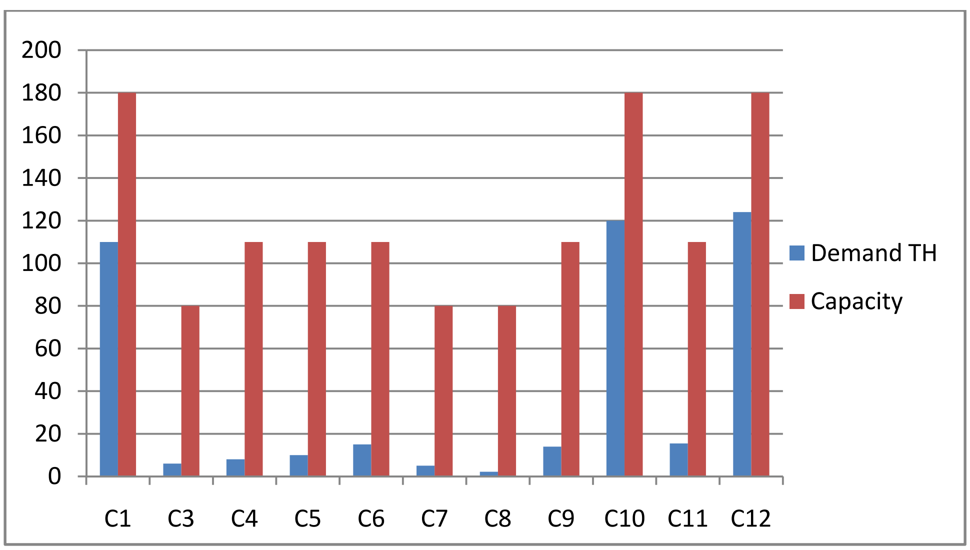

Fig 5.2 Shear V3 Demand and capacity

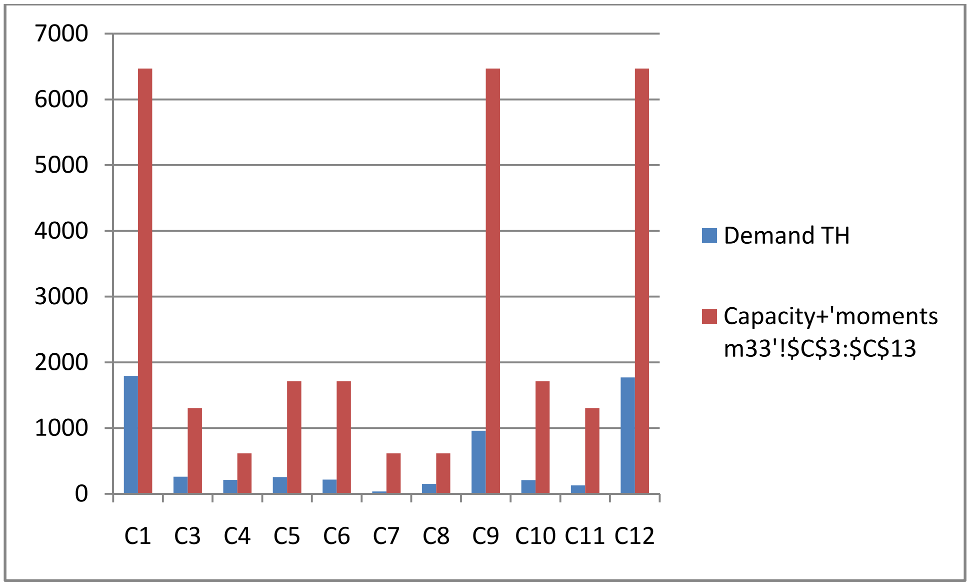

Fig 5.3 M33 Demand TH and Capacity

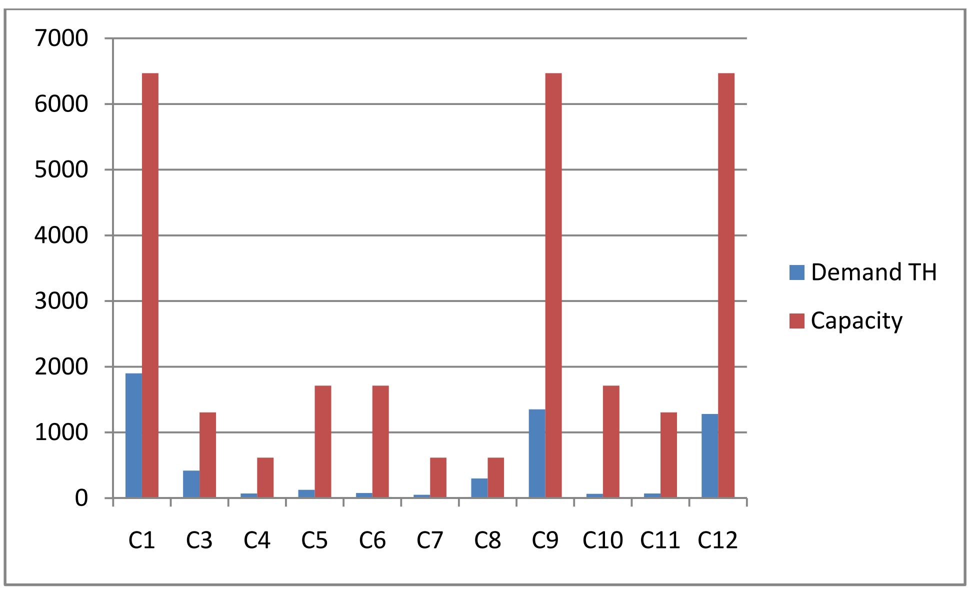

Fig 5.4 M33 Demand TH and Capacity

5.7 Description of Pushover Analysis

Pushover analysis is an approximate nonlinear analysis method in which the structure is subjected to monotonically increasing lateral loads until a target displacement is reached. A mathematical model of the building which includes load-deformation diagrams of all lateral force resisting elements is generated and gravity loads are applied initially. A predefined lateral load pattern which is distributed along the building height is then applied. The lateral forces are increased until some members yield and the structural model is modified to account for the reduced stiffness of yielded members. The lateral forces are again increased until additional members yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with base shear to get the global capacity curve. Pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines because it is conceptually and computationally simple. Pushover analysis determines the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure. Pushover analysis determines the force demand for brittle members and deformation demand for ductile members. It identifies the weak links in the structure and is therefore very useful for efficient retrofitting. Pushover analysis exposes the design weaknesses that may remain hidden in an elastic analysis. These are story mechanisms, excessive deformation demands, strength irregularities.

Pushover analysis can be performed as force-controlled or displacement controlled. Force-controlled is used when the load is known (such as gravity loading). In displacement-controlled procedure is used where the magnitude of applied load is not known in advance. The load is increased until the control displacement reaches a specified value. Generally, roof displacement at the center of mass of structure is chosen as the control displacement. The internal forces and deformations at the target displacement give inelastic strength and deformation demands which is compared with available capacities to find a performance point. In this study displacement based procedure is used for Seismic Loads and forced based procedure for gravity loads.

Available simplified conventional nonlinear static procedures are as follow.

- **Capacity Spectrum Method**, that uses intersection of capacity (pushover) curve and a reduced response spectrum in spectral coordinates (Acceleration Displacement Response Spectrum Format) to find a performance point. The specifications of this method are covered in ATC40.
- **Displacement Coefficient Method** described in FEMA-356 [20] is a non-iterative approximate procedure based on displacement modification factors. The expected maximum inelastic displacement of nonlinear MDOF system is obtained by modifying the elastic spectral displacement of an equivalent SDOF system with a series of coefficients. The procedure proposed by Newmark and Hall is based on the estimation of inelastic response spectra from elastic response spectra while displacement modification factor varies depending on the spectral region.

5.7.1 PUSHOVER ANALYSIS WITH SAP2000

Nonlinear static pushover analysis is a very powerful feature offered in the Non-linear version of SAP2000. Pushover analysis can be performed on both two and three dimensional structural models and as a force controlled or deformation controlled.

Geometric nonlinearity can be considered through P-delta effects or P-delta effects plus large displacements. In Sap 2000 Nonlinearity of the frame element is represented by specified hinges and a capacity drop occurs for a hinge when the hinge reaches a negative-sloped portion of its force-displacement curve during pushover analysis (Fig 5.5).

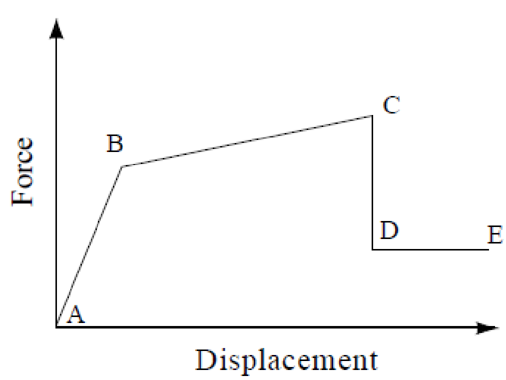

Figure 5.5 Generalized Force Displacement Characteristic of Frame element

Three different member unloading methods are provided in SAP 2000 to remove the load that the hinge was carrying and redistribute it to the rest of the structure. For "Unload Entire Structure" option, when the hinge reaches point C on its force displacement curve (Figure 5.5), try to increase the base shear. If the lateral deformation increases the analysis proceed otherwise base shear is reduced by reversing the lateral load on the whole structure until the force in that hinge is consistent with the value at point D on its force-displacement curve (Figure 5.5).As unloading completes, base shear is again increased, and other elements of the structure pick up the load that was removed from the unloaded hinge. For "Apply Local Redistribution" option, only the element in which hinge has formed is unloaded instead of unloading the entire structure. In the "Restart Using Secant Stiffness" whenever any hinge reaches point C (Fig 5.5) on force-displacement curve, all hinges that have become nonlinear are reformed using secant stiffness properties and the analysis is restarted.

In Fig 5.5, Point A corresponds to unloaded condition and point B represents yielding of the element. Point C corresponds to nominal strength and deformation at which significant strength degradation begins. The drop from C to D represents the initial failure of the element and resistance to lateral loads beyond point C is unreliable. The resistance from D to E allows the frame elements to resist gravity loads. Beyond point E, the maximum deformation capacity, gravity load can no longer be sustained. Hinges can be assigned at any location of potential yielding. Uncoupled moment (M2 and M3), torsion (T), axial force (P) and shear (V2 and V3) force-displacement relations can be defined. As the column axial load changes under lateral loading, there is also a coupled P-M2-M3 (PMM) hinge which yields based on the interaction of axial force and bending moments at the hinge location. More than one type of hinge can be assigned at the same location of a frame element.

SAP2000 considers three types of hinge properties. These are default hinge properties, user-defined hinge properties and generated hinge properties. Default hinges and user-defined hinges can be assigned to frame elements. When these hinge properties (default or user-defined) are assigned to a frame element, the program automatically creates generated hinge property. Default hinge properties could not be modified and are section dependent. The built-in default hinge properties for steel and concrete members are based on ATC-40 and FEMA-273 criteria. User-defined hinge properties can be based on default properties or they can be fully user defined.

5.7.2 Limitations of Pushover Analysis

Although pushover analysis has advantages over elastic analysis procedures, however the assumptions for pushover analysis and limitations of current pushover procedures must be identified. The estimate of target displacement, selection of lateral load patterns and identification of failure mechanisms due to higher modes of vibration are important issues that affect the accuracy of pushover results.

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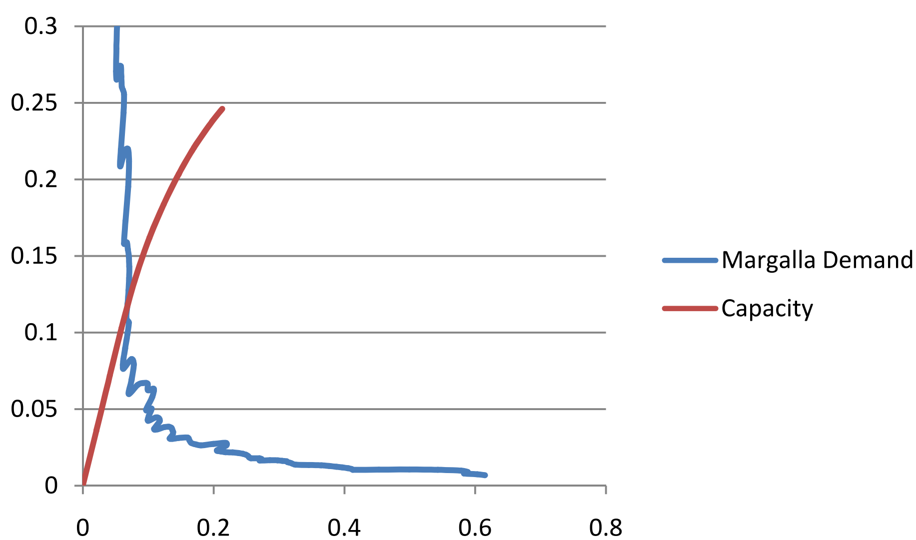


Fig 5.6 Capacity and Margalla Demand in X direction

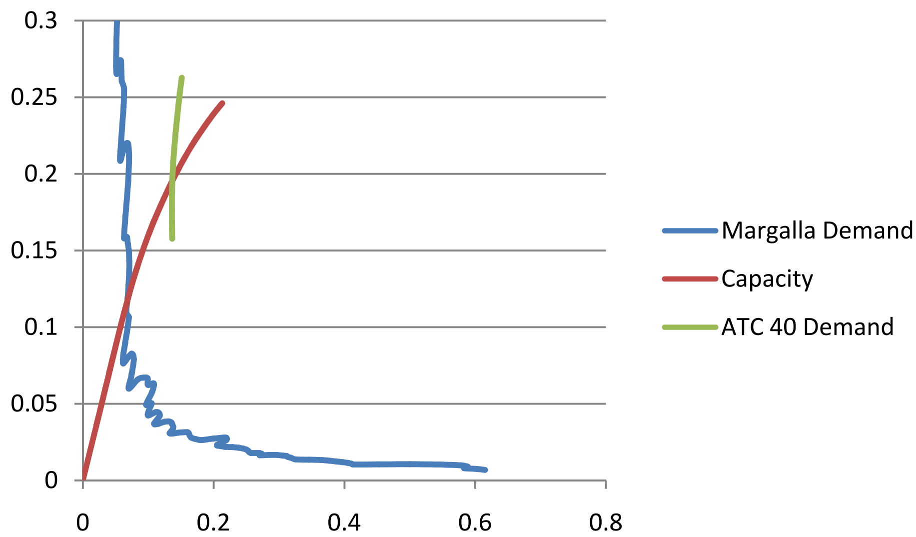


Fig 5.7 Comparison of ATC 40 & F-10 Site RS Performance Point in X direction

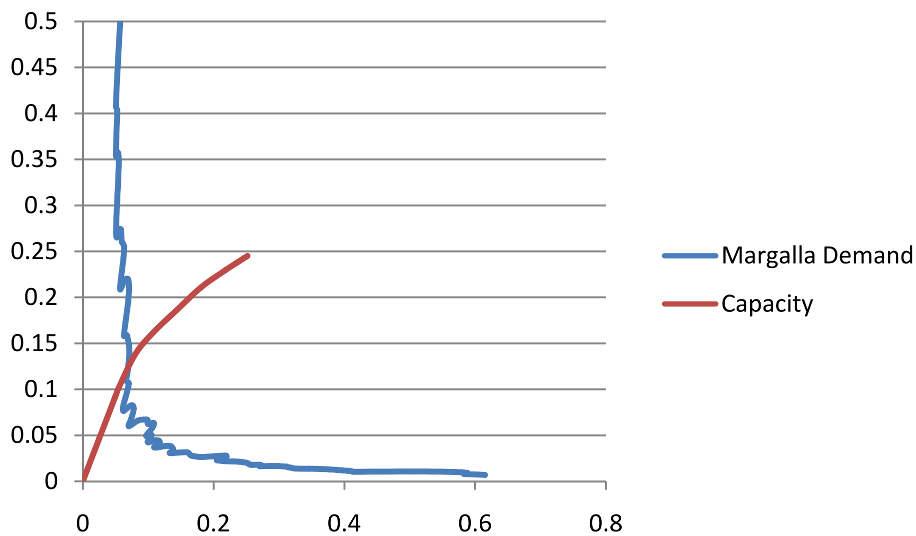


Fig 5.8 Capacity and Margalla Demand in Y direction

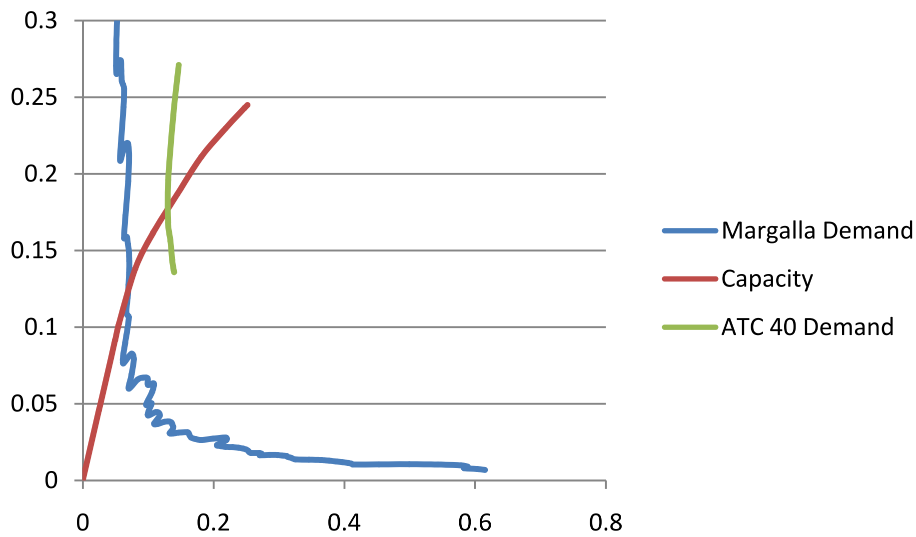
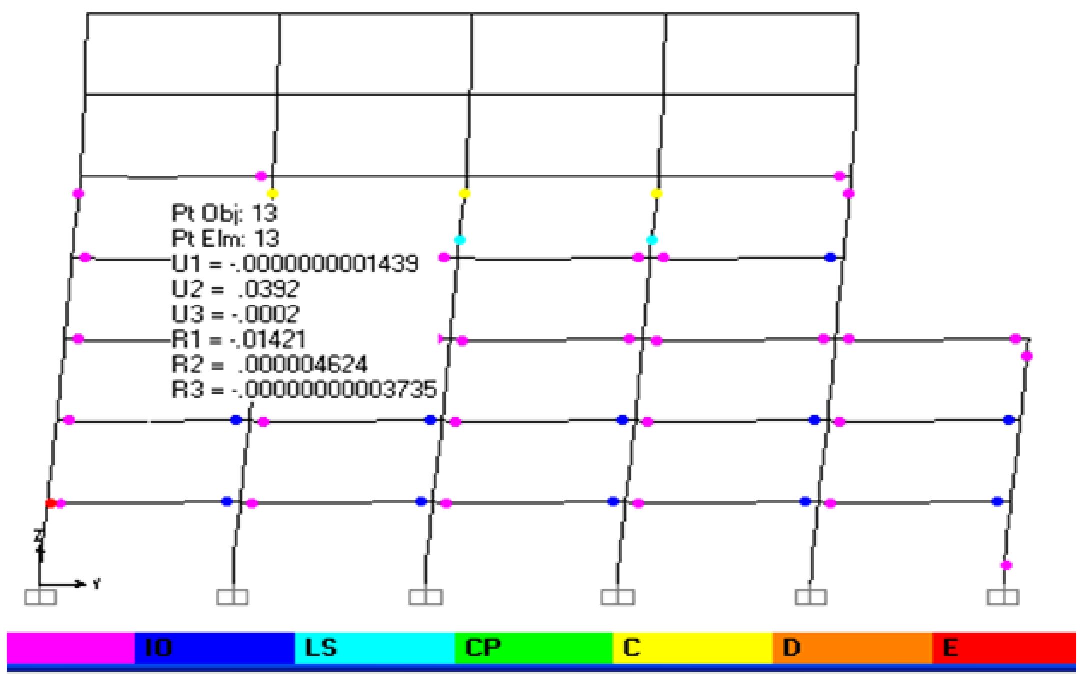


Fig 5.9 Comparison of ATC 40 & F-10 Site RS Performance point in Y direction



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Then the building under consideration is strengthened by column jacketing of the weak columns identified in 4th story and by placing steel bracing. Strengthening layout is shown in appendix A. Strengthened Column X-Sections are defined using Section Designer in Sap 2000. User Defined Hinges based on PM interaction diagram (from Sap 2000 section designer) are defined for strengthened columns and steel braces. Pushover analysis is run using same load cases and results are viewed. This time Hinges are developed in the beams, which is desirable.

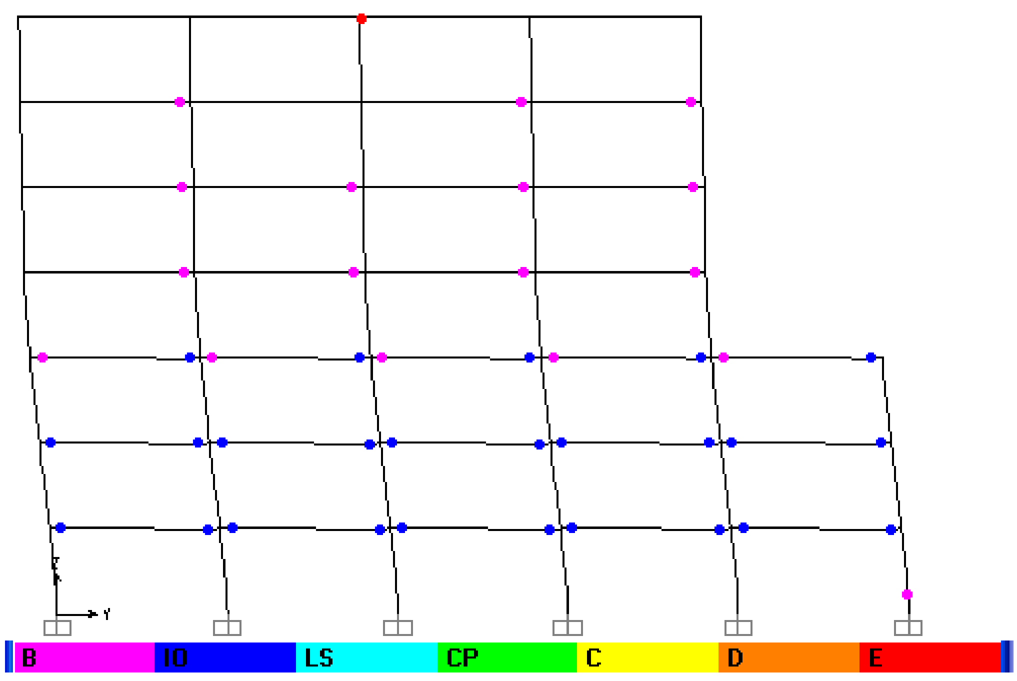


Figure 5.11 Plastic Hinge Formation of Retrofitted Structure

Chapter 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

1. Seismic analysis procedures provided by different Building Authorities differ significantly
2. ASCE 31-03 provides a generalized approach to seismic evaluation, which is thorough and provides several levels of assessment with varying degree of complexity.
3. Tier 1 analysis of ASCE 31-03 provides quantitative checks which makes it easy to use. These guidelines are most suitable for a developing country like Pakistan.
4. ASCE 31-03 uses Pseudo Lateral force which differs from Code based design procedures; it requires a higher degree of understanding on the part of design professionals.
5. In Linear Elastic Analysis ASCE 31-03, instead of using single R factor for the entire structure, different m factors are used depending on the ductility of component being evaluated. This is a more realistic approach.
6. No specification of Non-linear procedure is provided in ASCE 31-03. For these procedures it refers the use of other guidelines.
7. Pushover analysis is preferred method for nonlinear analysis because of its simplicity.
8. The display of plastic hinges mechanism for incremental load steps indicate weak links in the structure and is very useful for efficient retrofitting.
9. ATC 40 with the use BCP (2006) seismic parameters yields more conservative results

Recommendations

1. Seismic Evaluation guidelines based on ASCE 31-03 should be incorporated in building code of Pakistan.
2. For the buildings constructed before BCP (2007) Tier 1 analysis must be conducted under the supervision of Building authorities.
3. Microzonation studies to develop contours maps of spectral acceleration should be carried out throughout the country.

4. Studies to assess the adequacy of seismic analysis procedure in different guidelines should be carried.
5. Studies to simplify the adaptive and modal pushover analysis should be carried.

REFERENCES

ACI 318 (2008) Building Code Requirement for Reinforced Concrete, American Concrete Institute, Detroit, Michigan, USA

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

ASCE (2005), “Minimum Design Loads for Building and Other Structures”, SEI/ASCE 7-05, American Society of Civil Engineers, Reston, VA.

ASCE/SEI 31-03, Seismic Evaluation of Existing Buildings

BCP (2007), “Seismic Provision for Building Code of Pakistan” Ministry of Housing and Works Government of Pakistan Islamabad

Bracci J.M., Kunnath S.K. and Reinhorn A.M., 1997, Seismic Performance and Retrofit Evaluation of Reinforced Concrete Structures, Journal of Structural Engineering, ASCE, Vol. 123, 3-10.

CEN (2001), “Eurocode 8 - Design Provisions for Earthquake Resistance of Structures-Part 3”, Brussels

Clough, R.W. (1960). "The Finite Element Method in Plane Stress Analysis,"

Computers and Structures Inc. (CSI), 1998, SAP2000 Three Dimensional Static and Dynamic Finite Element Analysis and Design of Structures V7.40N, Berkeley, California.

Computers and Structures Inc CSI, (1995),” ETABS Extended Three Dimensional Analysis of Building Systems Non linear Version 9.5”, Berkeley, California USA

D'Ayla,D. and Charleson,A.W., (2002), "Review of seismic strengthening guidelines for R.C. buildings in developing countries," 12th European Conference on Earthquake Engineering, London, UK

Durgesh C Rai, Review of Documents on Seismic Evaluation of Existing Buildings

Edward L Wilson (2000); "Three Dimensional Static and Dynamic analysis of Structures" 3rd Edition, Computers and Structures, Inc. Berkeley, California, USA

ERRA (2007), "Principles, Themes and Lessons Learnt", Earthquake Reconstruction& Rehabilitation Authority Islamabad Pakistan

Fajfar P. and Gaspersic P., 1996, The N2 Method for the Seismic Damage Analysis of R/C Buildings, Earthquake Engineering and Structural Dynamics,Vol. 25, 31-46.

Federal Emergency Management Agency (FEMA), 1997, NEHRP Guidelines for the Seismic Rehabilitation of Buildings, FEMA-273

FEMA 356 (2000), Pre-standard and Commentary for the Seismic Rehabilitation of Buildings

G. LUPOI and *G.* M. CALVI, Comparison of different approaches for seismic assessment of existing buildings, Journal of Earthquake Engineering, Vol. 8, Special Issue **1** (2004) 121-160 @ Imperial College Press

IBC (2006), International Building Code IBC 2006, International Code Council, Washington, D.C.

Mwafy A.M, and Elnashai A.S, (2001). "Static pushover versus dynamic collapse analysis of RC buildings" Engineering Structures 23 (2001) 407–424

NESPAK (2006); "Revision/Updation of Building Code of Pakistan, Stage 1, Recommendations for preliminary seismic design parameters and criteria for seismic resistant design of buildings in Islamabad – Rawalpindi area" – Ministry of Housing and

Works, Government of Pakistan National Engineering Services Pakistan (Pvt.) Limited (NESPAK)

NZNSEE (2002) New-Zealand National Society for Earthquake Engineering, The Assessment and Improvement of the structural performance of Earthquake Risk Buildings, draft prepared for the HZ Building Industry Authority

Oğuz, Sermin (2005) "Evaluation of pushover analysis procedure for frame structure" Thesis

Otani, S. (2000) "Seismic vulnerability assessment **of** reinforced concrete buildings," Journal of ***Facsrlty*** of Engineering, University of Tokyo, Series E3**,** Vol XLVII, pp. 2 28.

Saiidi M and Sozen M.A. (1981) "Simple nonlinear seismic analysis of R/C structures", Journal of the Structural Division, ASCE 1981; 107(ST5):937–51.

Tapal K Sen, (2009), "Fundamental of Seismic Loading on Structures" Jhon Willey and son Limited.

UBC (1997), "Uniform Building Code 1997", International Conference of Building Officials, Whittier, California, USA.

W.F Chen, E.M Lui (2006) Earthquake Engineering For Structural Design

APPENDIX I

DESCRIPTION OF CASE STUDY BUILDINGS

A.1 CASE STUDY 1

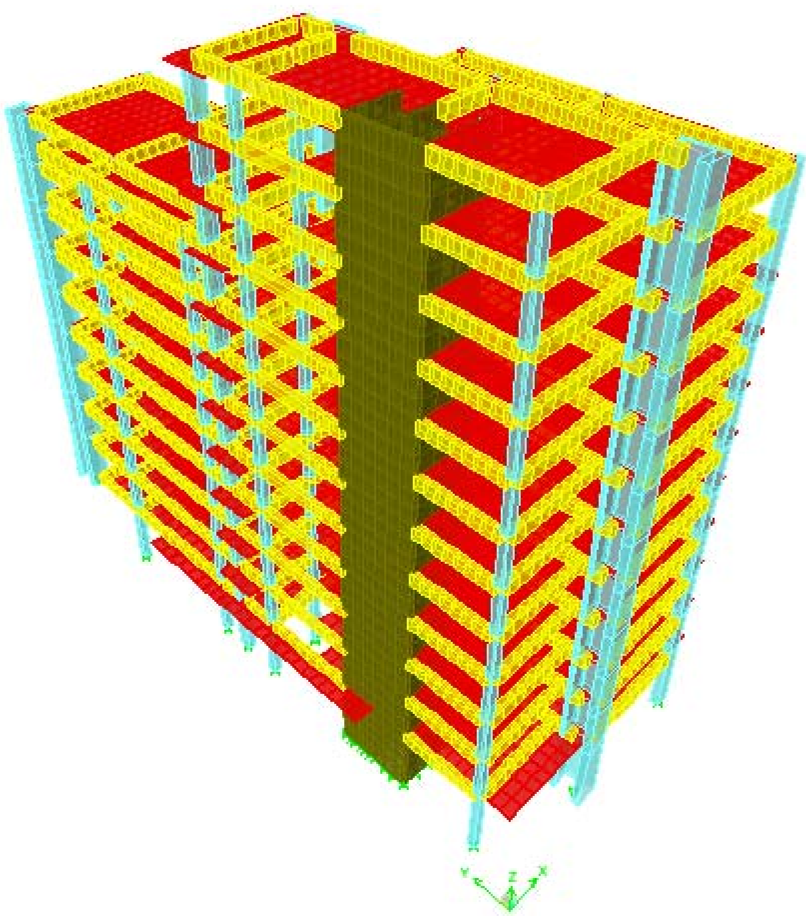
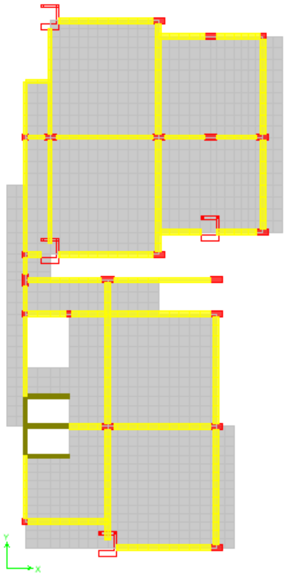


Figure A. 1. 3D Model Case Study 1



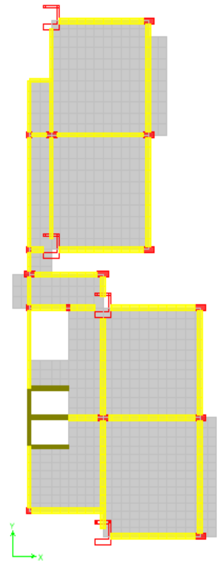


Figure A. 2 Basement Plane

Figure A.3 4th to 7th Floor Plane

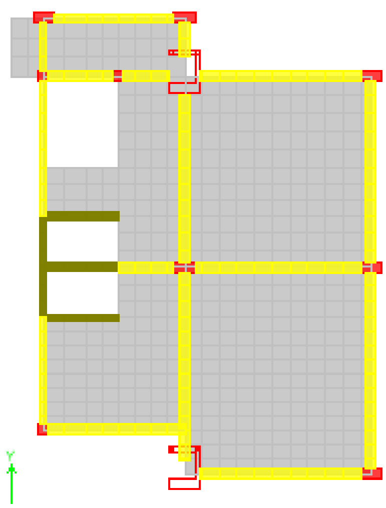


Figure A.4 8th &9th Floor Plane

Table A.1 Member Section Properties of Case Study 1

Section	Depth in	Width in	Area in^2	I_{22} in^4	I_{33} in^4	J in^4	Type
B1	24	12	288	3456	13824	9492	Beam
B2	30	12	360	4320	27000	12935	Beam
B3	24	18	432	11664	20736	25192	Beam
B4	30	18	540	14580	40500	36513	Beam
C1	C shaped		1890	1389989	525921	112396	Column
C2 ,C3,C4	36	18	648	17496	69984	48054	Column
C5,C6	18	18	324	8748	8748	14784	Column
C7	24	24	576	27648	27648	46725	Column

Slab and shear wall were modeled as RCC shell elements. Concrete compressive strength (f'_c) was taken as 3000 psi (pounds per square inch) and yield strength (f_y) for steel was taken as 60,000 psi. Following assumptions were made:-

a. Beam column joints were modeled as rigid joints.
b. The system was assumed to be linearly elastic and floors and beams were assumed to be rigid, which will over estimate the stiffness of the building. Considering the general trend of the acceleration response spectrum; this will decrease the fundamental period which in turn will result in larger earthquake induced forces. Hence the assumption of floor and beam rigidity can be thought as a safe engineering assumption.
c. For calculation of mass dead load and 25 % of live load was accounted. This mass as lumped at each story level.
d. Cracked section properties were incorporated as per ACI 10.10.4.1; for beams I_{eff} = 0.5 I_g which also includes contribution of slabs and for columns I_{eff} = 0.7 I_g..

A.2 CASE STUDY 2

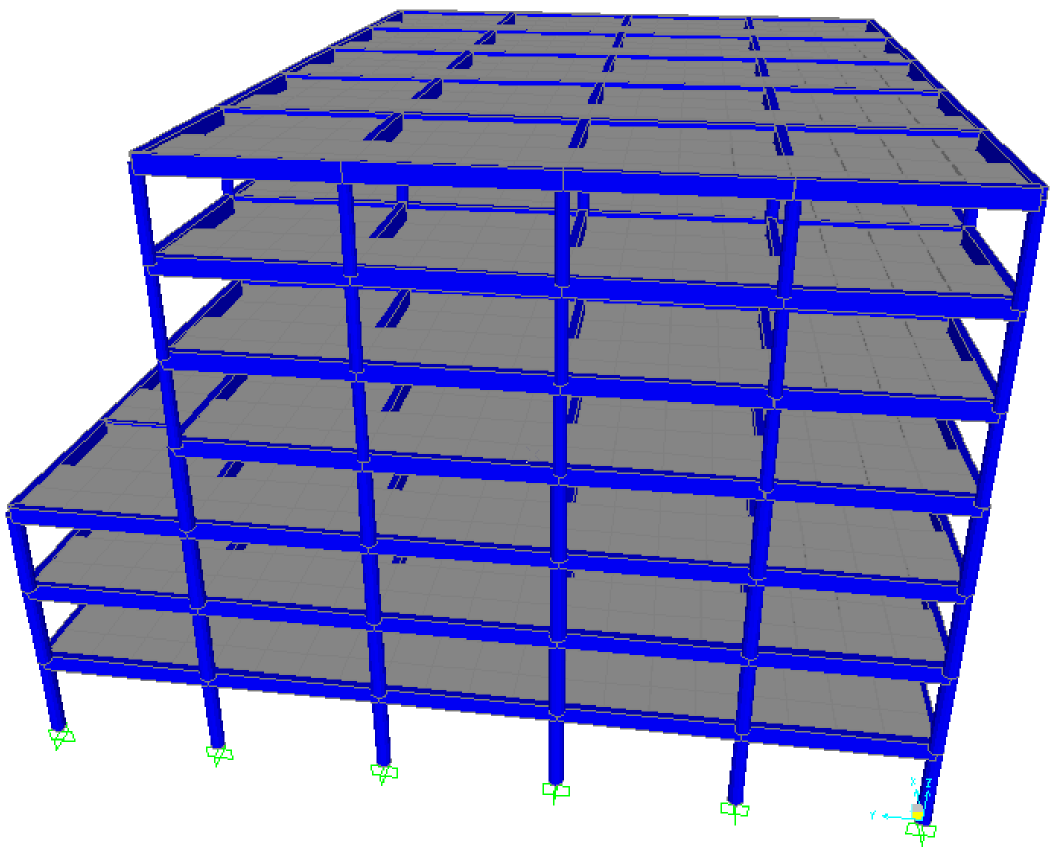


Figure A.5 3D model Case Study 2

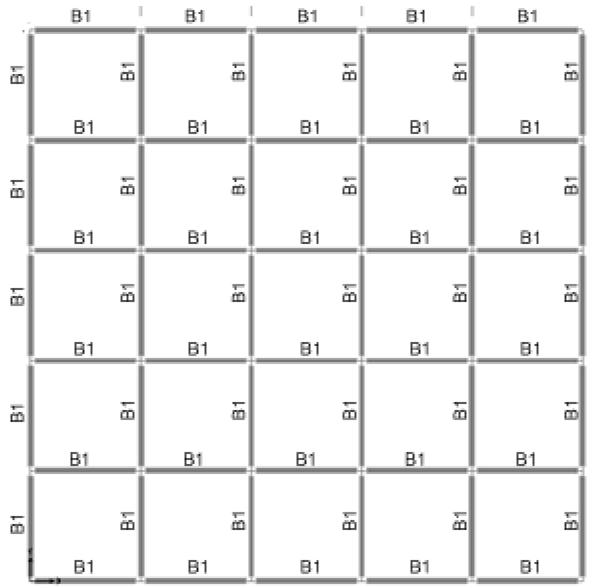
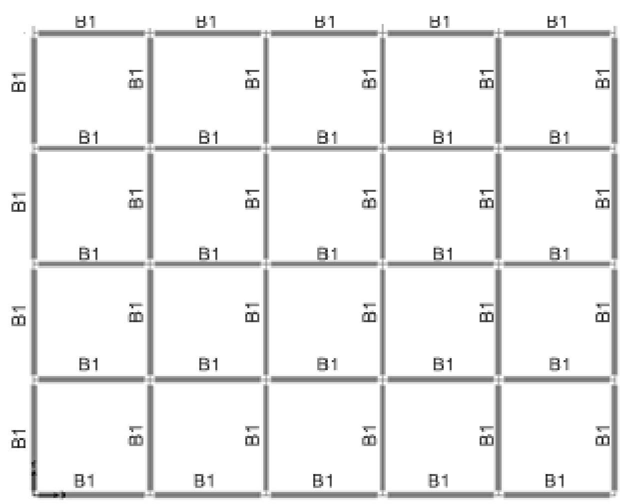


Figure A.6 Plan View Basement to storey 3



FigureA.7Plan View Storey 4 to Storey 7

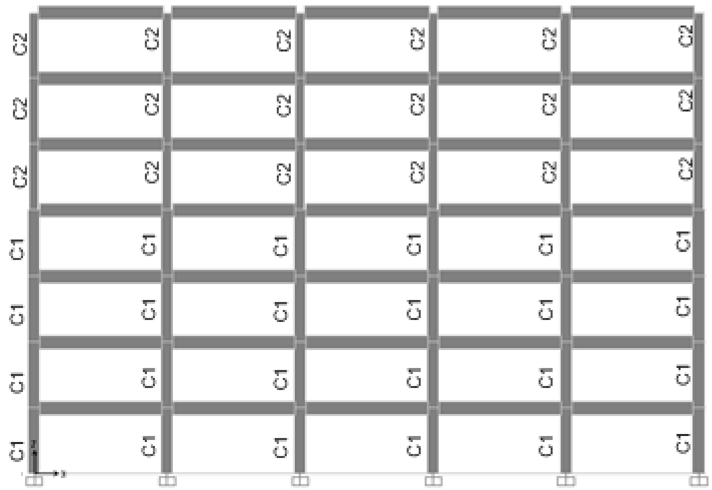


Figure A.8 Elevation in XZ plane

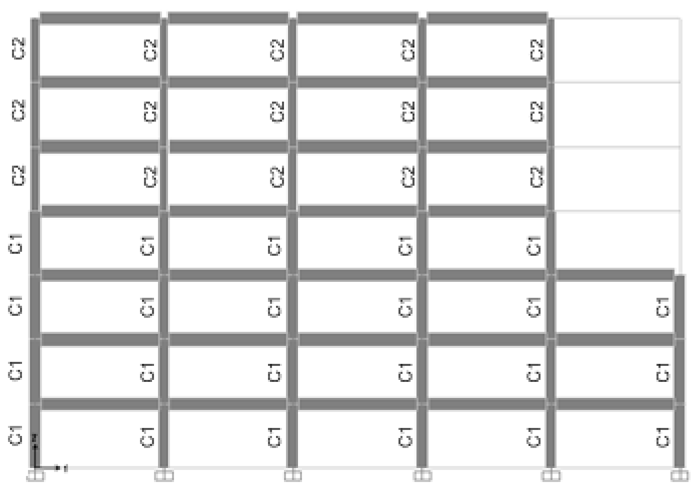


Figure A.9 Elevation in YZ Plane

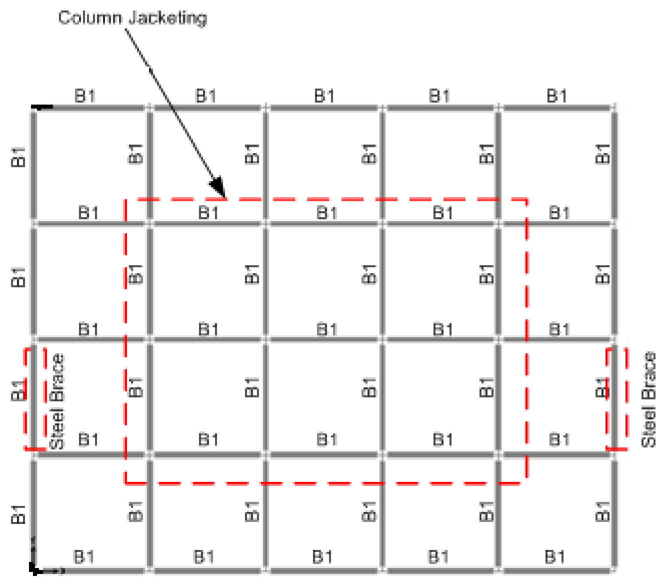


Figure A.10 Strengthening Plane of Storey 4

Table A.2 Members Section Properties of Case Study 2

Member	Dimension	Reinforcement
Beam – B1 (width x height)	12” x 24”	Top 5 # 6, Bot 5-#6 (Stirrup #3@6”)
Column – C1	18” x 18”	8 # 8 bars (Stirrups #3@6”)
Column – C2	15” x 15”	8#8 bars (Strirrups # 3@6”
Slab	Thickness = 6 “	
Steel Plate (Column Jacketing)	Thickness = 0.5”	
H Section	8” x 8” x 3/8”	

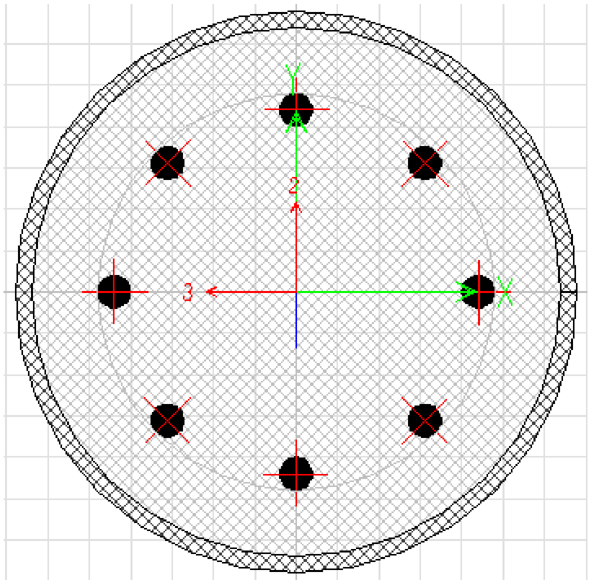
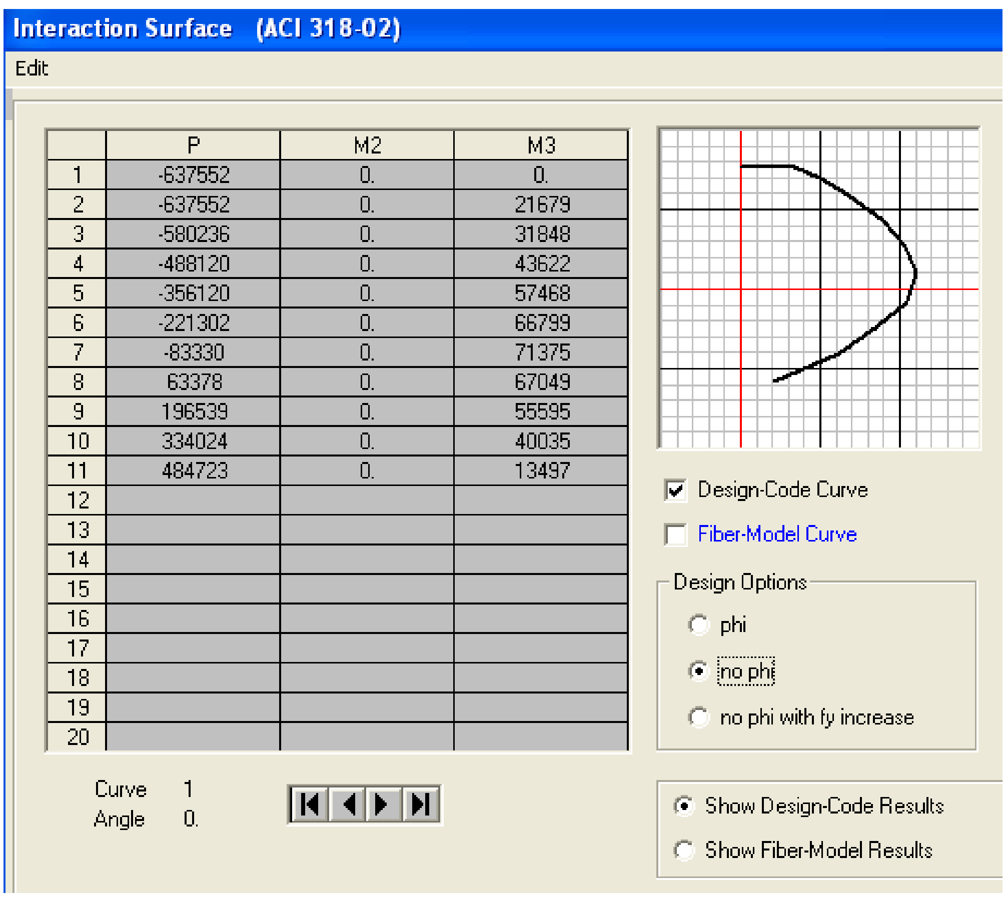


Figure A.11 Strengthened Column Section



	P	M2	M3
1	-637552	0.	0.
2	-637552	0.	21679
3	-580236	0.	31848
4	-488120	0.	43622
5	-356120	0.	57468
6	-221302	0.	66799
7	-83330	0.	71375
8	63378	0.	67049
9	196539	0.	55595
10	334024	0.	40035
11	484723	0.	13497
12			
13			
14			
15			
16			
17			
18			
19			
20			

Figure A.12 PM Interaction Diagram of Strengthened Column

APPENDIX II

Tier 1 Calculation

B.1 Story Masses

(a) Basement

Self Wt. of slabs = 0.5x 0.15 x 1066 + 0.66 x 0.15 x 5310 = 611 kips

D.L + 0.25 L.L = 0.055 x6376 = 351 kips

Load of Beams with 1 k/ft load of walls = 1040 kips

Wt. of Columns and shear walls = 275 kips

Total =2277 kips

(b) Ground and 1st Floors

Self Wt. of slabs = 0.5 x 0.15 x 1066 + 0.66 x 0.15 x 4815 = 557 kips

D.L + 0.25 L.L = 0.055 x5881 =325 kips

Load of Beams with 1 k/ft load of walls = 1020kips

Wt. of Columns and shear walls = 275 kips

Total =2177 kips

(c) 2nd and 3rd Floors

Self Wt. of slabs = 0.5 x 0.15 x 1066 + 0.66 x 0.15 x 4100 = 486kips

D.L + 0.25 L.L = 0.055 x5566 =284 kips

Load of Beams with 1 k/ft load of walls = 1020kips

Wt. of Columns and shear walls = 252kips

Total =2042 kips

(d) 4^{th}, 5^{th} & 6^{th} Floors

Self Wt. of slabs = 0.5 x 0.15 x 1066 + 0.66 x 0.15 x 3483 = 425kips

D.L + 0.25 L.L = 0.055 x4549 =251 kips

Load of Beams with 1 k/ft load of walls = 900kips

Wt. of Columns and shear walls = 252kips

Total =1828 kips

(e) 7^{th} Floor

Self Wt. of slabs = 0.5 x 0.15 x 1066 + 0.66 x 0.15 x 3373= 414 kips

D.L + 0.25 L.L = 0.055 x4439 =244 kips

Load of Beams with 1 k/ft load of walls = 900kips

Wt. of Columns and shear walls = 187kips

Total =1745 kips

(f) 8^{th} & 9^{th} Floor

Self Wt. of slabs = 0.5 x 0.15 x 1066 + 0.66 x 0.15 x1600 = 239 kips

D.L + 0.25 L.L = 0.055 x2666 =147 kips

Load of Beams with 1 k/ft load of walls = 460kips

Wt. of Columns and shear walls = 167kips

Total =1013 kips

(g) Roof

Self Wt. of slabs = 0.5 x 0.15 x 1250 = 95kips

D.L + 0.25 L.L = 0.06 x1250 =75 kips

Load of Beams with 1 k/ft load of walls = 250kips

Total =420 kips

Total Wt. of all floors = 2277+2x2177+2x2042+3x1828+1745+1013x2+420

Total = 20,390 Kips

B.2 Pseudo Lateral Force

$V = CS_aW$

C= 1 (From Table 3.4)

$S_s = 0.8g$, $S_1 = 0.15g$ (Zameer)

Soil type = Class D

Using Equations 3-4 to 3-6 and Fv & Fa values from table 3-5 & 3-6,

$F_v = 2.1$

$F_a = 1.18$

$S_{Dl} = 2/3x\ 2.1x0.15 = 0.21$

$S_{DS} = 2/3x\ 1.18x0.8 = 0.63$

Time Period, T= 0.1n = 1 second

$S_a = 0.08/1 = 0.08 < S_{DS}$

V = 1x 0.21 x 20390 = 4282 kips

B.3 Story Shear Forces

$$F_x = (w_x h_x^k / \sum w_i h_i^k) \text{ x } V$$

K = 1.25 (for T= 1 second)

Table B.1 Story Forces

Level	w_x (kips)	h_x (ft)	h_x^k (ft)	$w_x h_x^k$	$w_x h_x^k / \sum w_t h_t$	F (kips)	V (kips)
Roof	420	125	417.96	175543.2	0.0495	212	212
9th	1013	115	376.59	381485.7	0.1076	460.75	672.75
8th	1013	105	336.11	340479.4	0.0960	411	1083.8
7th	1745	95	296.59	517549.6	0.1460	625.2	1709
6th	1828	85	258.09	471788.5	0.1330	569.5	2278.5
5th	1828	75	220.71	403457.9	0.1138	487.3	2765.8
4th	1828	65	184.56	337375.7	0.0951	407.2	3133
3rd	2042	55	149.78	305850.8	0.0863	369.5	3542.5
2nd	2042	45	116.55	237995.1	0.0671	287.3	3829.8
1st	2177	35	85.13	185328.0	0.0523	224	4053.8
G.F	2177	25	55.90	121694.3	0.0343	147.9	4201.7
Basement	2277	15	29.52	67216.7	0.0190	81.4	4282.1
∑	20390	-	-	3545765	-	4282.1	-

B.4 Shear Stress in Concrete Frame Columns

X-Direction

a. Basement , GF & 1st

$V_{avg} = 1/m\ [n_c / (n_c - n_f)]\ (V_j/A_c)$

$= 1/2x\ [26/(26-4)]x(4282x1000)/(137.6x144)$

$= 128$ psi > 100psi **NC**

b. 2nd &3rd Floors

$V_{avg} = 1/2x\ [22/(22-4)]x(3829.8x1000)/(130.875x144)$

$= 124$ psi >100psi

c. 4^{th}, 5^{th}, 6^{th} & 7^{th}

$$V_{avg} = 1/2x\ [19/(19-4)]x(3133x1000)/(106.75\ x144)$$

$$=129\ psi > 100psi \quad \textbf{NC}$$

d. 9^{th} & 8^{th}

$$V_{avg} = 1/2x\ [11/(11-4)]x(1083.8x1000)/(60.25x144)$$

$$= 98.5\ psi < 100psi \quad \textbf{O.K}$$

e. Roof

$$V_{avg} = 1/2x\ [8/(8-3)]\ x\ (212x1000)/(33x144)$$

$$= 36\ psi < 100psi \quad \textbf{O.K}$$

<u>Y- Direction</u>

a. Basement , GF & 1st

$$V_{avg} = 1/m\ [n_c/(n_c-n_f)]\ (V_j/A_c) = 1/2x\ [26/(26-6)]x(4282x1000)/(137.6x144)$$

$$=140.5\ psi > 100psi \quad \textbf{NC}$$

b. 2^{nd} & 3^{rd} Floors

$$V_{avg} = 1/2x\ [22/(22-6)]x(3829x1000)/(130.875x144)$$

$$= 139.7\ psi > 100psi \quad \textbf{NC}$$

c. 4^{th}, 5^{th}, 6^{th} & 7^{th}

$$V_{avg} = 1/2x\ [19/(19-5)]x(3133x1000)/(106.75\ x144)$$

$$=138.3\ psi > 100psi \quad \textbf{NC}$$

d. 9th & 8th

V_{avg} =1/2x [11/(11-4)]x(1083.8x1000)/(60.25x144)

=98.2 psi < 100psi **O.K**

e. Roof

V_{avg} =1/2x [8/(8-3)]x(212x1000)/(33x144)

= 57.11psi < 100psi **O.K**

B.5 Stress in Shear Wall

$$V_{avg} = 1/m\ (Vj/Aw)$$

Basement X-Direction:

V_{avg} = 1/4 x (4282x1000/2160)

= 495 psi > 100 psi **NC**

Basement Y-Direction

V_{avg} = 1/4 x (4282x1000/4752)

= 225 psi >100 psi **NC**

B.6 Axial Stress Due to Overturning

$P_{ot} = 1/m\ (2/3)\ (Vh_n/\ Ln_f)\ x\ (1/A_{col})$

= 1/2x2/3 (4282x1000x125x12)/(16x700)x (1/648)

= 295 psi < 900 psi **O.K**

B.7 Center of Mass & Rigidity

Table B.2 Centre of Mass & Rigidity

Level	CMX	CMY	CRX	CRY	%age X	%age Y
ROOF	188.718	521.300	136.254	602.117	8%	5%
9TH	309.196	474.348	143.994	633.087	23%	10%
8TH	326.307	469.048	143.340	659.918	26%	12%
7TH	318.838	608.212	137.945	675.715	27%	4.5%
6TH	315.372	672.445	134.087	678.117	28%	0.01%
5TH	313.570	705.824	129.808	677.247	29%	2%
4TH	312.467	726.276	125.379	674.058	30%	3%
3RD	317.446	751.393	120.679	665.560	32%	6%
2ND	321.552	769.881	117.019	629.647	34%	9%
GF	337.045	804.897	118.269	602.591	35%	13%
BM	341.542	816.280	125.621	580.894	33%	16%

APPENDIX III

ANALYSIS RESULTS

C.1 Linear Static Analysis

ETABS v9.5.0 File:MARGALLA ASCE 31-03. Units: Kip-in

PROJECT INFORMATION, Company Name = NUST

S T A T I C   L O A D   C A S E S

Static Load Case	Case Type	Auto Lat Load	Self Wt. Multiplier
DEAD	DEAD	N/A	1.0000
LIVE	LIVE	N/A	0.0000
EQX	QUAKE	NEHRP97	0.0000
EQY	QUAKE	NEHRP97	0.0000

A U T O   S E I S M I C   N E H R P 9 7

Case: EQX

AUTO SEISMIC INPUT DATA

Direction: X

Typical Eccentricity = 5%, Eccentricity Overrides: No

Period Calculation: Program Calculated, Ct = 0.03 (in feet units)

R = 1

Seismic Group Type = I

Site Class = D

I = 1

Ss = 0.83g

S1 = 0.15g

Fa = 1.168

Fv = 2.2

hn = 1500.000 (Building Height)

AUTO SEISMIC CALCULATION FORMULAS

Sds = 2 Fa Ss / 3

Sd1 = 2 Fv S1 / 3

Ta = Ct (hn^(3/4))

Cu is linearly interpolated from NEHRP97 Table 5.3.3

If Tetabs <= Cu * Ta then T = Tetabs, else T = Cu * Ta

Cs = Sds I / R (Eqn. 1)

Cs <= Sd1 I / (R T) (Eqn. 2)

Cs >= 0.1 Sd1 I (Eqn. 3)

V = Cs W

k = exponent applied to story height when distributing shear over the building height

If T <= 0.5 sec, then k = 1

If 0.5 < T < 2.5 sec, then k = 0.5T + 0.75

If T >= 2.5 sec, then k = 2

AUTO SEISMIC CALCULATION RESULTS

Sds = 0.6463g

Sd1 = 0.2200g

The Seismic Design Category is determined based on worst case of NEHRP97 Tables 4.2.1a and 4.2.1b

Seismic Design Category = D

Ta = 1.1215 sec

Cu = 1.3800

T Used = 1.4585 sec

Cs (Eqn 1) = 0.6463

Cs (Eqn 2) = 0.1508

Cs (Eqn 3) = 0.0220

Cs Used = 0.1508

W Used = 22554.33

V Used = 0.1508W = 3402.08

K Used = 1.4793

AUTO SEISMIC STORY FORCES

STORY	FX	FY&FZ	MZ
ROOF	127.67	0.00	3132.066
9TH	395.40	0.00	596.998
8TH	367.31	0.00	510.442
7TH	530.39	0.00	4507.692
6TH	472.84	0.00	4095.284
5TH	392.92	0.00	3403.104
4TH	317.96	0.00	2753.866
3RD	280.76	0.00	2385.555
2ND	211.59	0.00	1795.248
1ST	158.17	0.00	1555.502
GF	97.06	0.00	975.789
Basement.	50.00	0.00	309.168

A U T O   S E I S M I C   N E H R P 9 7

Case: EQY

AUTO SEISMIC INPUT DATA

Direction: Y

Typical Eccentricity = 5%, Eccentricity Overrides: No

Period Calculation: Program Calculated

Ct = 0.03 (in feet units

R = 1

Seismic Group Type = I

Site Class = D

I, the occupancy importance factor, is from NEHRP97 Table 1.4

I = 1

Ss = 0.831g

S1 = 0.15g

Fa = 1.1676

Fv = 2.2

hn = 1500.000 (Building Height)

AUTO SEISMIC CALCULATION FORMULAS

Sds = 2 Fa Ss / 3

Sd1 = 2 Fv S1 / 3

Ta = Ct (hn^(3/4))

Cu is linearly interpolated from NEHRP97 Table 5.3.3

If Tetabs <= Cu * Ta then T = Tetabs, else T = Cu * Ta

Cs = Sds I / R (Eqn. 1)

Cs <= Sd1 I / (R T) (Eqn. 2)

Cs >= 0.1 Sd1 I (Eqn. 3)

V = Cs W

k = exponent applied to story height when distributing shear over the building height

If T <= 0.5 sec, then k = 1

If 0.5 < T < 2.5 sec, then k = 0.5T + 0.75

If T >= 2.5 sec, then k = 2

AUTO SEISMIC CALCULATION RESULTS

Sds = 0.6469g

Sd1 = 0.2200g

The Seismic Design Category is determined based on worst case of NEHRP97 Tables 4.2.1a and 4.2.1b

Seismic Design Category = D

Ta = 1.1215 sec

Cu = 1.3800

T Used = 1.5477 sec

Cs (Eqn 1) = 0.6469

Cs (Eqn 2) = 0.1421

Cs (Eqn 3) = 0.0220

Cs Used = 0.1421

W Used = 22554.33

V Used = 0.1421W = 3206.05

K Used = 1.5238

AUTO SEISMIC STORY FORCES

STORY	FX	FY	MZ
ROOF	0.00	123.07	-412.860
9TH	0.00	379.75	-4134.073
8TH	0.00	351.34	-5009.749
7TH	0.00	505.07	-2273.938
5TH	0.00	370.25	-1879.084
4TH	0.00	297.71	-1510.924
3RD	0.00	260.93	-1312.915
2ND	0.00	194.89	-982.188
1ST	0.00	144.06	-987.384
GF	0.00	87.09	-642.325
Basement	0.00	43.85	-60.653

DIAPHRAGM MASS DATA

Story	Mass-X	Mass-Y	MMI	X-M	Y-M
Roof	6.969E-01	6.969E-01	4.344E+04	188.718	521.300
9TH	2.635E+00	2.635E+00	2.601E+05	341.065	461.929
8TH	2.769E+00	2.769E+00	2.813E+05	346.893	462.671
7TH	4.779E+00	4.779E+00	1.198E+06	309.303	785.885
6TH	5.028E+00	5.028E+00	1.288E+06	307.871	811.432
5TH	5.028E+00	5.028E+00	1.288E+06	307.871	811.432
4TH	5.028E+00	5.028E+00	1.288E+06	307.871	811.432
3RD	5.697E+00	5.697E+00	1.487E+06	340.137	865.854
2ND	5.779E+00	5.779E+00	1.510E+06	344.043	871.165
1ST	6.249E+00	6.249E+00	1.705E+06	381.118	907.463
GF	6.300E+00	6.300E+00	1.725E+06	385.409	911.266
Bsmnt	6.937E+00	6.937E+00	1.890E+06	373.941	898.298

CENTERS OF CUMULATIVE MASS & CENTERS OF RIGIDITY

Story		Center of Mass------/		Center of Rigidity--/	
Level	Mass	Ordinate-X	Ordinate-Y	Ordinate-X	Ordinate-Y
ROOF	6.969E-01	188.718	521.300	136.254	602.117
9TH	3.332E+00	309.196	474.348	143.994	633.087
8TH	6.101E+00	326.307	469.048	143.340	659.918
7TH	1.088E+01	318.838	608.212	137.945	675.715

6TH	1.591E+01	315.372	672.445	134.087	678.117
5TH	2.094E+01	313.570	705.824	129.808	677.247
4TH	2.596E+01	312.467	726.276	125.379	674.058
3RD	3.166E+01	317.446	751.393	120.679	665.560
2ND	3.744E+01	321.552	769.881	117.019	629.647
GF	4.999E+01	337.045	804.897	118.269	602.591
Bsmnt.	5.692E+01	341.542	816.280	125.621	580.894

M O D A L P E R I O D S A N D F R E Q U E N C I E S

Mode No.	Period (Time)	Frequency (Cycles/Time)	Circulat Frequency (Radians/time)
Mode 1	1.68764	0.59254	3.72306
Mode 2	1.45851	0.68563	4.30796
Mode 3	1.17198	0.85326	5.36118
Mode 4	0.59641	1.67671	10.53508
Mode 5	0.42044	2.37849	14.94447
Mode 6	0.39930	2.50437	15.73543
Mode 7	0.33846	2.95458	18.56415
Mode 8	0.29606	3.37765	21.22242
Mode 9	0.23948	4.17574	26.23697
Mode 10	0.23020	4.34411	27.29483
Mode 11	0.17962	5.56739	34.98093
Mode 12	0.15058	6.64101	41.72667

M O D A L P A R T I C I P A T I N G M A S S R A T I O S

Mode No.	X-Trans <Mass <sum>	Y-Trans Mass <sum>	RX-Rotn Mass<sum>	RY-Rotn Mass <sum>	RZ-Rotn Mass <sum>
Mode 1	1.11 < 1>	37.47 <37>	51.39 <51>	0.31 < 0>	30.57 <31>
Mode 2	68.91<70>	0.06 <38>	0.12 <52>	98.60<99>	1.51 <32>
Mode 3	0.51 <71>	30.94 <68>	47.15<99>	0.00 <99>	38.67 <71>
Mode 4	2.96 <73>	7.44 <76>	0.45 <99>	0.02 <99>	7.70 <78>
Mode 5	0.01 <73>	0.04 <76>	0.00 <99>	0.00 <99>	0.01 <78>
Mode 6	14.45<88>	3.46 <79>	0.13 <99>	0.68 <100>	0.01 <78>
Mode 7	0.24 <88>	0.05 <79>	0.01 <99>	0.02 <100>	9.28 <88>
Mode 8	0.80 <89>	12.02 <91>	0.58 <100>	0.05 <100>	2.40 <90>
Mode 9	0.00 <89>	0.06 <92>	0.00 <100>	0.00 <100>	2.48 <93>
Mode 11	3.79 <94>	1.88 <94>	0.06 <100>	0.19 <100>	0.47 <93>
Mode 12	0.74 <95>	0.61 <94>	0.01 <100>	0.01 <100>	0.54 <94>

S T O R Y F O R C E S

Story	Load	VX	VY	T	MX	MY
Roof	EQX	-1.277E+02	8.624E-10	6.342E+04	8.256E-08	-1.532E+04
9TH	EQX	-5.231E+02	9.996E-10	2.455E+05	5.684E-07	-7.809E+04
8TH	EQX	-8.904E+02	1.094E-09	4.159E+05	7.491E-07	-1.849E+05
7TH	EQX	-1.421E+03	6.022E-10	8.282E+05	8.842E-07	-3.554E+05
6TH	EQX	-1.894E+03	1.724E-10	1.208E+06	1.222E-06	-5.827E+05
5TH	EQX	-2.287E+03	-2.000E-10	1.523E+06	1.729E-06	-8.570E+05
4TH	EQX	-2.604E+03	-4.910E-10	1.779E+06	2.425E-06	-1.170E+06
3RD	EQX	-2.885E+03	-8.237E-10	2.019E+06	2.613E-06	-1.516E+06

2ND	EQX	-3.097E+03	-1.096E-09	2.202E+06	3.121E-06	-1.887E+06
1ST	EQX	-3.255E+03	-1.407E-09	2.344E+06	3.445E-06	-2.278E+06
GF	EQX	-3.352E+03	-1.641E-09	2.431E+06	3.779E-06	-2.680E+06
Bsm	EQX	-3.402E+03	-1.707E-09	2.476E+06	4.206E-06	-3.293E+06
Roof	EQY	1.527E-09	-1.231E+02	-2.281E+04	1.477E+04	2.046E-07
9TH	EQY	1.870E-09	-5.028E+02	-1.482E+05	7.511E+04	4.482E-07
8TH	EQY	3.183E-09	-8.542E+02	-2.651E+05	1.776E+05	8.635E-07
7TH	EQY	2.211E-09	-1.359E+03	-4.186E+05	3.407E+05	1.139E-06
6TH	EQY	1.030E-09	-1.807E+03	-5.542E+05	5.576E+05	1.296E-06
5TH	EQY	1.488E-10	-2.178E+03	-6.664E+05	8.189E+05	1.330E-06
4TH	EQY	6.852E-11	-2.475E+03	-7.565E+05	1.116E+06	1.385E-06
3RD	EQY	-2.307E-10	-2.736E+03	-8.439E+05	1.444E+06	1.357E-06
2ND	EQY	-6.840E-10	-2.931E+03	-9.100E+05	1.796E+06	1.295E-06
1ST	EQY	-1.211E-09	-3.075E+03	-9.639E+05	2.165E+06	1.171E-06
GF	EQY	-1.172E-09	-3.162E+03	-9.968E+05	2.544E+06	1.042E-06
Bsm	EQY	-1.216E-09	-3.206E+03	-1.013E+06	3.122E+06	8.315E-07

STORY DRIFTS

Story	Dir	Load	Max Drift
Roof	X	EQX	1/166
9TH	X	EQX	1/166
8TH	X	EQX	1/158
7TH	X	EQX	1/151
6TH	X	EQX	1/150
5TH	X	EQX	1/151
4TH	X	EQX	1/153
3RD	X	EQX	1/158

2ND	X	EQX	1/173
1ST	X	EQX	1/202
GF	X	EQX	1/234
Bsmnt	X	EQX	1/378
Roof	Y	EQY	1/212
9TH	Y	EQY	1/174
8TH	Y	EQY	1/152
7TH	X	EQY	1/281
7TH	Y	EQY	1/141
6TH	X	EQY	1/245
6TH	Y	EQY	1/132
5TH	X	EQY	1/225
5TH	Y	EQY	1/127
4TH	X	EQY	1/229
4TH	Y	EQY	1/129
3RD	X	EQY	1/267
3RD	Y	EQY	1/140
2ND	X	EQY	1/281
2ND	Y	EQY	1/149
1ST	Y	EQY	1/158
GF	X	EQY	1/360
GF	Y	EQY	1/190
Bsmnt	X	EQY	1/591
Bsmnt	Y	EQY	1/353

STORY MAXIMUM AND AVERAGE LATERAL DISPLACEMENTS

Story	Load	Dir	Max.	Avg.	Ratio
Roof	EQX	X	8.1158	7.8276	1.037
9TH	EQX	X	7.4371	7.2141	1.031
8TH	EQX	X	6.7124	6.5494	1.025
7TH	EQX	X	5.9532	5.7523	1.035
6TH	EQX	X	5.1608	5.0838	1.015
5TH	EQX	X	4.3645	4.3626	1.000
4TH	EQX	X	3.6409	3.6033	1.010
3RD	EQX	X	2.8983	2.8410	1.020
2ND	EQX	X	2.2122	2.1191	1.044
1ST	EQX	X	1.5617	1.4473	1.079
GF	EQX	X	0.9894	0.8634	1.146
Bsmnt	EQX	X	0.4768	0.3843	1.240
Roof	EQY	Y	7.3601	6.7612	1.089
9TH	EQY	Y	8.3280	6.9874	1.192
8TH	EQY	Y	7.6379	6.3721	1.199
7TH	EQY	Y	6.8475	5.6962	1.202
6TH	EQY	Y	6.0191	4.9956	1.205
5TH	EQY	Y	5.1370	4.2579	1.206
4TH	EQY	Y	4.2244	3.5017	1.206
3RD	EQY	Y	3.3215	2.7540	1.206
2ND	EQY	Y	2.4911	2.0601	1.209
1ST	EQY	Y	1.8588	1.4842	1.252
GF	EQY	Y	1.1215	0.8892	1.261
Bsmnt	EQY	Y	0.5105	0.4019	1.27

C.2 Time History Analysis Results

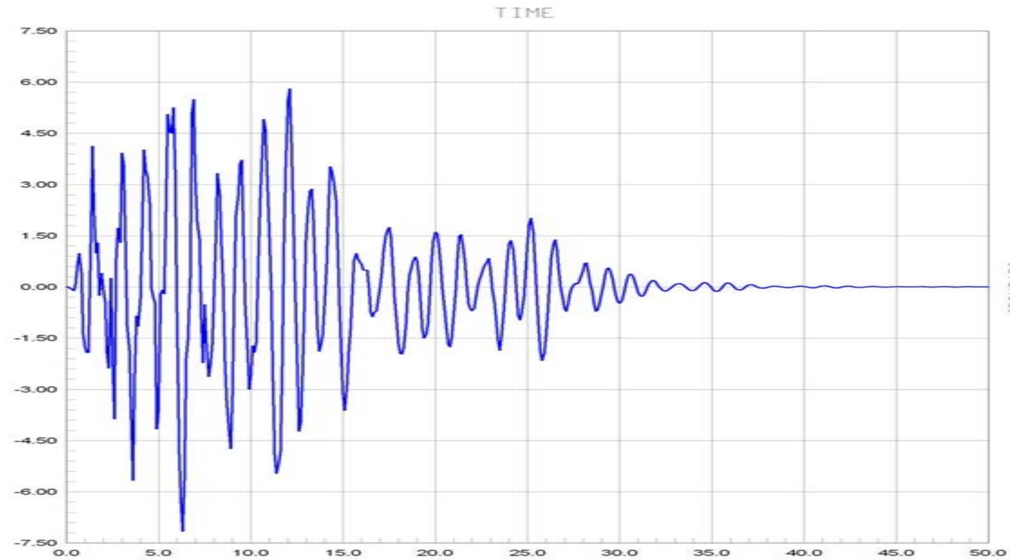


Fig C.1 Displacement Time History X direction due EQ1

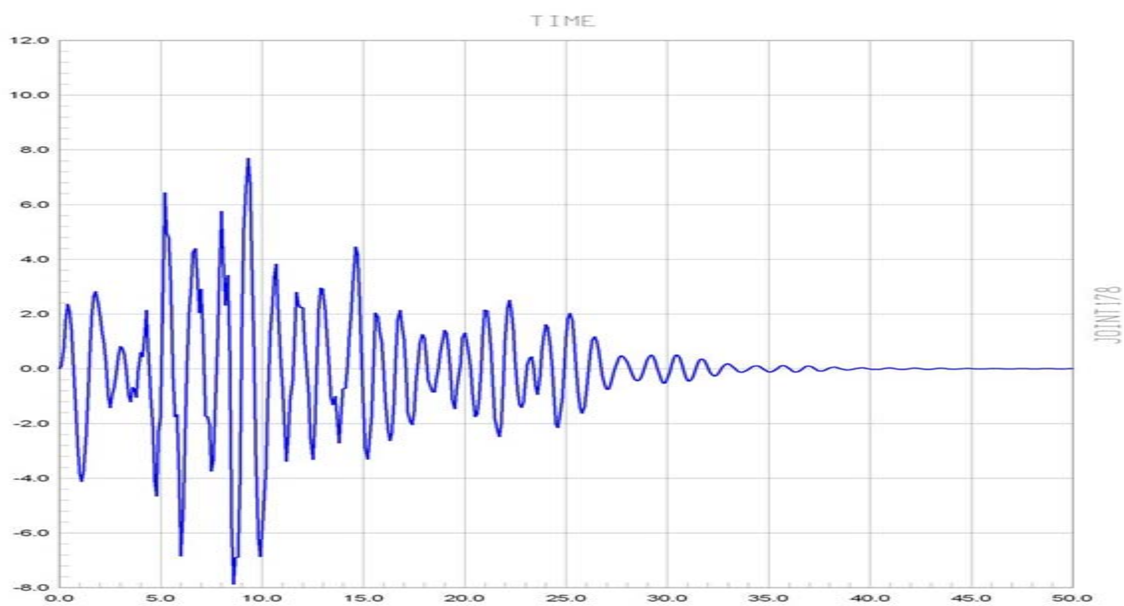


Fig C.2 Displacement Time History X direction due EQ2