

Effects of Earthquake in Hossana Town Selected buildings



ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Effects of Earthquake in Hosanna Town Selected buildings

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A thesis submitted to the school of graduate Studies of Addis Ababa University

Presented in partial fulfillment of the requirements for the Degree of

Master of Science in Civil Engineering

Addis Ababa University

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November, 2015

Effects of Earthquake in Hossana Town Selected buildings

Plagiarism Declaration

I, the undersigned, declare that this thesis is my original work and has not been presented as a thesis for a degree in elsewhere in any other universities. All sources of materials used in this thesis have been duly acknowledged.

Name: Aklilu Tadesse

Signature _____

Date November, 2015

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This is to certify that the thesis prepared by Aklilu Tadesse, entitled: Effects of Earthquake in Hosanna Town Selected buildings and submitted in partial fulfillment of the requirements for the Degree of Master of Science in Civil Engineering complies with the regulations of the university and meets the accepted standards with respect to originality and quality.

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Abstract

Effects of Earthquake in Hosanna Town Selected buildings

Aklilu Tadesse

Addis Ababa University, 2015

This thesis is conducted to investigate the effects of December 19, 2010 and November 24, 2011 earthquakes in Hosanna town. Due to the limitation of the available data only seven buildings were selected for the investigation. The paper examines the five buildings by qualitative analysis and two of buildings, both qualitative and quantitative studies.

The qualitative method was based on the damage evaluation method using FEMA and EMS. The results of the qualitative evaluation of buildings show that the damages were mainly non-structural and minor structural damages. The quantitative study of the two buildings was performed by using pushover analysis with SAP2000 software as per ATC-40 life safety performance criteria. Results of case study 01 showed that some structural members of building are heavily damaged, whereas the structural members of case study 02 building are in the safe range. Therefore, in order to sustain future earthquake some of the structural members of case study 01 need to be strengthened.

Keywords: Seismic evaluation; Damage, Nonlinear Static Analysis; pushover curve; Performance point

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1. Introduction

1.1.General

Hosanna town situated in Hadiya Zone in Southern regional state of Ethiopia. It is located at 230km away from Addis Ababa. It covers about 40.7 Km² of land. Based on 2007 census and using the regional grow rate 2.9, the population of the town is approximately 74,074 in 2009.

It is geographically located at 7°34'60" North and 37°52'60" East longitude with an elevation of 2177 meters above sea level, which is found at the western margin of the main Ethiopian Rift Valley that runs from northeast to southwest almost dissecting Ethiopia into two parts. Seismic activity is frequent in this region. The earthquakes which occurred in Dec.19, 2010 has a PGA of 0.35cm/s² which is much less than a design earthquake and Nov.24, 2011 which is not recorded, had created heavy damages and the collapse of some buildings of Hosanna town. This damage highlights the need of giving focus while designing for earthquakes.

In the Hosanna town, starting from past 10 years there is strong growth in the number of high rise buildings, residential houses, schools, health centers and other infrastructure constructions.

The most of buildings were not designed as per a strict earthquake resistant building guidelines and proper quality control procedures to assure compliance with some basic

requirements of the country's building code standards. Obviously this leads to the question "how much damage will be sustained when a major earthquake occurred?"

After each major earthquake in seismically active countries, there is a group of professionals who evaluates the damage in affected buildings and vulnerability of existing buildings for future earthquake. The vulnerability assessment may include qualitative and quantitative techniques with the aim of studying the level of damage and/or predicting the expected performance and safety of existing buildings as well as for carrying out necessary rehabilitation, if required.

Seismic evaluation of existing buildings after each earthquake happened is common with other countries for the purpose of safety of occupants. In our country Ethiopia especially in Hosanna town two major earthquakes are occurring during 2010 and 2011; however no seismic evaluations has been done. Here in this paper qualitative and quantitative seismic evaluation of selected existing buildings are studied.

1.2. Objectives

- To investigate qualitatively the level of damage of buildings in Hosanna due to the two earthquakes.
- To make quantitative investigation of selected buildings for future earthquakes.
- To create awareness for the need for seismic design of buildings and introduce the evaluation techniques from international practice.

1.3. Methodology

The complete work of this thesis is divided into two stages. The first part involves qualitative evaluation of actual damage occurred during past earthquakes using seismic evaluation guidelines. The second stage involves performing a quantitative evaluation using nonlinear static analysis of buildings whose design data were available.

1.4. Limitations of the study

Limitations

- a) Due to the scarcity of building data only seven buildings were investigated, out of which five qualitative and the remaining two for both qualitative and quantitative studies.
- b) The unavailability of the actual acceleration time history data to evaluate the actual damage with analytical study.

1.5. Organization of the thesis

The thesis has a total of six chapters. Chapter one introduction, it also includes the objective of the thesis work, scope and limitations of the thesis, as well as organization of the thesis. Chapter two is totally devoted to a literature review, to summarize the seismic evaluation methods. Chapter three devotes on qualitative and quantitative methodology. Analysis of Selected buildings discussed in chapter four. The results and findings are discussed in chapter five. Chapter Six contains the conclusion and indicates points for further researches. Appendix-A Structural drawings of Case study buildings, Appendix-B hinge state of the element at performance point, Appendix-C pushover analysis steps.

2. Literature Review

General

To provide a detailed review of the literature related to seismic evaluation of existing structures in it's entirely would be difficult to address in this chapter. A brief review of previous studies on the seismic evaluation using qualitative and quantitative methods is presented in this section.

2.1. Seismic Evaluation Documents

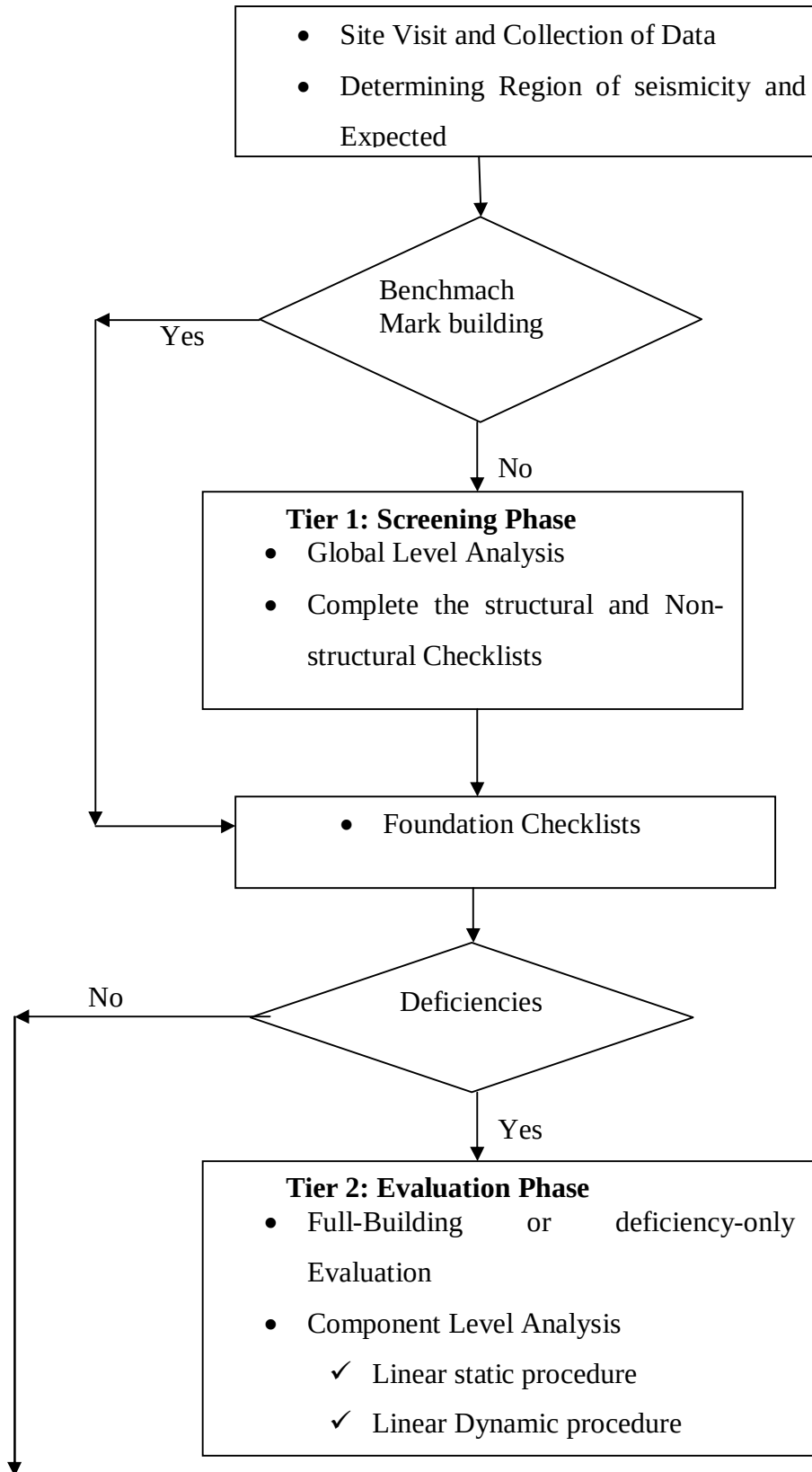
2.1.1. FEMA 310 –Handbook for the seismic Evaluation of Building

[21]:

It is probably the most advanced seismic evaluation procedure for buildings developed in the USA in the recent years, which grew out of the earlier document NEHRP Handbook for seismic Evaluation of Existing Buildings. The Evaluation procedure is based on a rigorous approach to determine structural Conditions. Buildings are evaluated for certain extent of structural damage that is expected in the building when subjected to earthquake. This level of damage (or performance level) is determined by the design professional considering the importance of building and consequences of damage.

FEMA 310 Considers two levels of performance defined as Life Safety and Immediate Occupancy during design earthquake. For Life Safety performance, the building can sustain significant damage to both structures non structural components with the margin against either partial or total structural collapse such that the level of risk for life –treating injury and getting trapped is low. Immediate occupancy building performance means

very limited damage to both structures non structural components during the design earthquake.



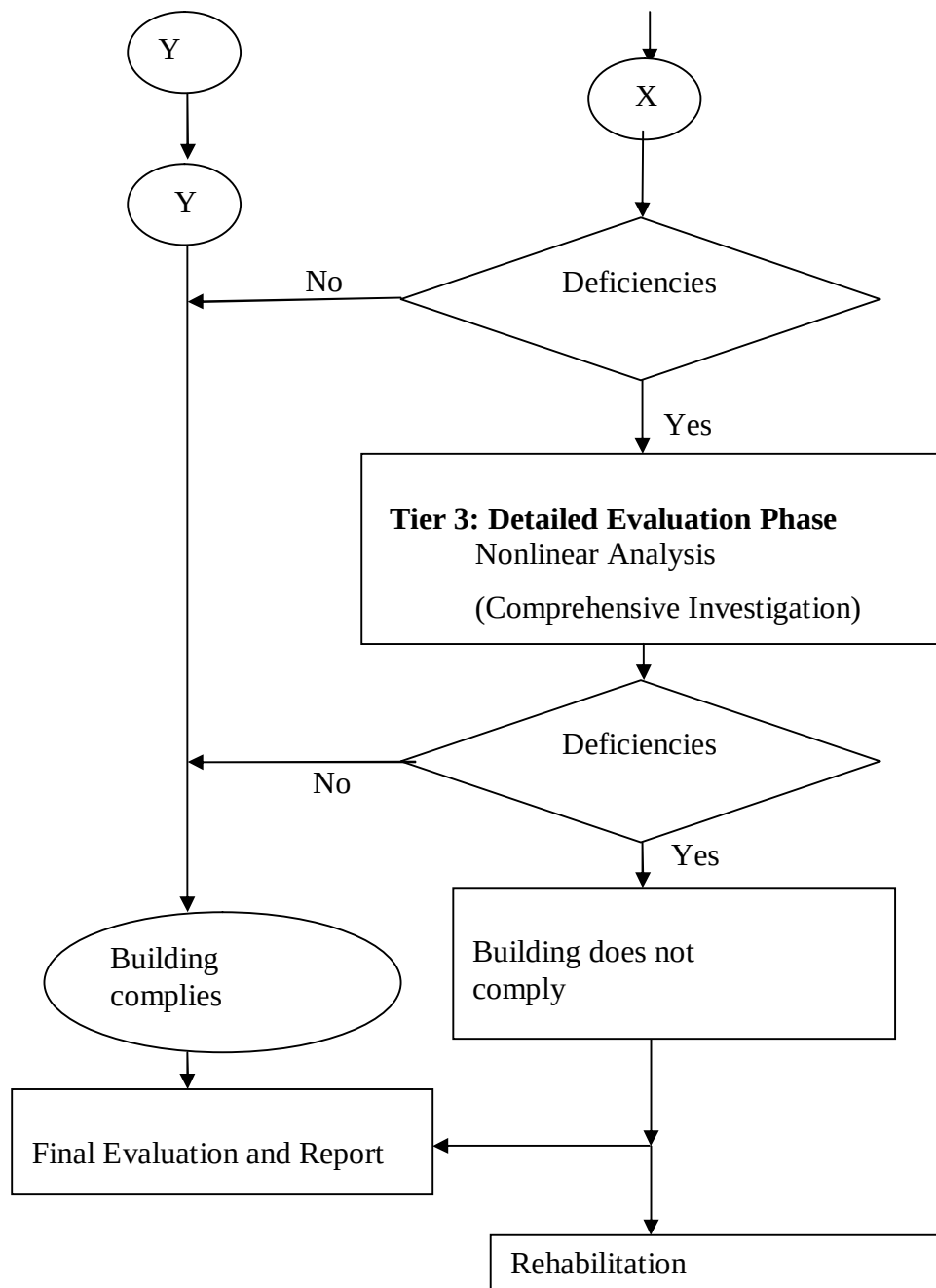


Figure 2.1: FEMA 310 Evaluation Procedures

2.1.2. Euro Code-8 Design Provisions for Earthquake Resistance of Structures-parts 1-4 General Rules for Strengthening and Repair of Buildings

The scope of this document is to provide criteria for the evaluation of seismic performance of existing individual structures, to describe the approach of selecting necessary corrective measures and to set forth criteria for the design of the repair/strengthening measures.

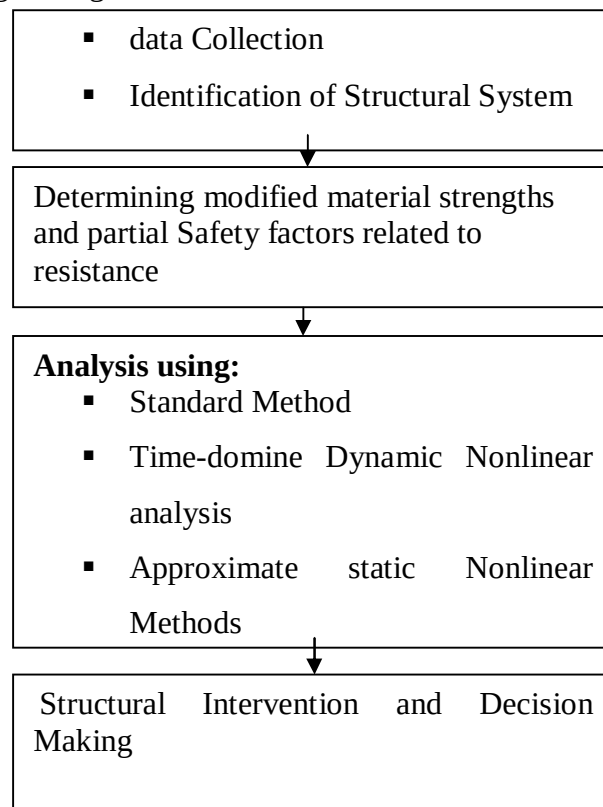


Figure 2:2: Eurocode 8-Evaluation Procedure

2.2. Summary of Evaluation procedures

All Evaluation Procedures follow similar assessment steps which can be broadly grouped into two categories: (a) Configuration- related and (b) strength-related checks

Table 2.1: FEMA 310 and Eurocode 8 Evaluation guidelines

	FEMA 310	EUROCODE 8
GENERAL		
	Three tiered approaches with increasing complexity and decreasing conservatism very thorough and detailed	Sound principles for two tiered assessments, but lacks specificity
CONFIGURATION RELATED CHECKS		
Load path	The structure shall contain one load path	A complete load path should be available from top to bottom
Soft storey	The stiffness of the lateral force resisting system in any storey shall not be less than 70% of the stiffness of adjacent storey	
Weak storey	The strength of the lateral force resisting system in any storey shall not be less than 80% of strength in an adjacent storey	
Geometry	There shall be no changes in the horizontal dimension of a lateral resisting system of more than 30% in a storey relative to adjacent stores	Criteria for regularity in plan and elevation are checked

Effective mass	There shall be no change in effective mass more than 50% from one store to next	There should not be abrupt changes in the mass of the individual storey
Torsion	Distance between storey mass center and storey center of rigidity shall be less than 20% of the building width in either plan dimension	Frame, dual and wall system shall possess a minimum Torsional rigidity $\frac{r}{I_s} > 0.80$
Adjacent Buildings	An adjacent building shall not be located next structure being evaluated closer than 4% of the height.	$\delta \geq \sqrt{\delta_1^2 + \delta_2^2}$
STRENGTH RELATED CHECKS		
Analysis philosophy	Calculates the pseudo lateral force to impose the expected actual deformation of the structure in its yielded state when subjected to design earthquake motions.	Evaluation consists of the verification of the seismic resistance of an existing damaged or undamaged building, taking into account both non-seismic and seismic actions for the period of its intended lifetime.

Table 2.1: Continued

2.3. Damage Classification Reinforced Concrete Structure [19]

The way in which a building deforms under earthquake loading depends on the building type. As a broad categorization one can group together types of masonry buildings as well as buildings of reinforced concrete.

Table 2.2: Classification of damage to reinforced Concrete buildings

Damage Grade	Description of Damage
Grade 1	Negligible to slight damage (No structural damage, slight non-structural damage) Fine cracks in plaster over framing members Or in walls at the base. Fine cracks in partitions and in fills.
Grade 2	Moderate damage (Slight structural damage, moderate non-structural damage) Cracks in columns and beams of frames and in structural walls. Cracks in partition and infill walls; fall of brittle cladding and plaster. Falling mortar from the joints of wall panels.
Grade 3	Substantial to heavy damage (Moderate structural damage, heavy non-structural damage) Cracks in columns and beam column joints of frames at the base and at joints of coupled walls. Spalling of concrete cover, buckling of reinforced rods. Large cracks in partition and infill walls, failure of individual infill panels.
Grade 4	Very heavy damage (Heavy structural damage, very heavy non-structural damage) Large cracks in structural elements with compression failure of concrete and fracture of rebars; bond failure of beam reinforced bars; tilting of columns. Collapse of a few columns or on a single upper floor.
Grade 5	Destruction (very heavy structural damage) Collapse of ground floor or parts (e. g .Wings) of buildings.

2.4. Seismic performance assessment of buildings

The seismic performance of buildings [4] is measured by the state of damage under a certain level of seismic hazard. The state of damage is quantified by the drift of the roof and the displacement of the structural elements. Initially, gravity push is carried out using the force control method. It is followed by a lateral push with displacement control using SAP2000 [6]. For carrying out displacement based pushover analysis, target displacement needs to be defined. Pushover analysis gives an insight into the maximum base shear that the structure is capable of resisting. A building performance level is a combination of the performance levels of the structure and the nonstructural components. A performance level describes a limiting damage condition which may be considered satisfactory for a given building with specific ground motion. The performance levels as per [5] and [4] are:

Immediate occupancy (IO): damage is relatively limited; the structure retains a significant portion of its original stiffness and most if not all of its strength.

Life safety level (LS): substantial damage has occurred to the structure, and it may have lost a significant amount of its original stiffness. However, a substantial margin remains for additional lateral deformation before collapse would occur.

Collapse prevention (CP): at this level the building has experienced extreme damage, if laterally deformed beyond this point; the structure can experience instability and collapse.

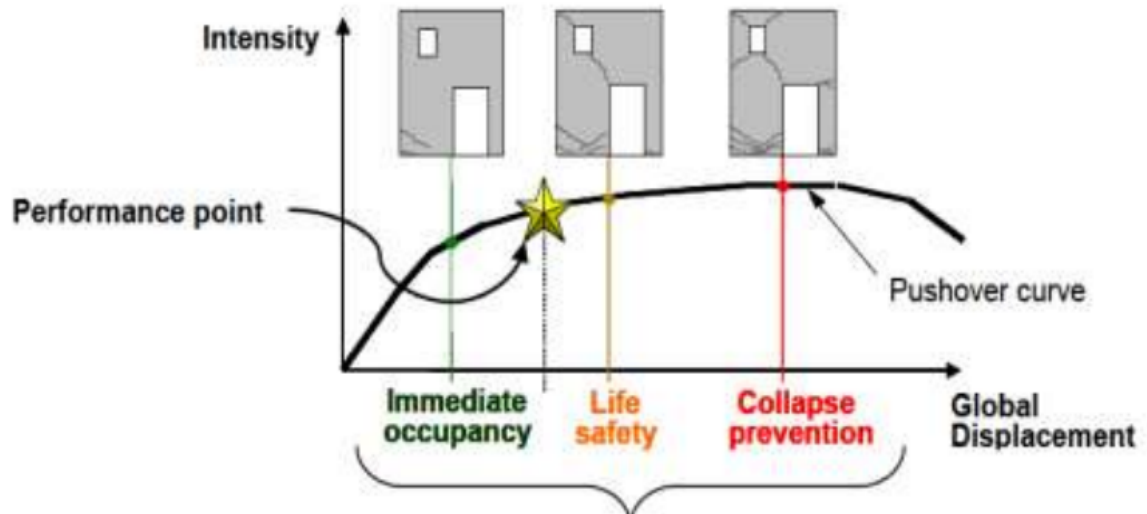


Figure 2:3: performance levels described by a pushover curve [8]

Whittaker A summarizes the next (second) generation tools and procedures for performance-based earthquake engineering in the United States [2]. The methodology, which is described in detail in the draft Guidelines for the Seismic Performance Assessment of Buildings, builds on the first generation deterministic procedures, which were developed in the ATC-33 project in the mid 1990s and in ASCE Standard: ASCE/SEI 41-06 Seismic Rehabilitation of Existing Buildings.

The procedures and methodologies described in these guidelines include an explicit treatment of the large uncertainties in the prediction of losses due to earthquakes. This formal treatment of uncertainty and randomness represents a substantial advance in performance based engineering and a significant departure from the first generation deterministic procedures. Fig.2-4: identifies the five basic steps proposed for a next-generation seismic performance assessment. Unlike prior assessment procedures that addressed either structural damage or repair cost, three measures of seismic

performance are proposed in the guidelines: 1) direct economic loss (repair cost), 2) indirect economic loss (downtime or business interruption), and 3) casualties (including injuries and death). Each of three performance measures is treated as a potential loss. Section 2 of the paper introduces the three types of performance assessment that can be performed using the draft Guidelines and identifies the basic procedure for each. Section 3 describes the five steps for seismic performance assessment.

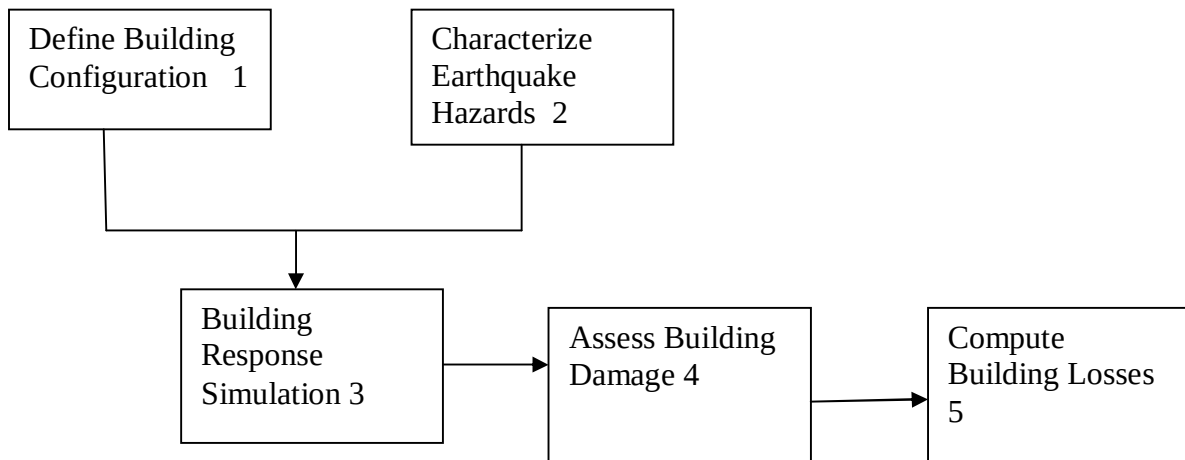


Figure 2:4: Procedures for Performance Assessment [3]

The procedures set forth in these guidelines represent a substantial departure from the deterministic tools and procedures used at this time because uncertainty and randomness is captured explicitly in every step of the proposed procedures. Fragility functions, damage states and building-level consequence functions, are used in the proposed procedures to compute losses.

2.5. Pushover analysis

Pushover Analysis option will allow engineers to perform pushover analysis as per ATC 40 and FEMA 356. Pushover analysis is a static, nonlinear procedure using the simplified nonlinear technique to estimate seismic structural deformations. It is an incremental static analysis used to determine the force-displacement relationship, or the capacity curve, for a structure or structural element. The analysis involves applying horizontal loads, in a prescribed pattern, to the structure incrementally, i.e. pushing the structure and plotting the total applied shear force and associated lateral displacement at each increment, until the structure or collapse condition. [21] Pushover analysis is a technique by which a computer model of the building is subjected to a lateral load of a certain shape (i.e., inverted triangular or uniform). The intensity of the lateral load is slowly increasing and the sequence of cracks, yielding, plastic hinge formation, and failure of various structural components is recorded. Pushover analysis can provide a significant insight into the weak links in seismic performance of a structure. A series of iterations are usually required during which, the structural deficiencies observed in one iteration, are rectified and followed by another. This iterative analysis and design process continues until the design satisfies a pre-established performance criteria. The performance criterion for pushover analysis is generally established as the desired state of the building given rooftop or spectral displacement amplitude. Static Nonlinear Analysis technique, also known as sequential yield analysis, or simply “pushover” analysis has gained significant popularity during the past few years.

2.6. Developing Pushover Curve [21]:

As per FEMA 310 after assigning all properties of the models, the displacement controlled pushover analysis of the models is carried out. The models are pushed in the monotonically increasing order until target displacement is reached or structure loses equilibrium which occurs first. For this purpose, target displacement at roof level and the number of steps in which this displacement must be defined. In this study, target displacement takes 4% of building height. Pushover curve is the base shear force versus a roof displacement curve. The peak of this curve represents the maximum lateral load carrying capacity of the structure. The initial stiffness of the structure are obtained from the tangent at pushover at zero level. The collapse is assumed when the structure loses 75% strength and corresponding roof displacement is called “maximum roof Displacement.” It is plot drawn between base shear and roof displacement. Performance point and location of hinges in various stages can be obtained from pushover curve. The range AB is elastic range, B to IO is a range of immediate occupations. IO tools are a range of life safety and LS to CP is arranged of collapse prevention. When a hinge reaches appoint C on its force displacement curve that hinge must begin to drop a load. The way load is dropped from a hinge that has reached point C is that pushover force (base shear) is reduced until the force in that hinge is consistent with force at point D. As the force is dropped, all elements unloaded, and the displacement is reduced. Once the yielded hinge reaches the point D force level, the pushover force is again increased and the displacement begins to increase again. If all the hinges are within the CP limit, then the structure is said to be safe. However, depending upon the importance of the structure the hinge after IO range may also need to be retrofitted.

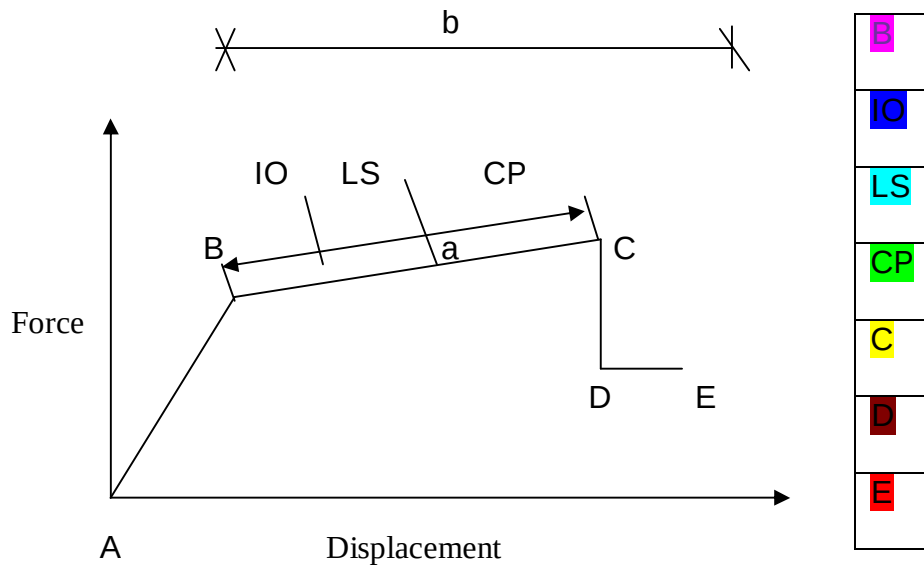


Figure 2:5 Pushover Curve and Hinge state

Slope of BC percentage (0-10%) of elastic slope which represents a strain hardening and ordinate C- represents strength hardening.

2.7. Summary of literature review

Many qualitative and analytical works have been done by many researchers in the area of seismic evaluation of existing buildings. A number of conclusions can be drawn from the above review of seismic evaluations. Firstly, pushover analysis is becoming a standard seismic assessment and design tools for quantitative evaluations. Secondly, different countries evaluation guidelines are used for the purpose of qualitative evaluations. This research is concerned with the pushover analysis of the selected RCC buildings and qualitative damage evaluation of past earthquakes.

3. Qualitative and Quantitative Evaluation

3.1. Qualitative Evaluation Method

3.1.1. General

The qualitative evaluation is based on the inspection of the building as a whole and in detail and on the examination of:

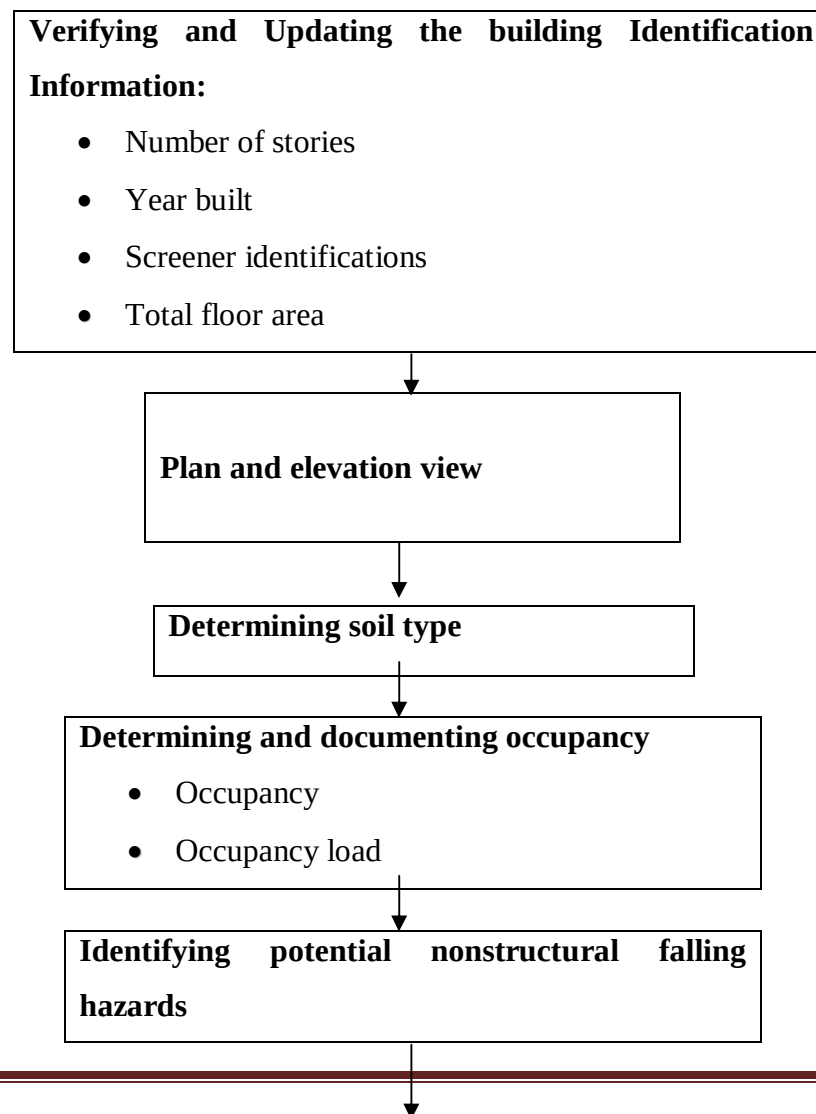
- the structural and architectural design of the building;
- the plotting of the building, its members and details considered significant in the assessment of the protection level, whenever the original design is not available or when the building of the structure fails to comply with the design, or when the building has suffered structural transformations during its service life without any purpose-made documentation;
- the designs and documentation on which previous interventions were based, as well as other information on the history of the structure;
- information on the building behavior during previous seismic events;
- considerations related to the norms on which the design of the building was based or, if necessary, related to the date practices compared with the present specifications in force;
- the plotting of the possible damage and deterioration;
- the building as a whole and in detail.

The object of the qualitative assessment is the structural system as well as the non-structural internal or external members that are likely to cause accidents during seismic events (partition walls, parapets, ornaments, blind walls, chimneys, etc.).

The qualitative methods are generally analyzed by two methods: visual inspection and nondestructive tests.

3.1.2. Visual Inspection [24]

Visual inspection provides the most valuable information, as it can be used to identify any configuration issues, and it allows the measurement of component dimensions, and the determination of whether the degradation is present. The basic procedure of visual inspection indicated as follows:



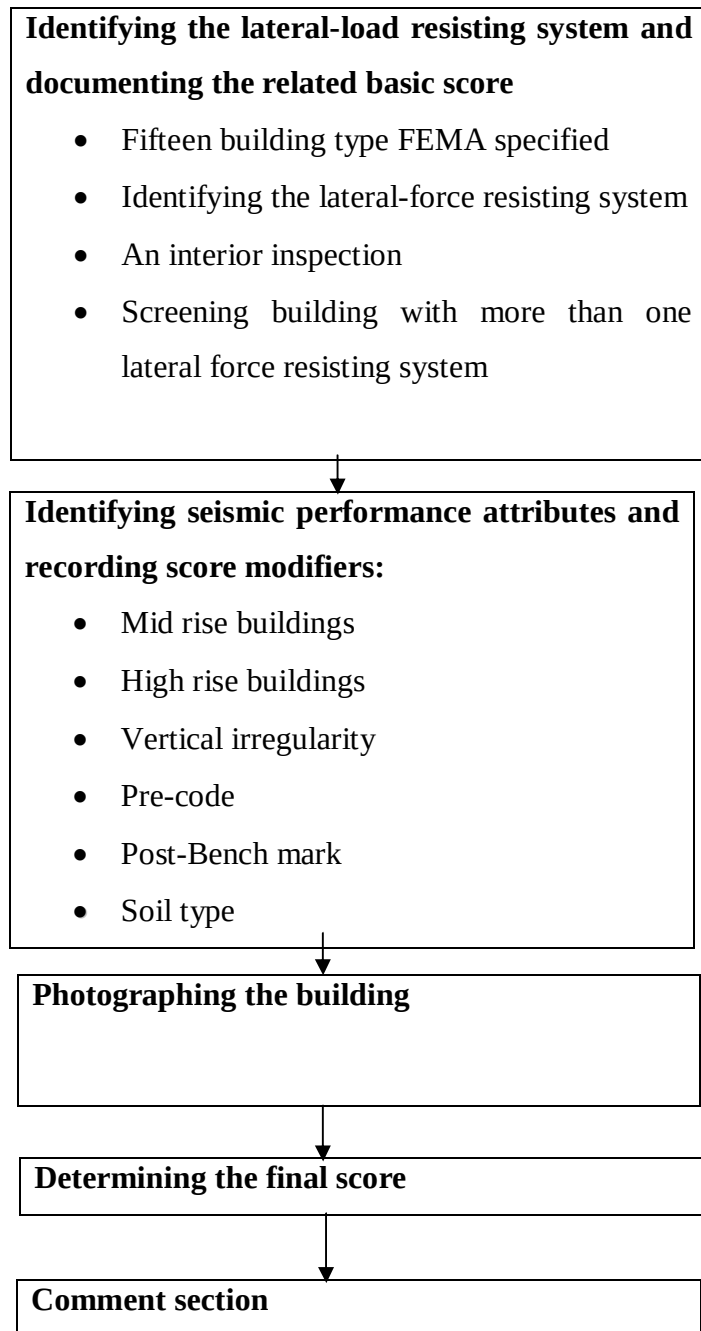


Figure 3:1: FEMA Visual inspection guidelines

3.1.3. Non-destructive test [4]

The physical conditions of components dictate the need for certain destructive and nondestructive test methods. Such methods may be used to determine the degree of damage or presence, contamination, and to improve understanding of the internal condition and quality of the concrete.

Guidelines and procedures for destructive and nondestructive tests that may be used in the condition assessment. The following paragraphs identify those nondestructive examination (NDE) methods having the greatest use and applied to condition assessment.

Surface NDE methods include infrared thermography, delamination sounding, surface hardness measurement, and crack mapping. These methods may be used to find surface degradation in Components such as service-induced cracks, corrosion, and construction defects.

Volumetric NDE methods, including radiography and ultrasound, may be used to identify the presence of internal discontinuities, as well as to identify loss of section. Impact-echo ultrasonic is particularly useful because of ease of implementation and proven capability in concrete.

Structural condition and performance may be assessed through on-line monitoring using acoustic emissions and strain gauges, and in-place static or dynamic load tests. Monitoring is used to determine if active degradation or deformations are occurring, while nondestructive load testing provides direct insight on load-carrying capacity.

3.2.Quantitative Evaluation Method

3.2.1. General

The quantitative evaluation method aims at:

- Determining the conventional capacity to bear the seismic loads of the inspected building;
- Identifying highly vulnerable members/areas of the structure;

3.2.2. Non-linear Time history analysis

The method consists of performing a time-history analysis in the nonlinear domain.

The seismic action is directly applied, by means of accelerograms, at the base of the structure. The use of at least 3 significant accelerograms consistent with the site conditions of the building is needed. Although it is the most rigorous method from the theoretical point of view, this approach has important limitations due especially to the large amount of time required. For the analysis in the case of big structures, as well as the high level of technical knowledge needed for design engineers.

3.2.3. Capacity Demand analysis

Capacity/Demand (C/D) method: The method has been initially presented by Applied Technology Council (ATC).The forces and displacements resulting from an elastic analysis for design earthquake are called demand. These are compared with the capacity of different members to resist these forces and displacements. A (C/D) ratio less than one indicates member failure and thus needs retrofitting. The seriousness of the deficiencies can be easily be assessed by listing the seismic-demand –to-capacity ratio in descending order. The C/D procedures have been subjected to more detailed examination in the light

of recent advances in earthquake response studies. The main difficulty encountered in using this method: there is no relationship between member and structure ductility factor because of nonlinear behavior and the demand involves gravity effects, the ratio of demand to capacity of an element may not be necessary to be a good indicator of the seriousness of hazards due to earthquake hazard.

3.2.4. Pushover analysis

The pushover analysis of a structure is a static nonlinear analysis under permanent vertical loads and gradually increasing lateral loads. The equivalent static lateral loads approximately represent earthquake induced forces. A plot of the total base shear versus top displacement in a structure is obtained by this analysis that would indicate any premature failure or weakness. The analysis is carried out up to failure, thus it enables determination of collapse load and ductility capacity. On a building frame, and plastic rotation is monitored, and lateral inelastic forces versus displacement response for the complete structure is analytically computed. This type of analysis enables weakness in the structure to be identified. The decision to retrofit can be taken in such studies.

The purpose of pushover analysis is to evaluate the expected performance of structural systems by estimating the performance of a structural system by estimating its strength and deformation demands in design earthquakes by means of static inelastic analysis, and comparing these demands to available capacities at the performance levels of interest.

Push over procedure is gaining popularity during the last few years as appropriate analytical tools are now available (SAP- 2000, ETABS).

Pushover analysis is of two types, (i) force controlled or (ii) displacement controlled. In the force control, the total lateral force is applied to the structure in small increments. In the displacement control, the displacement of the top storey of the structure is incremented step by step, such that the required horizontal force pushes the structure laterally. The distance through which the structure is pushed, is proportional to the fundamental horizontal translational mode of the structure. In both types of pushover analysis, for each increment of the load or displacement, the stiffness matrix of the structure may have to be changed, once the structure passes from the elastic state to the inelastic state. The displacement controlled pushover analysis is generally preferred over the force controlled one because the analysis could be carried out up to the desired level of the displacement

Advantage of pushover analysis over other methods

- Reasonable estimates of inelastic deformation or damage in structures
- Elastic Analysis is not capable of providing information
- Nonlinear dynamic response history analysis is capable of providing the required information, but may be very time-consuming
- Nonlinear static pushover analysis may provide reasonable estimates of the location of inelastic behavior
- Does not require selection and scaling of ground motions.

The use of pushover analysis is simple and realistic. The elastic analysis does not have the capability to compute inelastic deformations, hence it is out. Nonlinear response history analysis (NRHA) is certainly viable, but is very time consuming. Also, it may produce a very wide range of responses for a system subjected to a suite of appropriately

scaled ground motions. Computed deformation demands can easily range by an order of magnitude (or more) making it difficult to make engineering decisions. Hence, I left with Nonlinear Static Pushover Analysis (NSPA) as a reasonable alternative for this thesis work.

3.2.5. Quantitative Analysis of case study buildings using pushover analysis

Nonlinear static analysis, or pushover analysis, could be performed directly by a computer program which can model the nonlinear behavior of lateral load resisting members of a structure. Due to lack of design data two buildings were selected for quantitative analysis.

The buildings designated by case study 01 (Hosanna condominium) and case study 02 (Sawo building). In this study, pushover analyses were performed using SAP2000 V14 software.

Following assumptions are made while analyzing a structure in the SAP2000:

- (i) The material is homogeneous, isotropic,
- (ii) All columns supports are considered as fixed at the foundation ,
- (iii) Tensile strength of concrete is ignored in sections subjected to bending ,
- (iv) The super structure is analyzed independently from foundation and soil medium, on the assumptions that foundations are fixed ,
- (v) Pushover hinges are assigned to all the member ends. In case of Columns PMM hinges (i.e. Axial Force and Biaxial Moment Hinge) are provided while in case of beams M3 hinges (i.e. Bending Moment hinge) are provided

- (vi) The maximum target displacement of the structure is calculated in accordance with the guidelines given by FEMA 356.

3.2.5.1. Pushover analysis procedure

- a. Collect information on the selected case study buildings.
- b. Evaluate if the collected data is sufficient enough to carry out a seismic evaluation of the buildings.
- c. Create the basic computer model (without the pushover data)
- d. Define properties and acceptance criteria for the pushover hinges
- e. Model the buildings using SAP2000 analysis software.
- f. Locate the pushover hinges on the model by selecting one or more frame members and assigning them one or more hinge properties and hinge locations.
- g. A predefined lateral load pattern which is distributed along the building height is then applied.
- h. Run the basic nonlinear static analysis
- i. Display the pushover curves
- j. Review the pushover displaced shape and sequence of hinge formed on a step-by-step basis.
- k. Determine steps at which performance point is located
- l. At performance point review hinge state of each member

4. Analysis of selected buildings by Qualitative and Quantitative methods

4.1. Case Study building descriptions

Table 4.1: Case study building description

Case Study	Building type	Building use
01	G+2	Residential
02	G+3	Mixed
03	G+2	Office
04	G+3	Commercial
05	G+0	School

4.2. Qualitative analysis of Case study buildings

All buildings are analyzed as per EMS-98 and FEMA -154 guidelines as follows:

4.2.1. Case study 01 Building

It is a two storey reinforced concrete residential building which is built in 2008 and has a floor of 286.2m². It is located seismic zone 4.



Figure 4.1: Front view of case study 01

Table 4.2: Visual inspection chart for case study 01

Earthquake/site			No. Stories	Two				
2011/Gofarmeda			Screener	Aklilu T				
			Total Floor Area	286m2				
Occupancy			Soil Type					
Assembly	Gov	Office	A	B	C	D	E	F
Commercial	Historic	Residential	Hard	Average	Dense	Stiff	Soft	Poor
Emer Services	Industrial	School	Rock	Rock	Soil	soil	soil	soil
FALLING HAZARDS								
Unreinforced	Parapets	Cladding	other					
chimneys			Window glass					
BASIC SCORE,MODIFIERS,AND FINAL SCORE,S								
Building type			C1					
			(MRF)					
Basic score		3						
Mid Rise (4 to 7 stories)		0.2						
High Rise (>7 stories)		0.5						
Vertical Irregularity		-2						
plan Irregularity		-0.5						
pre-code		-1						
Post-Bench mark		1.2						
Soil type C		-0.6						
Soil type D		-1						
Soil type E		-1.6						
FINAL SCORE								
COMMENTS								
By adding the highlighted numbers, final score of 2.6 was determined. Hence a final score is greater than cutoff score 2.0. Hence, detailed evaluation of the building is not required								



Figure 4:2: Case study 01 actual damage photo

Damage incurred: Diagonal cracks on the wall between axis B and C in the exterior wall of the ground floor, which is on the longer direction. The damages are large and extensive, but not all of them penetrate the whole wall thickness of the wall. Moderate non-structural damage and categorized on damage grade2 as per EMS-98.



Figure 4:3: Case study01 damage view photo

Damage incurred: damage of window glass on the ground floor. Slight non structural damage categorized grade1 as per EMS-98.



Figure 4:4: Case Study 01 actual damage photo

Damage incurred: removal of clear cover from floor beam and bulging of window frame.

Moderate non- structural damage categorized grade 2 as per EMS-98.

Key damage observed in case study 01

Visual inspections by damage photo indicate that the building has suffered minor earthquake damage in the long direction of the building.

The key damage observed includes:

- Diagonal Cracks in the external and internal walls
- The fall of plaster and removal of concrete cover some beams
- Misalignment of doors and windows
- It is generally categorized as per EMS-98 from damage grade 1 to grade2

4.2.2. Case study 02 Building

Building Description

It is a three storey reinforced concrete mixed use building which is built in 2009 and has a floor of 380m². This structure is occupied by the commercial units at ground story and by residential units at the upper three stories. It is located seismic zone 4.



Figure 4:5: Case study 02 Front view

Table 4.2: Visual Inspection chart for Case study 02

Earthquake/site			No. Stories	Three					
2011/around Bus station			Screener	Aklilu T					
			Total Floor Area	360m2					
Occupancy			Soil Type						
Assembly	Gov	Office	A	B	C	D	E	F	
Commercial	Historic	Residential	Hard	Average	Dense	Stiff	Soft	Poor	
Emer Services	Industrial	School	Rock	Rock	Soil	soil	soil	soil	
FALLING HAZARDS									
Unreinforced	Parapets	Cladding	other						
chimneys									
BASIC SCORE,MODIFIERS,AND FINAL SCORE,S									
Building type			C1						
			(MRF)						
Basic score		3							
Mid Rise (4 to 7 stories)		0.2							
High Rise (>7 stories)		0.5							
Vertical Irregularity		-2							
plan Irregularity		-0.5							
pre-code		-1							
Post-Bench mark		1.2							
Soil type C		-0.6							
Soil type D		-1							
Soil type E		-1.6							
FINAL SCORE									
COMMENTS									
By adding the highlighted numbers, final score of 3.2 was determined. Hence a final score is greater than cutoff score 2.0. Hence detailed evaluation of the building is not required									

Recorded typical damage photo of Case Study 02 Mixed use building



Figure 4:6: Case study02 actual damage view photo

Damage incurred: from the ground to third floor cracks on infill wall and partial removal of clear cover on floor beam. Slight non -structural damage categorized grade1 as per EMS-98.



Figure 4:7: Case study02 actual damage view photo

Damage incurred: from the ground to third floor slight cracks between infill wall and reinforced concrete beam. Damage categorized grade1 as per EMS-98.

Key damage observed

Visual inspections indicate the building has suffered slight earthquake damage.

The key damage observed includes:

- Diagonal Cracks in the mortar to the block work cladding
- The removal of plaster and concrete cover on floor beam
- Minor cracks to concrete masonry block linings units
- It is generally categorized as per EMS-98 from damage grade 1

4.2.3. Case Study 03 Building:

Case study 03 building is office building which is two stories reinforced concrete frame building. It is L-shaped in plan but has construction joint between two parts and located in zone 4. It is the one of the building which is affected by Hosanna earthquake 2010.



Figure 4:8: Figure Front view of Case study 03 building

Table 4.3: Visual inspection chart for Case study 03 building

Earthquake/site 2010/			No. Stories Three Screener Aklilu T Total Floor Area							
Occupancy			Soil Type							
Assembly	Gov	Office	A	B	C	D	E	F		
Commercial	Historic	Residential	Hard	Average	Dense	Stiff	Soft	Poor		
Emer Services	Industrial	School	Rock	Rock	Soil	soil	soil	soil		
FALLING HAZARDS										
Unreinforced	Parapets	Cladding								other
chimneys		Some HCB								
BASIC SCORE,MODIFIERS,AND FINAL SCORE,S										
Building type			C1							
			(MRF)							
Basic score		3								
Mid Rise (4 to 7 stories)		0.2								
High Rise (>7 stories)		0.5								
Vertical Irregularity		-2								
plan Irregularity		-0.5								
pre-code		-1								
Post-Bench mark		1.2								
Soil type C		-0.6								
Soil type D		-1								
Soil type E		-1.6								
FINAL SCORE										
COMMENTS By adding the highlighted numbers, final score of 3.2 was determined. Hence a final score is greater than cutoff score 2.0. Hence, detailed evaluation of the building is not required										

Recorded damage photo of Case study 03 office building



Figure 4:9 : Actual damage view photos of case study 03 building

Damage incurred: from the ground to third floor, cracking at the beam and wall interface of infill walls. Damage categorized grade2 as per EMS-98



Figure 4:10: Actual damage view photo of case study 03 building

Damage incurred: Large cracks in partition walls. Damage categorized grade2 as per EMS-98.

Key damage observed

Visual inspections indicate the building has suffered minor earthquake damage.

The key damage observed includes:

- Diagonal Cracks in the wall
- The fall of plaster and concrete cover on floor beam
- Minor cracks to concrete masonry block linings units
- It is generally categorized as per EMS-98 from damage grade 1 to grade 2

4.2.4. Case Study 04 Building

It is a commercial building with three story reinforced concrete frame building. It is rectangular-shape in plan and located on zone 4. It is the one of the building which is affected by Hosanna earthquake.



Figure 4:11: Front view of Case Study 04 Building

2010/			Aklilu Screener T Total Floor Area					
Occupancy			Soil Type					
Assembly	Gov	Office	A	B	C	D	E	F
Commercial	Historic	Residential	Hard	Average	Dense	Stiff	Soft	Poor
Emer Services	Industrial	School	Rock	Rock	Soil	soil	soil	soil
FALLING HAZARDS								
Unreinforced	Parapets	Cladding	other					
chimneys								
BASIC SCORE,MODIFIERS,AND FINAL SCORE,S								
Building type			C1					
			(MRF)					
Basic score		3						
Mid Rise (4 to 7 stories)		0.2						
High Rise (>7 stories)		0.5						
Vertical Irregularity		-2						
plan Irregularity		-0.5						
pre-code		-1						
Post-Bench mark		1.2						
Soil type C		-0.6						
Soil type D		-1						
Soil type E		-1.6						
FINAL SCORE								
COMMENTS								
By adding the highlighted numbers, final score of 3.2 was determined. Hence a final score is greater than cutoff score 2.0. Hence ,detailed evaluation of the building is not required								

Table 4.4: Visual inspection of Case study 04 building

Recorded damage photo of Case study 04 Building

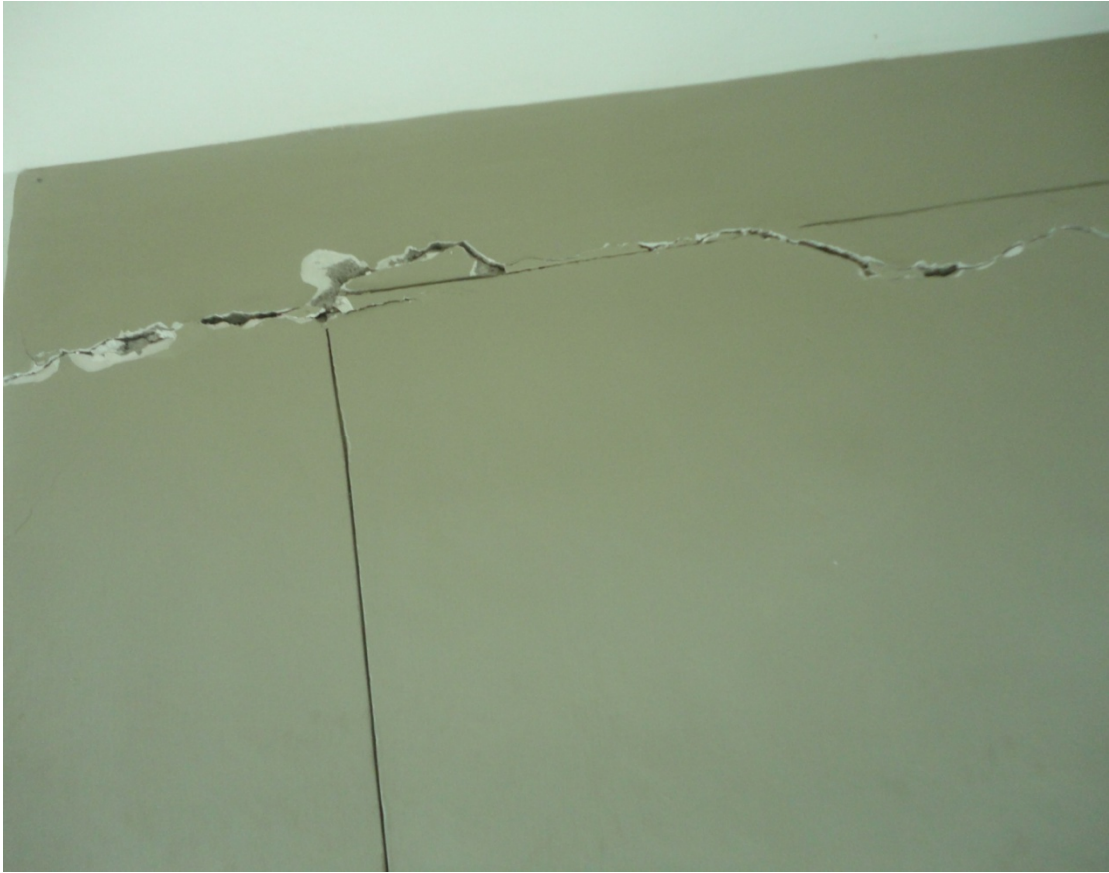


Figure 4:12: Damage photo of Case Study 04 Building

Damage incurred: from ground floor to third floor cracks on walls and junction of frame.

Damage categorized grade2 as per EMS-98.



Figure 4:13 : Damage photo of Case study 04 Building

Damage incurred: from ground floor to third floor partial removal of the plastering from walls. Damage categorized grade2 as per EMS-98.

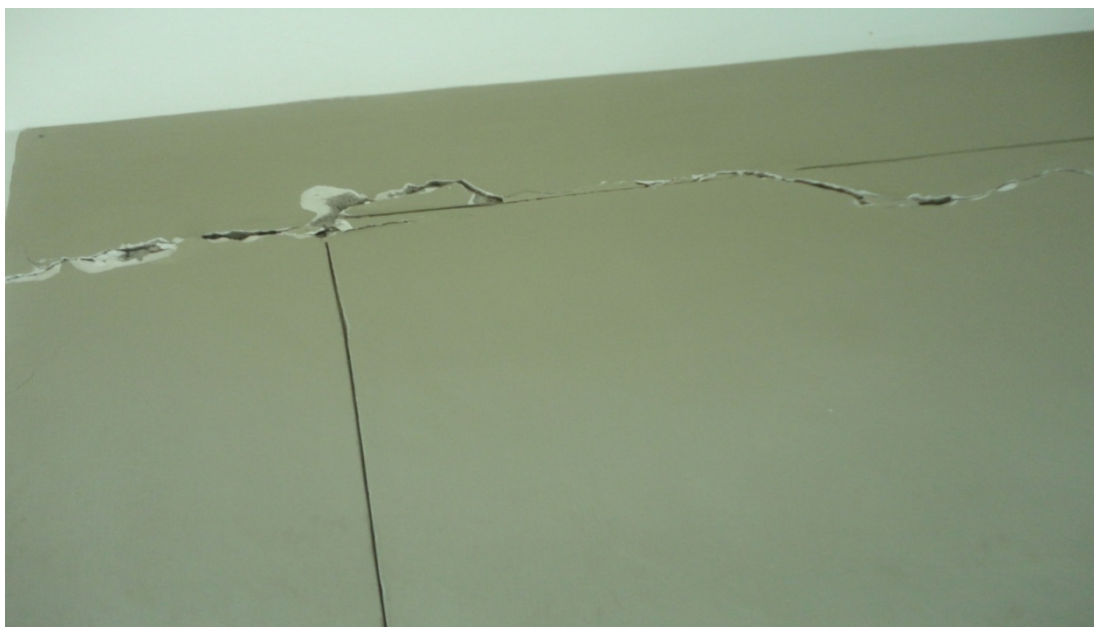


Figure 4:14: Damage photo of Case Study 04 Building

Damage incurred: from ground floor to third floor cracks on walls and junction of frame

Damage categorized grade2 as per EMS-98.

Key damage observed

Visual inspections indicate the building has suffered minor earthquake damage.

The key damage observed includes:

- Diagonal Cracks in the mortar to the block work cladding
- the fall of plaster and concrete cover on floor beam
- Minor cracks to concrete masonry block linings units
- It is generally categorized as per EMS-98 from damage grade 2

4.2.5. Case Study 05 Building

Table 4.5 Visual inspection

The Case Study 05 School is located 9km away from Hosanna Town.

Earthquake/site 2010/			No. Stories 0 Screener Aklilu T Total Floor Area						
Occupancy			Soil Type						
Assembly	Gov	Office	A	B	C	D	E	F	
Commercial	Historic	Residential	Hard	Average	Dense	Stiff	Soft	Poor	
Emer Services	Industrial	School	Rock	Rock	Soil	soil	soil	soil	
FALLING HAZARDS									
Unreinforced	Parapets	Cladding							other
chimneys									
BASIC SCORE,MODIFIERS,AND FINAL SCORE,S									
Building type			W1						
Basic score			4.4						
Mid Rise (4 to 7 stories)			N/A						
High Rise (>7 stories)			N/A						
Vertical Irregularity			-2.5						
plan Irregularity			-0.5						
pre-code			0.1						
Post-Bench mark			2.4						
Soil type C			0						
Soil type D			0						
Soil type E			0						
FINAL SCORE									
COMMENTS									
By adding the highlighted numbers, final score of 4.4 was determined. Hence a final score is greater than cutoff score 2.0. Hence detailed evaluation of the building is not required									



Figure 4:15: Damage photo of case study 05

Damage incurred partial collapse of building. Damage categorized grade3 as per EMS98

Key damage observed

Visual inspections indicate the building has suffered minor earthquake damage.

The key damage observed includes:

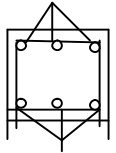
- Partial collapse of walls.
- It is generally categorized as per EMS-98 from damage grade 2

4.3.Quantitative analysis of Case study buildings

4.3.1. Case study buildings descriptions

Due to lack of design data only case study 01 and 02 buildings analyzed by quantitative method using SAP2000 pushover analysis.

Table 4.6: Asbuilt Design data

Description	Unit/material type	Case Study		Remark
		01	02	
Plan area	m ²	286	380	
Height of the ground floor	m	2.50	2.80	
Remaining floors	M	3.01	3.0	
Material Quality:	Concrete	C-25	C-25	
	Steel:	Φ6, Φ8, Φ10, Φ12	Φ6, Φ8, Φ10, Φ12	S-300
	Steel:	Φ14, Φ16, Φ20	Φ14, Φ16, Φ20	S-400
Beam	Grade Beam	500x250mm Bottom bar: 3 Φ14 Top bar: 3Φ14 Stirrup: Φ8c/c200	300x250mm Bottom bar: 2Φ16 Top bar: 3Φ14 Stirrup: Φ6c/c150	

	Typical	For first and second floor: Sec A-A 400X200mm Bottom bar:3 Φ14 Top bar: 3Φ14 Stirrup: Φ8c/c200	300X250mm Bottom bar:2 Φ16 Top bar: 3Φ16 Stirrup: Φ6c/c150	
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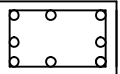
Description		Case Study		Remark
Beam	Typical	Sec B-B 400X200mm Bottom bar:4 Φ14 Top bar: 3Φ14 Stirrup: Φ8c/c190		
Column	Footing	Col 1 and Col2: 200x400mm: 8 Φ20 Stirrup: Φ8c/c200	Col 1,Col2 and C3: 350x350mm: 8 Φ20 Stirrup: Φ8c/c200	
		Col3: 200x400mm: Longitudinal 6 Φ20 Stirrup: Φ8c/c200		
	Typical	Col1 and clo2 200x400mm: Longitudinal 8 Φ20 Stirrup: Φ8c/c200	Col1, clo2 and Col3 350x350mm: 8 Φ16 Stirrup: Φ8c/c200	

Table 4.6: Continued.

Lateral Load profile [25]

$$F_i = \frac{(F_b - F_t) * W_i h_i}{\sum_{j=1}^n W_j h_j} \dots\dots\dots \text{Equation 4-1}$$

$$Fb = S_d(T_1)W \dots\dots\dots \text{Equation 4-2}$$

Where Fb=Base Shear force and W= Total weight of the structure

Table 4.7: Distribution of Base shear force for case Study 01

Floor Level	F _i (KN)
Roof	39.17632
2nd floor	577.7271
1st floor	360.8994
Ground floor	63.7508

Table 4.8: Distribution of Base shear force for case Study 01

Floor Level	F _i (KN)
Roof	83.77709
3nd floor	228.9734
2nd floor	170.3988
1st floor	112.8842
Ground floor	54.67593

Pushover analysis is carried out separately in the X and Y directions.

4.3.2. The target displacement δ_t [4]

Under the Non-linear Static Procedure, a model directly incorporating inelastic material response is displaced to a target displacement, and resulting internal deformations and

forces are determined. The mathematical model of the building is subjected to monotonically increasing lateral forces or displacements until either a target displacement is exceeded, or the building collapses. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake.

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} * g \dots\dots\dots \text{Equation 4-3}$$

where T_e is the effective fundamental period (in seconds) of the building in the direction under consideration, S_a is the response spectrum acceleration (in g) at the effective fundamental period and damping ratio of the building in the direction under consideration; and g is the gravity acceleration.

The correction factors C_0 , C_1 , C_2 and C_3

C_0 = converts SDOF spectral displacement to MDOF roof displacement (based on number of stories given on ATC-40)

C_1 = expected maximum inelastic displacement divided by elastic displacement

C_2 = effects of pinched hysteretic shape, stiffness degradation and strength deterioration (values are given on ATC-40 based on structure performance level.)

C_3 = increased displacement due to dynamic p - Δ effects.

Target displacement for case study 01 and case study 02

The coefficients of target displacements for Case study 01 taken as $C_0=1.20$, $C_1=1.2$, $C_2=1.10$, $C_3=1.00$ and $T_e=0.320$ Sec. For the design level earthquake, the target displacement computed for this structure $\delta_t=0.05$ m and the coefficients of target displacements for Case study 02 taken as $C_0=1.30$, $C_1=1.0$, $C_2=1.10$, $C_3=1.00$ and $T_e=0.50$ Sec. For the design level earthquake, the target displacement computed for this structure $\delta_t=0.10$ m.

5. Results and Discussions

5.1. General

This chapter presents the results of pushover analysis of case study 01 and 02 using SAP2000 software.

5.2. Pushover Curves

Pushover Curves (Base shear versus top displacement) of case study 01 and case study 02. Pushover analysis was carried out separately in the X and Y directions. The resulting pushover curves, in terms of Base Shear – Roof Displacement ($V-\Delta$). The slope of the pushover curves is gradually changed to increase the lateral displacement of the building. This is due to the progressive formation of plastic hinges in beams and columns throughout the structure. The pushover curves reach a maximum which corresponds to the failure of the building. There are many plastic hinges formed with big plastic rotations and the building can no longer sustain them.

5.2.1. Pushover Curves of Case Study 01

Pushover Curve in positive X Direction Loading:

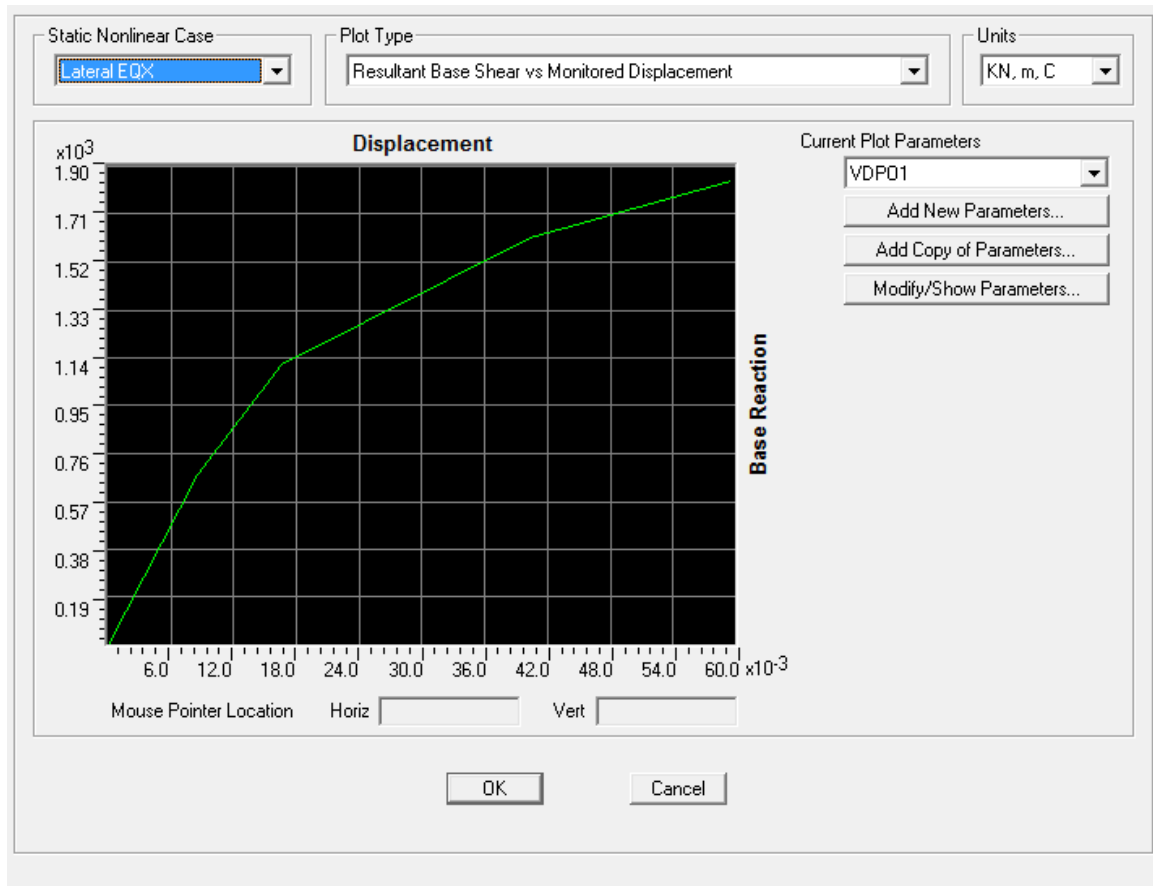


Figure 5:1: The pushover curve positive X direction loading of case study 01

The maximum base shear (V) of the structure in positive X direction is 1850 KN and ultimate roof displacement (Δ) of 0.059 m and linear range up to base shear at 760KN and roof displacement (Δ) of 0.015m. Building with in linear range the hinge state is immediate occupant level which A to B.

Pushover Curve in negative X Direction Loading of Case Study 01:

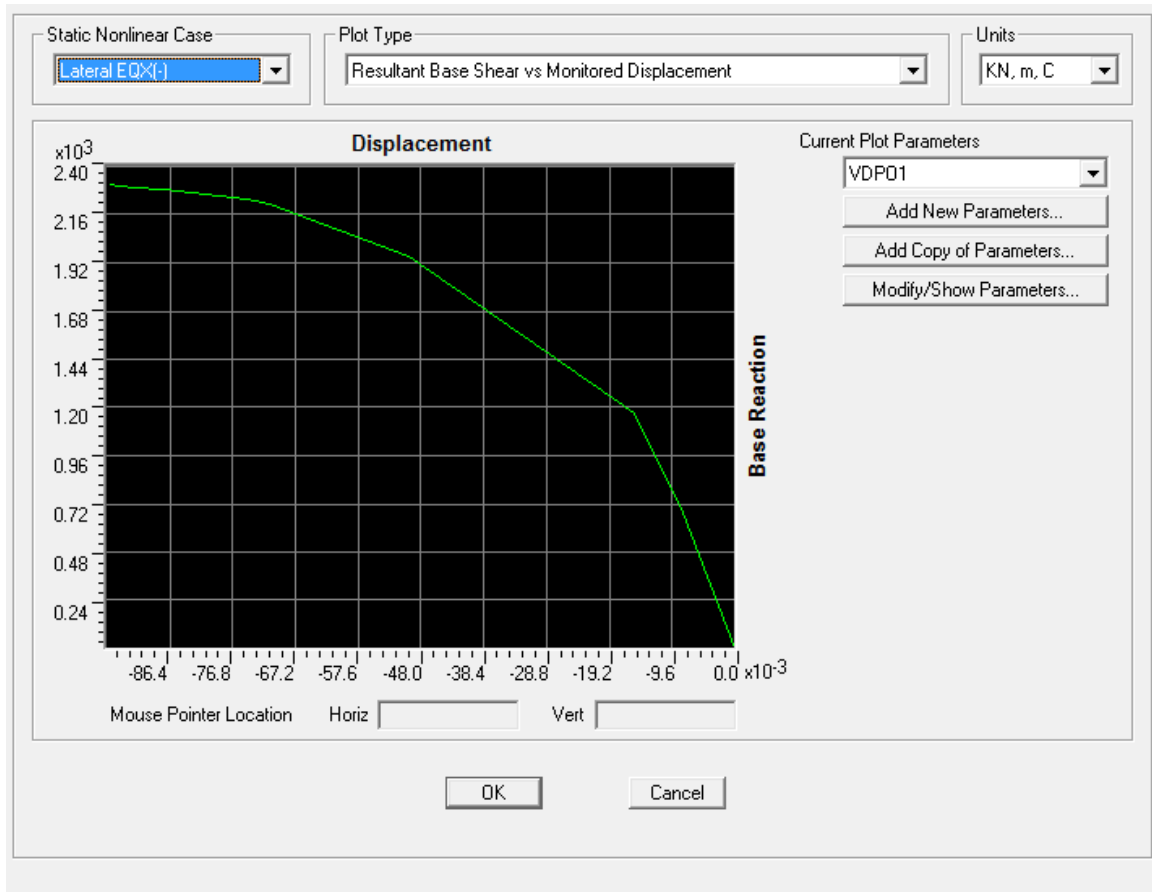


Figure 5.2: Pushover Curve in negative X Direction Loading of Case Study 01

The maximum base shear (V) of the structure in negative X direction is 2340 KN and ultimate roof displacement (Δ) of 0.086 m and linear range up to base shear at 1200KN and roof displacement (Δ) of 0.017m. Building with in linear range the hinge state is immediate occupant level which A to B.

Pushover Curve in negative Y Direction Loading:

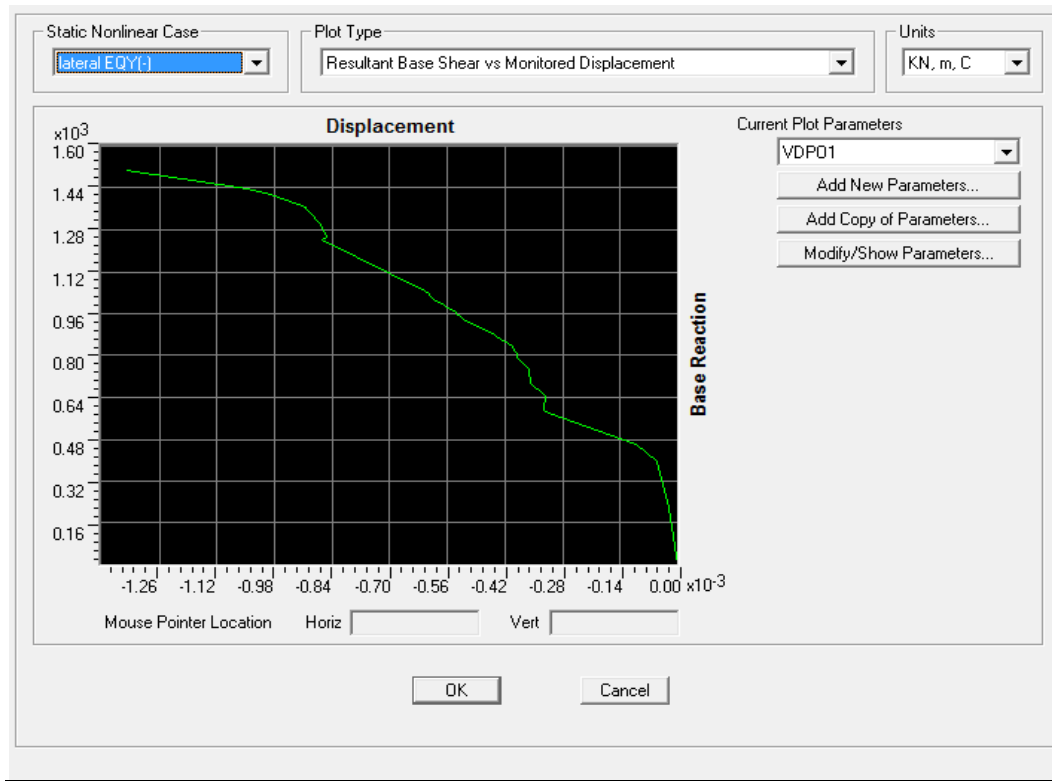


Figure 5:3: The pushover curve for direction –Y of case study 01

The maximum base shear (V) of the structure in negative y direction is 1520 KN and roof displacement (Δ) of $(-1.26 \times 10^{-3} \text{ m})$. Building with in linear range the hinge state is immediate occupant level which A to B.

Final it is observed from the pushover curve of case study 01:

The pushover curves in y direction (longer direction) are being stiffer than the pushover curves in X direction (shorter direction).

This is explained by the fact that in the Y direction more number of moment resisting frames (MRF) exists than X direction. The building at y direction receives similar forces with smaller displacement. Therefore the structure in Y direction has more stiffness as compared to X direction which is more flexible.

Hence, the results of SAP 2000 output matches with actual damage occurred, most nonstructural damage like diagonal cracks in walls, damage of the window is occurring on shorter direction (lesser number of moment resisting frame exists).

5.2.2. Pushover Curve (Base shear versus top displacement) of case study 02

Pushover Curve in positive X Direction Loading:

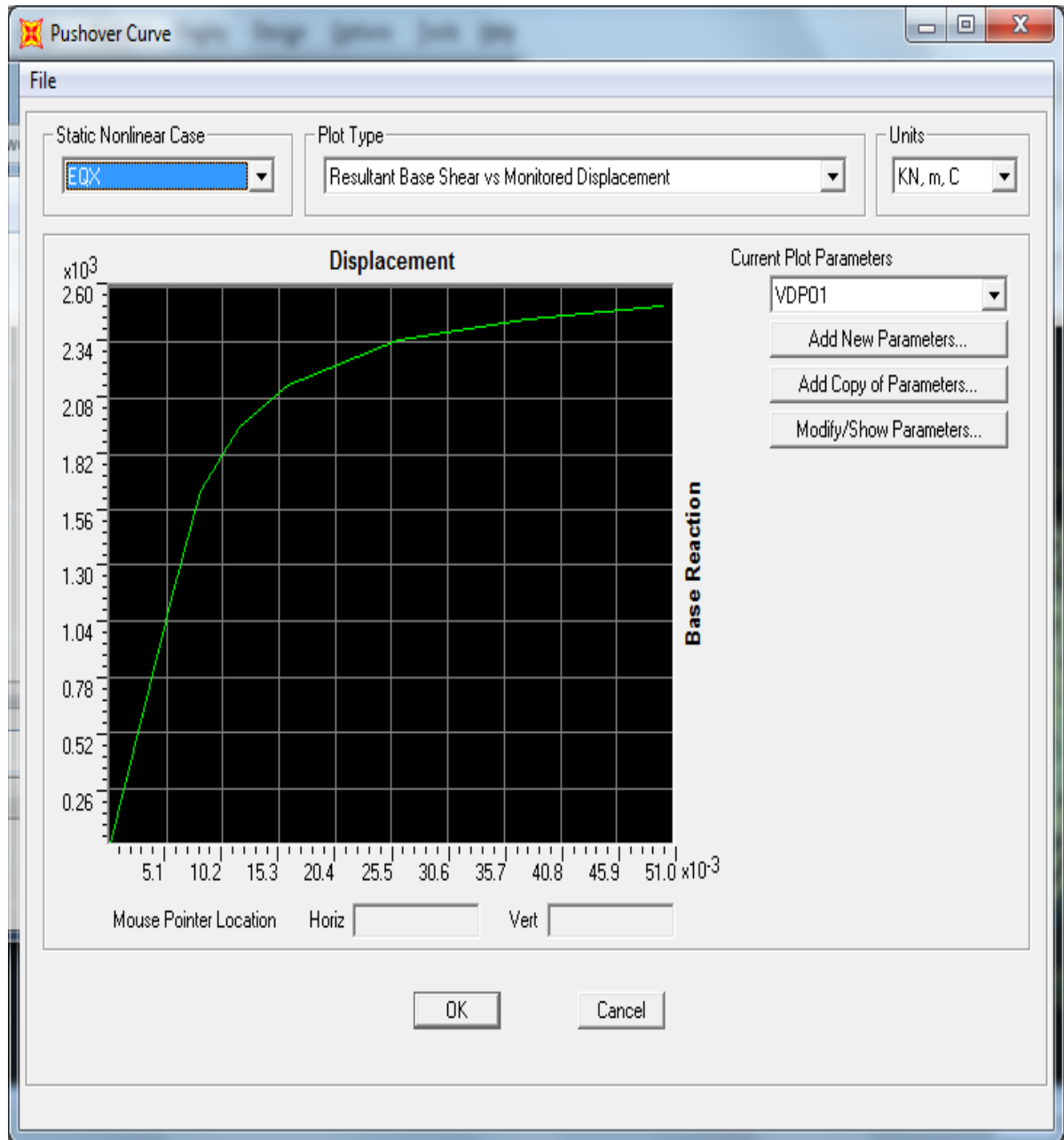


Figure 5:4: The pushover curve) for positive X direction of case study 02

The maximum base shear (V) of the structure in positive X direction is 2450 KN and ultimate roof displacement (Δ) of 0.051 m and linear range up to base shear at 1820KN and roof displacement (Δ) of 0.012m. Building with in linear range the hinge state is immediate occupant level which A to B.

Pushover curve in positive y Direction Loading of case study 02:

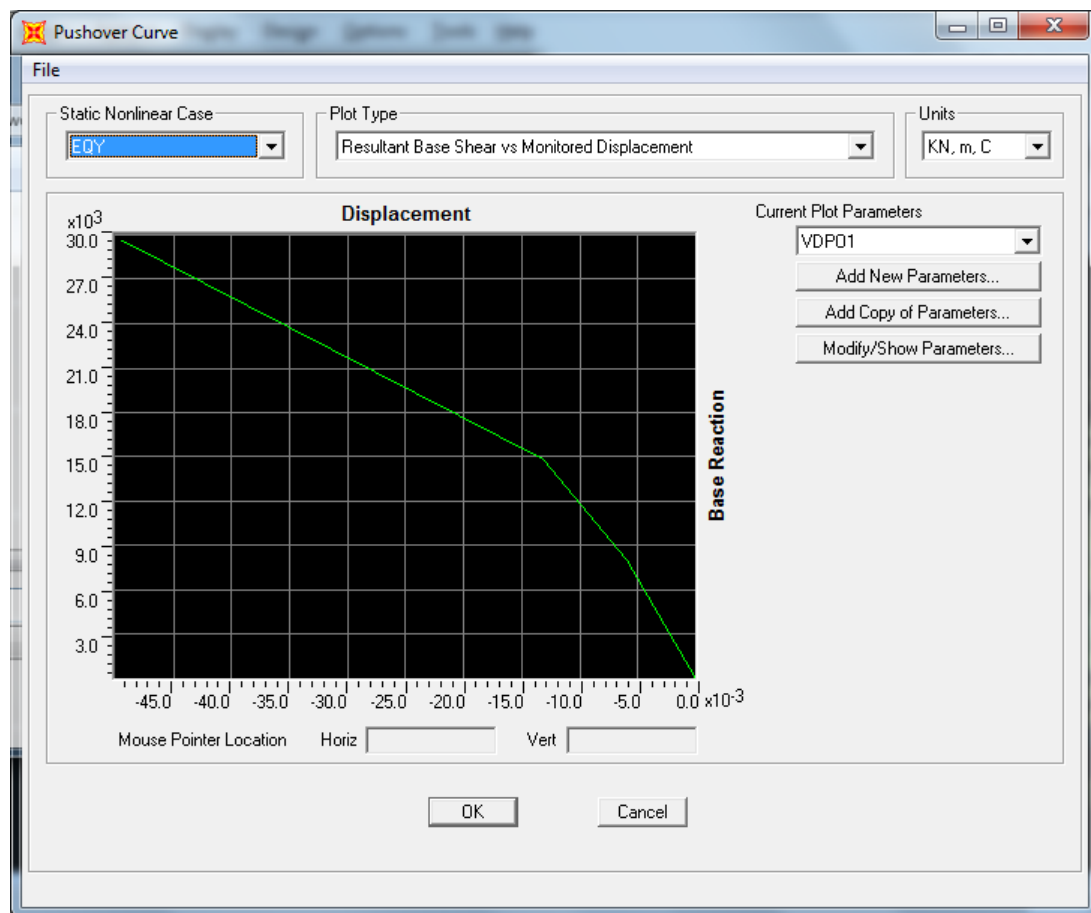


Figure 5:5: The pushover curve) for positive y direction of case study 02

The maximum base shear (V) of the structure in positive y direction is 30000 KN and ultimate roof displacement (Δ) of 0.047 m and linear range up to base shear at 15000KN and roof displacement (Δ) of 0.012m. Building with in linear range the hinge state is immediate occupant level which A to B.

Finally, it is observed from the pushover curve in case study 02, the pushover curve y direction is being stiffer than the pushover curve in x direction. This is explained by the fact that in the y direction has more number of MRF than X direction.

The building in the Y direction receives larger forces with smaller displacement.

SAP2000 result confirms, actual damage occurred which is mostly shorter direction.

5.3. Performance Point and Hinge State

5.3.1. Performance Point and Hinge State of Case Study 01

The performance point is the point where the capacity curve crosses the demand curve according to [4]. Based on this performance point location the damage intensity will be calculated. If this point located in the immediate occupancy the damage intensity is light. If this point lies in the life safety the damage intensity is moderate. Similarly, if performance point lies in collapse prevention then damage intensity is severe. The next figures show the performance point for all pushover curves. With red color is the elastic spectrum of EC8 and a series of reduced responses to ADRS format. The green curve represents the spectrum resistance of the equivalent SDOF system as shown by the resistance curve. With the orange line defined as the locus of points as defined by [4].

Performance point in *Positive X direction loading*:

The performance point of case study 01 building occurred at Step 3 of the analysis.

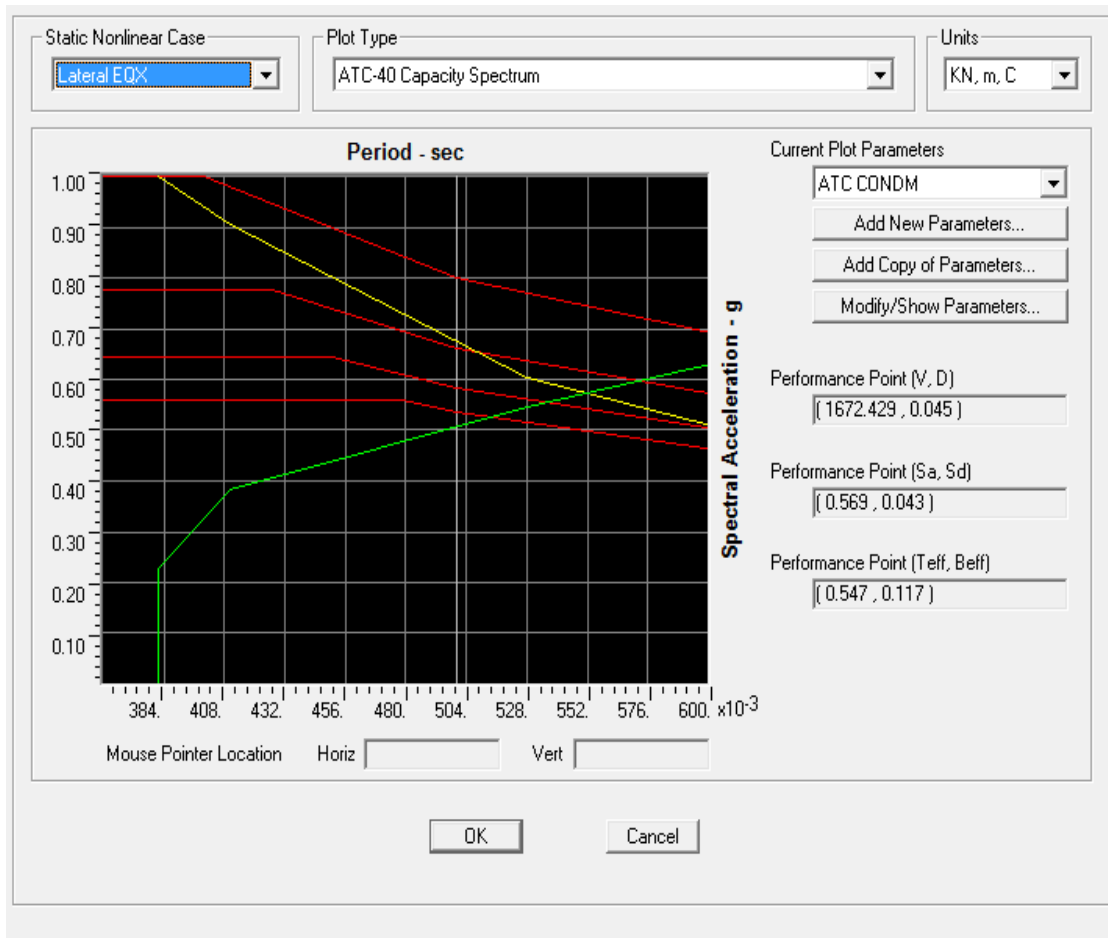


Figure 5:6: Capacity Spectrum, demand Spectrum and parameter of ATC-40 method for positive X direction of case study 01

The curve shows spectral displacement and spectral acceleration. The performance point for a given set of values is defined by the intersection of the capacity curve (green) and the single demand spectrum curve (yellow).

From Figure 4:5, Observed Capacity spectrum and a demand spectrum curve intersection point of case study 01 in X direction.

- Performance point of this structure is occurring at base shear force (V) of 1672.429KN and roof displacement of 0.045m.

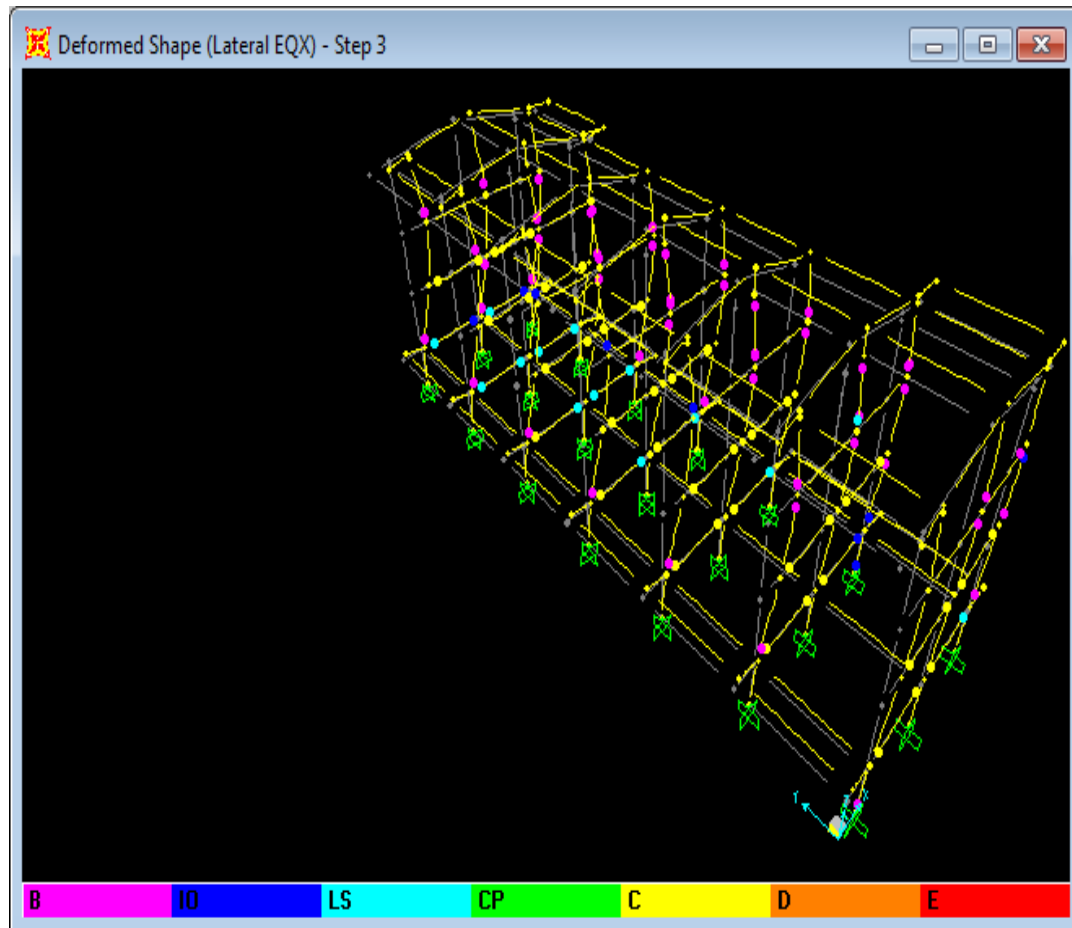


Figure 5:7: Presents the overall yielding pattern of the structure at the Performance point of case study 01 at positive X direction Loading

Table 5.1: Plastic hinge Pattern for pushover analysis at different damage level of case study 01 in positive x direction loading:

The sequence of plastic hinge formation and state of hinge at various levels of building performance can be obtained from SAP2000 output. This gives the information about the weakest member and so the one which is to be strengthened in case of a building need to be retrofitted.

Step	Displa(m)	Base Shear force(KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E
1	0.0085	-673.574	139	1	0	0	0	0	0	0
2	0.0168	-1121.043	178	30	2		1			
3	0.043	-1619.712	141	46	8	12	6	24		

From the table 5.1: can be concluded that most elements have entered in the plastic zone (blue color). Six elements of the building are in the Collapse prevention (CP-C) limit state and 24 elements of building are in collapse limit state (C to D). This means that the building of almost thirty elements requires Strengthening. This also confirms less number of moment resisting frames for X direction loading.

Performance point Due to Positive Y direction loading of case study 01

This analysis was completed in 34 steps and performance point was set Steps 22 of the analysis. The performance point S_d is equal to 0.076 m.

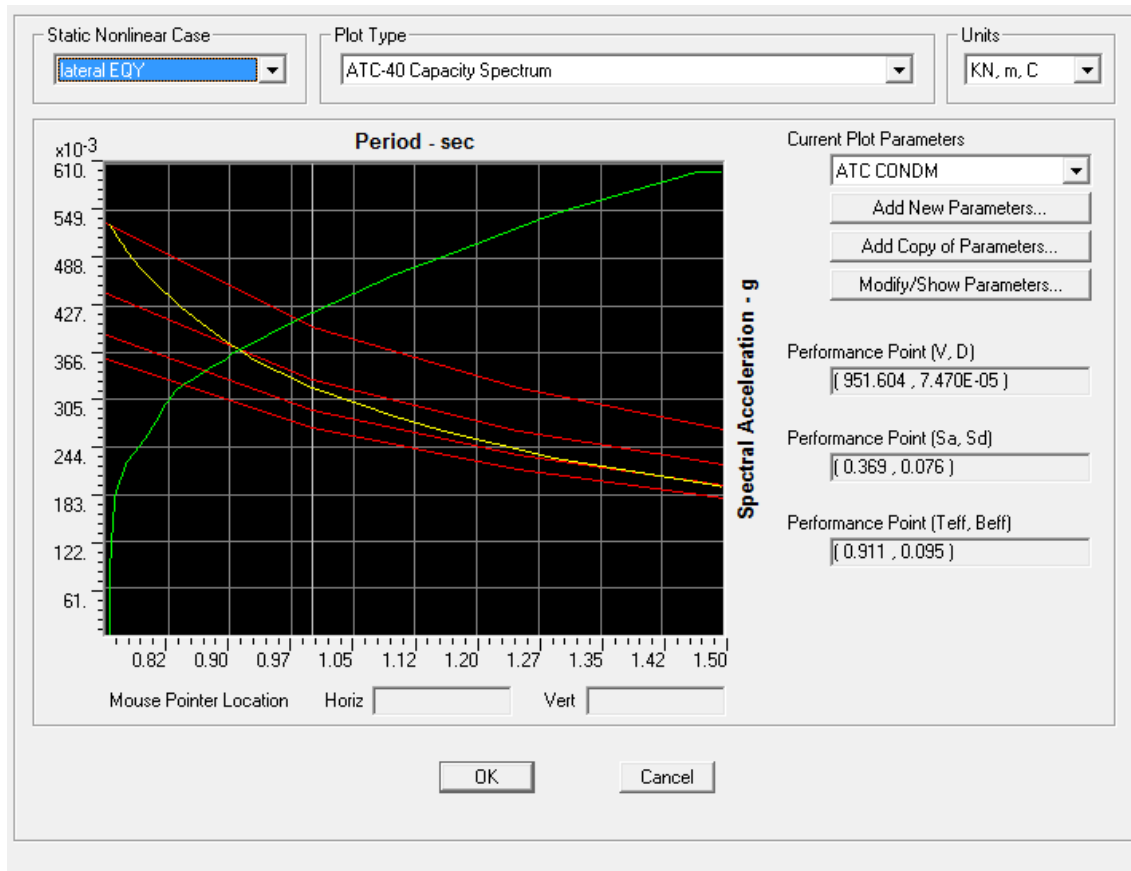


Figure 5:8: Capacity Spectrum, demand Spectrum and parameter of ATC-40 method for +y direction of case study 01

From 4:7 Observed Capacity spectrum and demand spectrum curve intersection point of case study 01 in Y direction.

Performance point for this structure is occurred at base shear force of 951.604KN and roof displacement 7.47×10^{-5} m.

Figure 5:9: presents the overall yielding pattern of the structure at the Performance point of Case study 01 y direction Loading

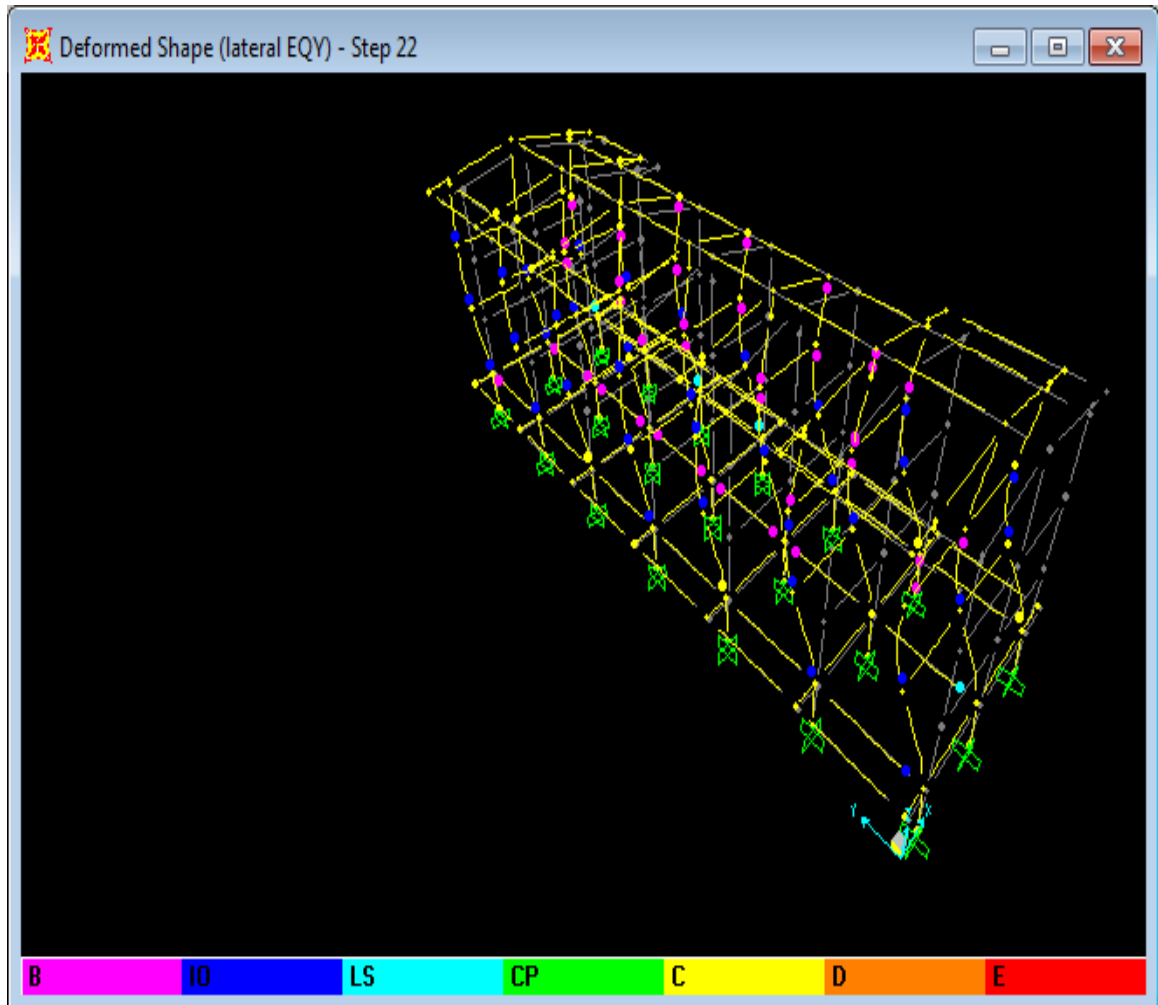


Table 5.2: Elements States at Performance Step and other Steps of case study 01 Y direction loading:

Step	Displa(m)	Base Shear force(KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E
21	0.05	1954.13	200	9	2					
22	0.103	2247.394	155	29	32	4	2	4		

From the above can be concluded that most elements have entered in the plastic zone (blue color). Two elements of the building are in the Collapse prevention (CP-C) limit state and 4 elements of building are in collapse limit state (C to D). This means that the building of almost six elements requires Strengthening. This means that the building almost safe.

In addition to pushover capacity curve, capacity-demand curve confirms weaker elements exists in x direction loading.

5.3.2. Performance Point and Hinge State of Case Study 02

Performance point Due to Positive X direction loading of case study 02

The performance point of case study 02 building occurred at Step 3 of the analysis.

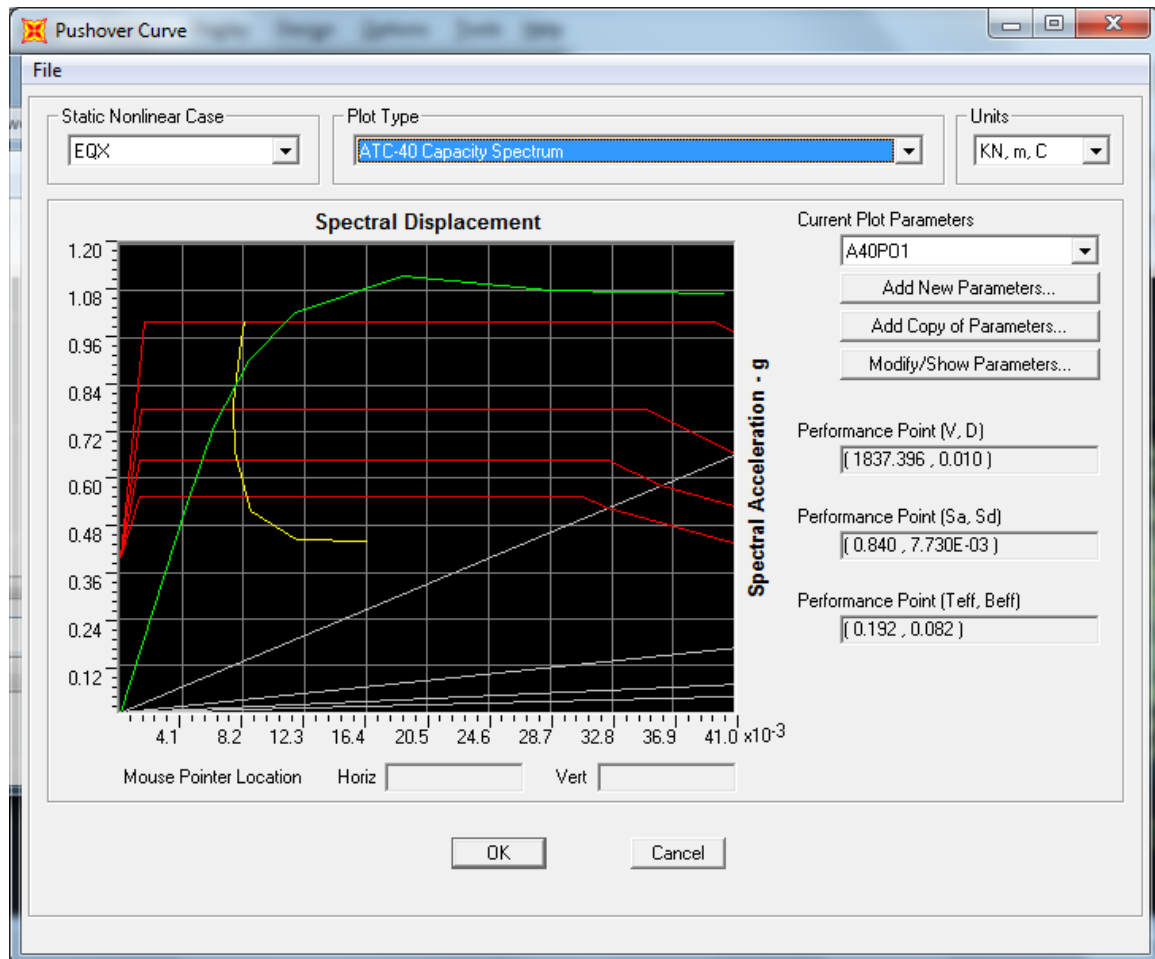


Figure 5:10: Capacity Spectrum, demand Spectrum and parameter of ATC-40 method for positive X direction of case study 02:

From figure 5.10: Observed Capacity spectrum and demand spectrum curve intersection point of case study 02 in X direction.

- Performance point for this structure is occurred at base shear force of 1837.396KN and roof displacement 0.01 m.

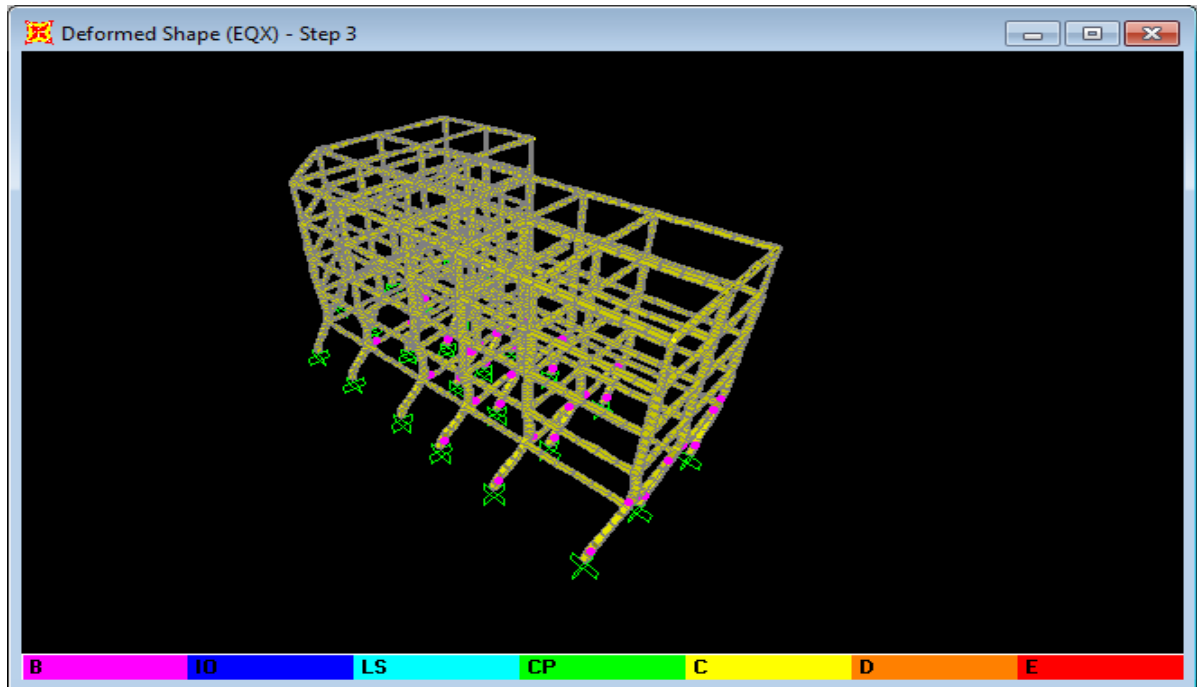


Figure 5.11: presents the overall yielding pattern of the structure at the Performance point of case study 02 X direction Loading.

Table 5.3: Computed limit states for the case 02 studied building in +X direction case study 02.

Step	Displa(m)	Base Shear force(KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E
1	0.0056	-1172.002	210.5	1						
2	0.0082	-1645.848	201.5	9.5						
3	0.0117	-1951.269	192	18.5						
4	0.016	-2144.685	182.5	27	3					
5	0.0262	-2354.102	171	25.5	14.5					
6	0.0378	-2449.407	166	15	30					
7	0.0499	-2513.364	165	7.5	30	14				

From the above can be concluded that most elements have entered in the plastic zone (blue color). All elements are within the immediate occupant level.

Performance point Due to Positive y direction loading of case study 02

The performance point of case study 01 building occurred at Step 3 of the analysis.

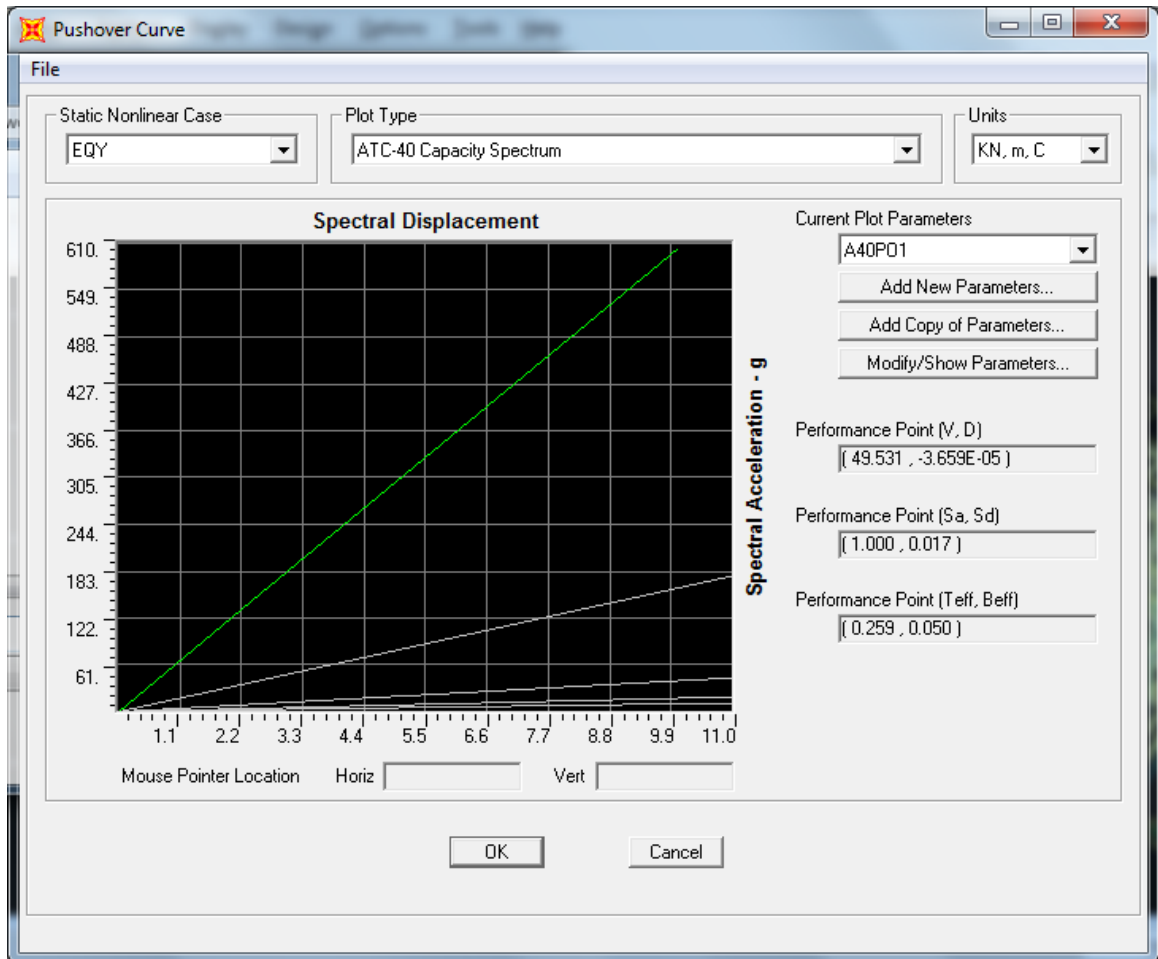


Figure 5:12: Capacity Spectrum, demand Spectrum and parameter of ATC-40 method for +y direction case study 02

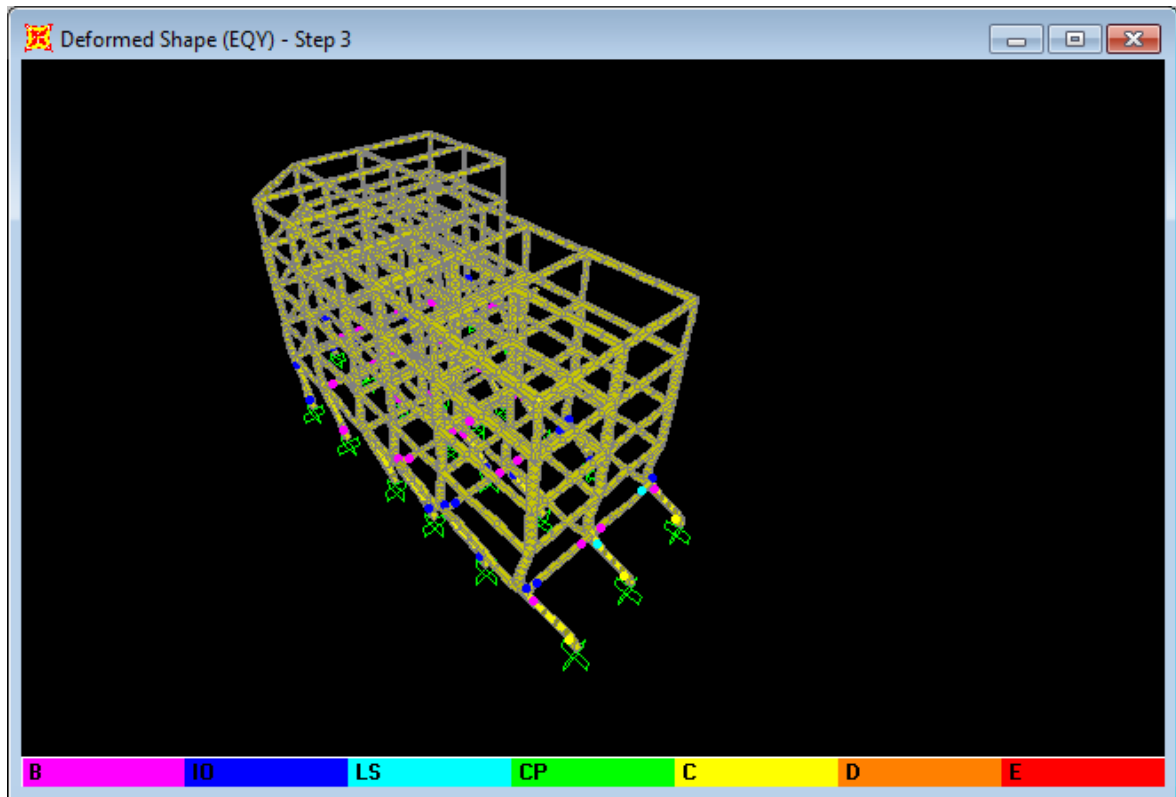


Figure 5:13: presents the overall yield pattern of the structure at the Performance point case study 02

Table 5.4: Elements State at performance point of case study 02

Step	Displa(m)	Base Shear force(KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E
1	-0.0307	-8067.663	210.5	1						
2	-0.0131	14799.745	199	12						
3	-0.0496	29750.064	177.5	29	5	1				

From the above can be concluded that most elements have entered in the plastic zone (blue color). Only one element is within LS to CP limit state.

6. Conclusions

6.1. CONCLUSIONS

Seismic evaluation of five buildings constructed in Hossana town was conducted by using both qualitative and quantitative method. The qualitative and quantitative is based on the FEMA and EMS while the quantitative method is nonlinear static (pushover) analysis performed using SAP2000 software as per ATC 40 life performance criteria.

The main conclusions drawn from the study can be summarized as:

- Case study 02 building has suffered only minor non structural damage from the actual earthquake .Both qualitative and quantitative analysis results also confirm with observation.
- Case study 01 building has relatively more damages than case study 02. The damages were mainly on the infill walls located in the longer direction. This observation was also confirmed the quantitative analysis with formation of more number of plastic hinges in the frames of the longer direction.
- The pushover analysis indicates the case study 01 should not satisfy life safety level. It can be concluded that the building may not withstand further earthquake and hence need strengthening of vulnerable members.

6.2.RECOMMENDATIONS

- It is recommended that the Ethiopian building code standards include provisions for seismic evaluation guidelines.
- Finally the author suggests that such a study shall further be conducted on a large number of buildings located in Hosanna town and other towns, in order to get a detailed and realistic existing state of the buildings.

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References

1. A. Shuraim, A. Charif. Performance of pushover procedure in evaluating the seismic adequacy of reinforced concrete frames. King Saud University ashuraim@gmail.com. (2007)
2. Whittaker A, Constantnou M, Tsopelas P. Displacement estimates for performance-based seismic design. J Structure Div, ASCE; 124 (8):905–912, (2007)
3. SAP2000 “Three Dimensional Static and Dynamic Finite Element Analysis and Design of Structures”, University of California, Berkeley, CA, 2001. (6 perform) FEMA273 “Federal Emergency Management Agency”, recommended Provisions for Seismic Regulations for New Buildings and Other Structures
4. ATC 40 “Applied Technology Council, Seismic Evaluation and Retrofit of Concrete Buildings”, Volume 1 Report, Redwood City, California, 1996.
5. FEMA 356 “Federal Emergency Management Agency”, Pre standard and commentary for the seismic rehabilitation of buildings.
6. (2000).CSI “Analysis Reference Manual for SAP2000, ETABS and SAFE – Computers and Structures, Inc”, Berkeley, California, USA, October 2005.
7. Vision 2000 committee “performance based seismic engineering of buildings”, structural Engineers Association of California (SEAOC), California
8. Saiidi M. and Sözen M.A., 1981, Simple Nonlinear Seismic Response of R/C Structures, Journal of Structural Division, ASCE, Vol. 107, 937-952.

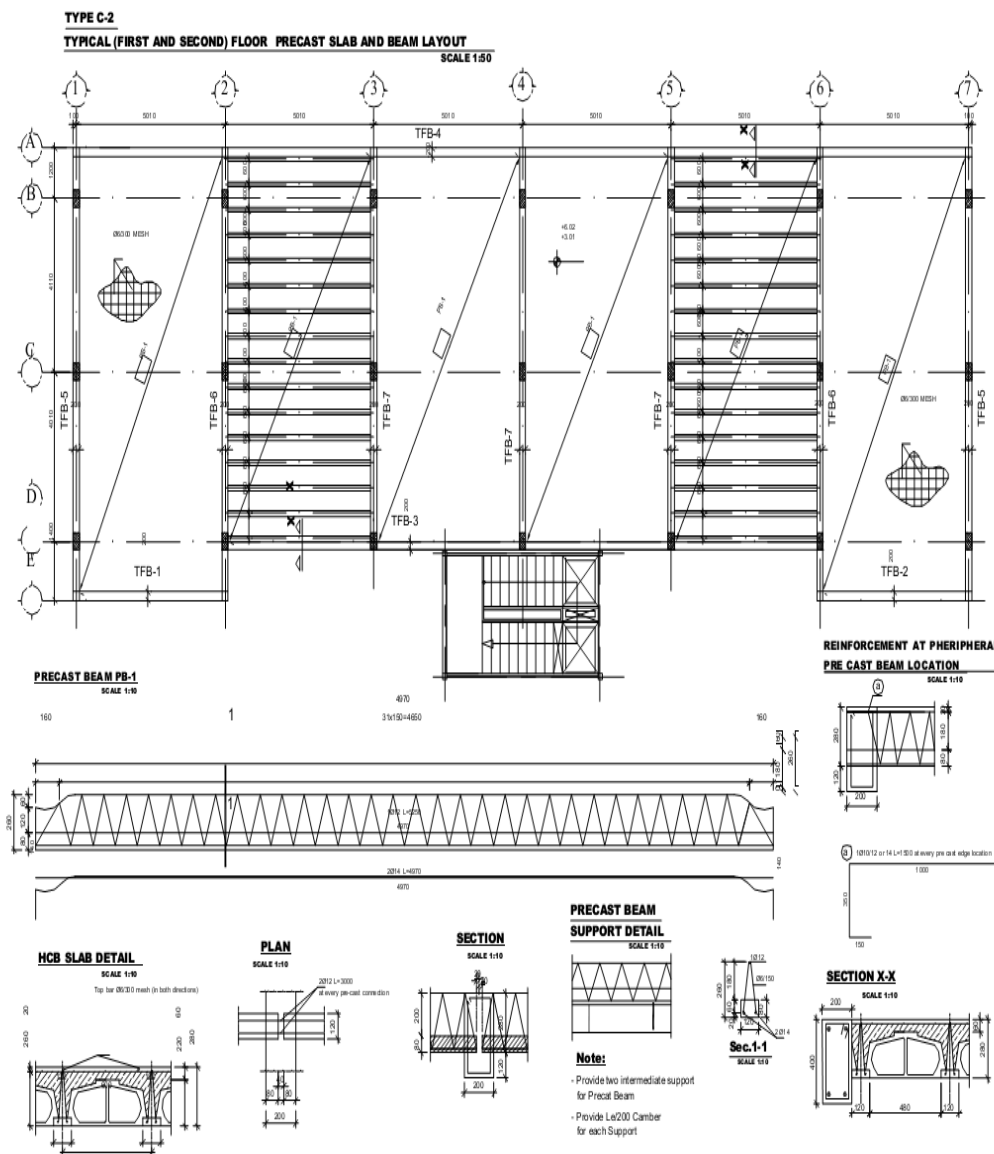
9. Albanesi, T., Nuti, C and Vanzi, I. (2002), "State of the Art Review for the Nonlinear Static Methods", Proceedings of 12th European Conference on Earthquake Engineering, London, Paper No.602, pp. 1-10
10. EL-Esnawy, N.A. (2004), "Evaluation of Inelastic Deformation Spectra for Performance-Based Seismic Design", Proceedings of Third Egyptian Conference on Earthquake Engineering, Cairo, Egypt, pp. 122-136.
11. Fajfar, P. (2000), "A Nonlinear Analysis Method For Performance Based Seismic Design", Earthquake Engineering and Structural Dynamics, Vol. 16, No. 3, pp. 573-592
12. Fajfar, P. and Fischinger, M. (1988), "N2-A Method for Nonlinear Seismic Analysis of Regular Structures," Proceedings of Ninth World Conference on Earthquake Engineering, Tokyo-Kyoto, Japan, Vol. 5, pp. 111-116.
13. Krawinkler, H. and Seneviratna, G.D.P.K. (1998), "Pros and Cons of a Pushover Analysis of Seismic Performance Evaluation", Journal of Engineering Structures, Vol. 20, No. 4-6, pp. 452-464.
14. Mwafy, A.A. (2001), "Seismic Performance of Code-Designed RC Buildings", Ph.D.Thesis, Department of Civil and Environmental Engineering, Imperial College, London.
15. . Rodriguez, M.E., Restrepo, J.I., Carr, A.J. (2002), "Earthquake-Induced Floor Horizontal Accelerations in Buildings", Earthquake Engineering and Structural Dynamics, Vol. 31, No. 3, pp. 693-718.

16. Sasaki, K.K., Freeman, S.A. and Paret, T.F. (1998), "Multimode Pushover Procedure (MMP) — A Method to Identify the Effects of Higher Modes in a Pushover Analysis," Proceedings of 6th U.S. National Conference on Earthquake Engineering, Earthquake Engineering Research Institute, Seattle, Washington, Paper No. , pp. 1-6.
17. Vamvatsikos, D. and Cornell, C. A. (2002), "Incremental Dynamic Analysis", Earthquake Engineering and Structural Dynamics, Vol. 31, No. 3, pp. 491-514
18. Durgesh C Rai (2011)," Review of documents on Seismic Evaluation of Existing Buildings", IITK, GSDMA-EQ03-V1.0, PP1-10
19. G. GRUNTHAL (1998),"European, Macro seismic Scale 1998", Luxembourg 1998, PP16
20. Kadid.A and Boumrkik .A," Pushover analysis of reinforced concrete framed structures ", Asian journal of Civil engineering (building and housing) Vol.9,no.1,2008
21. FEMA(1998),"FEMA 310-Handbook for the Seismic Evaluation of Buildings-A prestandard', Federal Emergency Management Agency, Washington DC,USA
22. Sermi Oguz. A thesis on "EVALUATION OF PUSHOVER ANALYSIS PROCEDURES FOR FRAME STRUCTURES", April, 2005. Middle East Technical University

23. X.-K. Zou, C.-M. Chan. Optimal seismic performance-based design of reinforced concrete buildings using nonlinear pushover analysis. Department of Civil Engineering, Hong Kong University of Science and Technology, Kowloon, Hong Kong, China. Available online 10 May 2005.
24. FEMA 154 “Federal Emergency Management Agency”, Rapid Visual Screening of Buildings for Potential Seismic Hazards, march 2002
25. Ethiopian Building Code Standard: EBCS-8 Design of Structures for Earthquake Resistance. (1995) Ministry of Works and urban Development, Addis Ababa.

APPENDIX A-Case Study Structural Drawings

Case Study 01



NOTES:
• All dimensions are in mm
• This drawing should be read in conjunction with architectural drawing

TYPE-C2

**ADDIS ABABA
UNIVERSITY SCHOOL
OF GRADUATE**

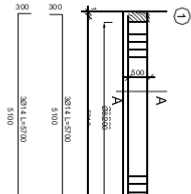
**SNIPPLS HOUSING
DEVELOPMENT PROJECT OFFICE**

**TYPICAL FLOOR SLAB LAYOUT,
PRECAST BEAM AND SECTIONS**

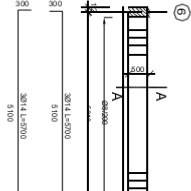
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TYPE C-2
GROUND FLOOR BEAMS
SCALE 1:50

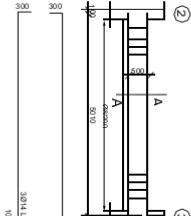
GFB.1 Ground Floor Beam 1-2 on axis E



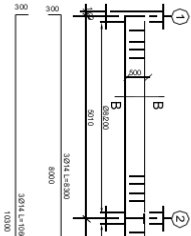
GFB.2 Ground Floor Beam 6-7 on axis E



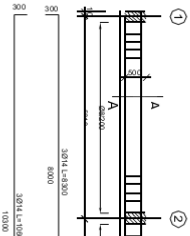
GFB.3 Ground Floor Beam 2-3-4-5-6 on axis D



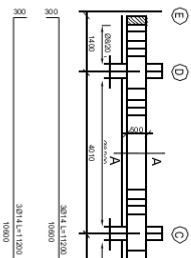
GFB.4 Ground Floor Beam 1-2-3-4-5-6-7 on axis C



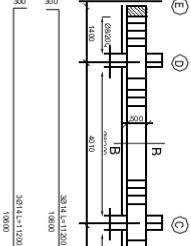
GFB.5 Ground Floor Beam 1-2-3-4-5-6-7 on axis A



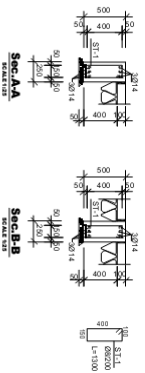
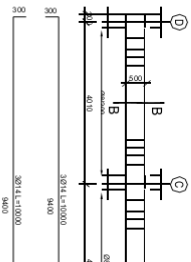
GFB.6 Ground Floor Beam E-D-C-B-A on axes 1 and 7



GFB.7 Ground Floor Beam E-D-C-B-A on axes 2 and 3



GFB.8 Ground Floor Beam D-C-B-A on axes 3, 4 and 5



NOTES:
- All dimensions are in mm.
- This drawing should be used in conjunction with architectural drawings.

TYPE-C2

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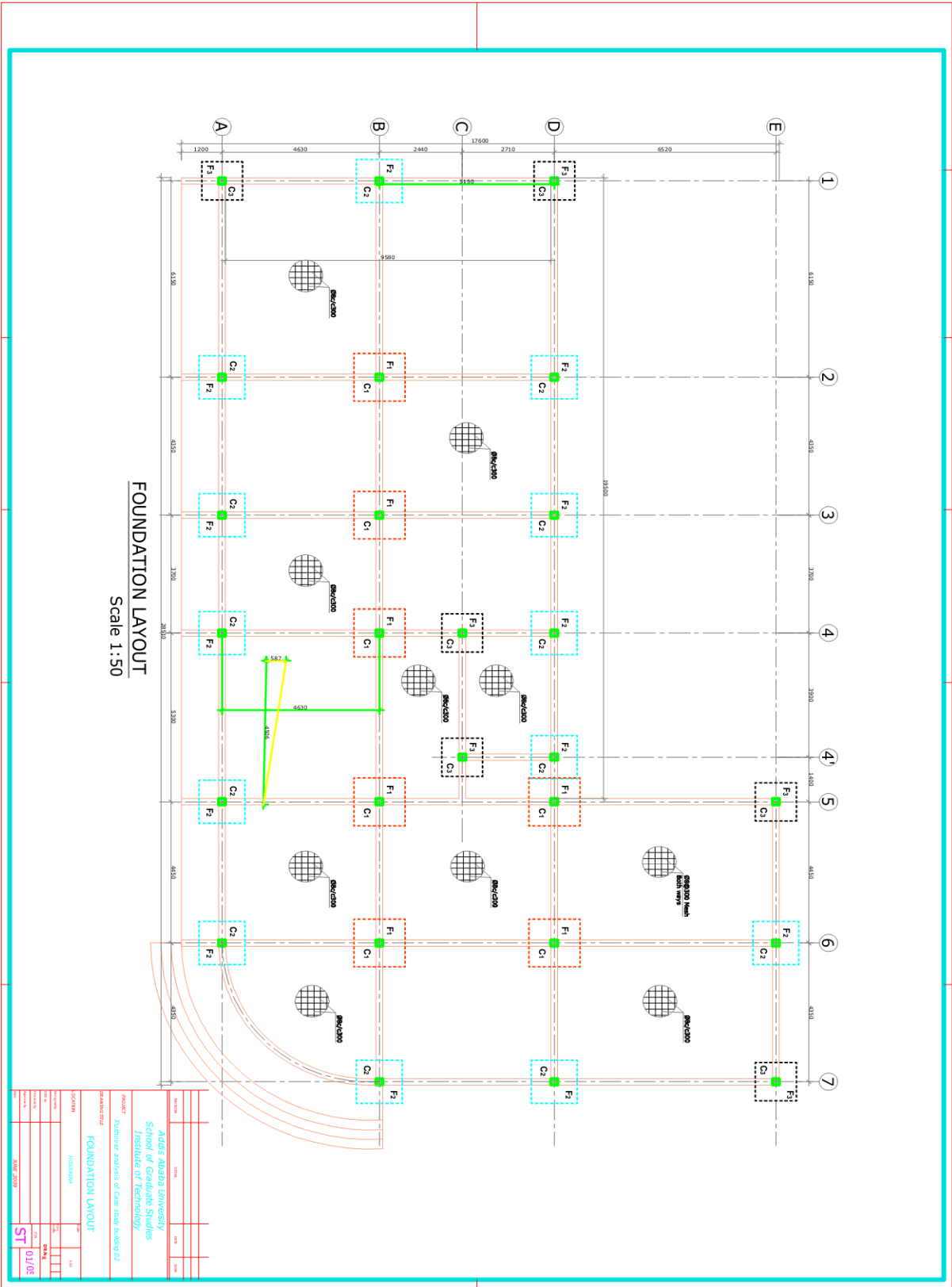
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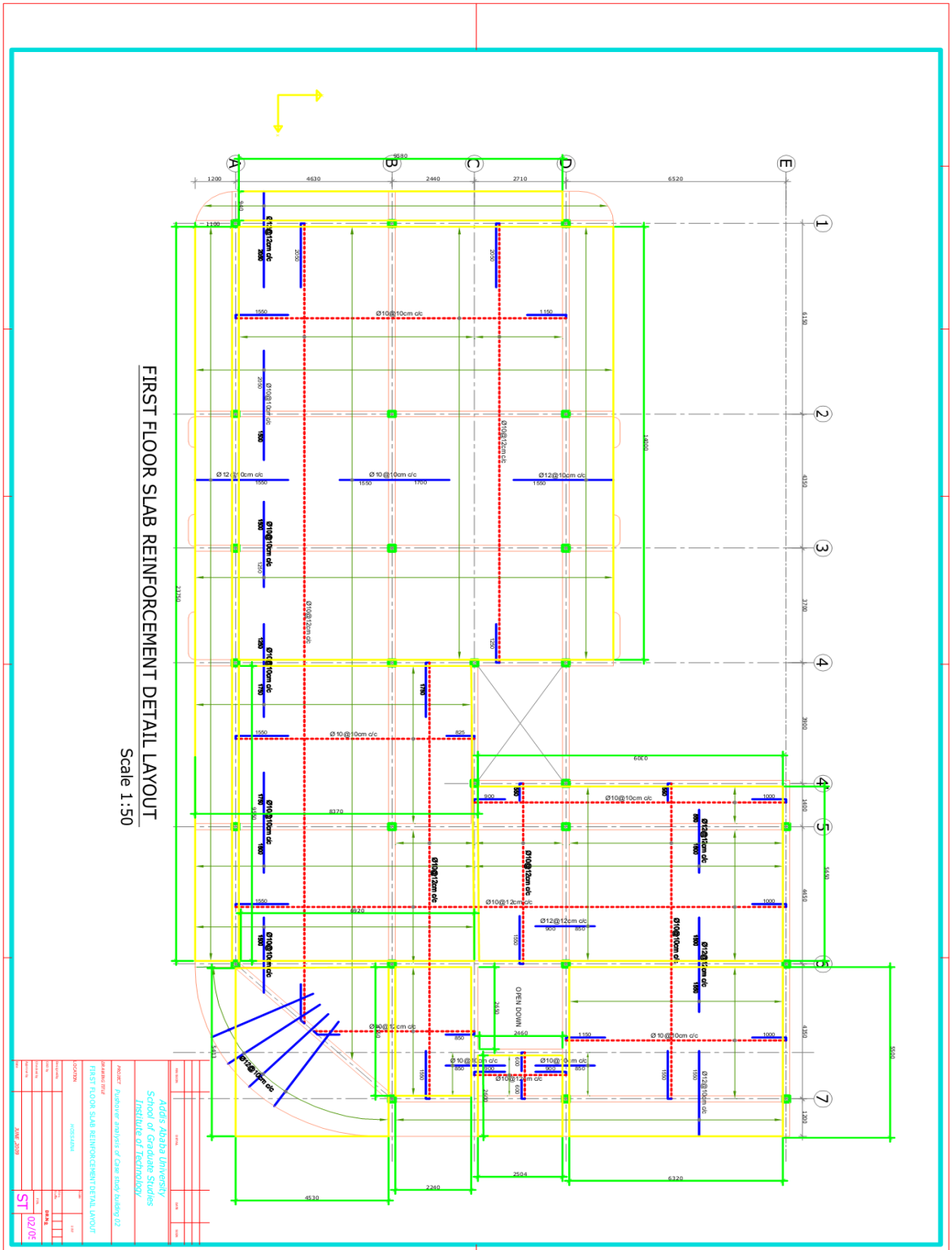
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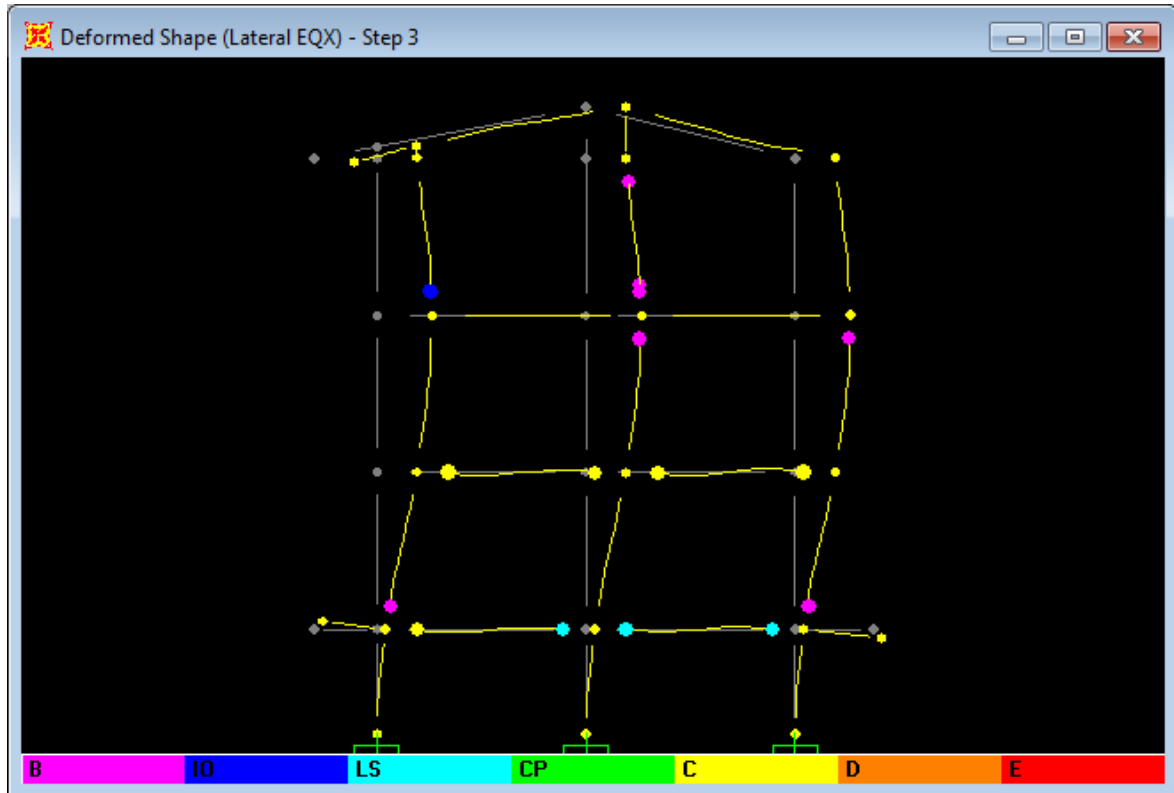




Appendix B –Hinge state of Case Study Buildings

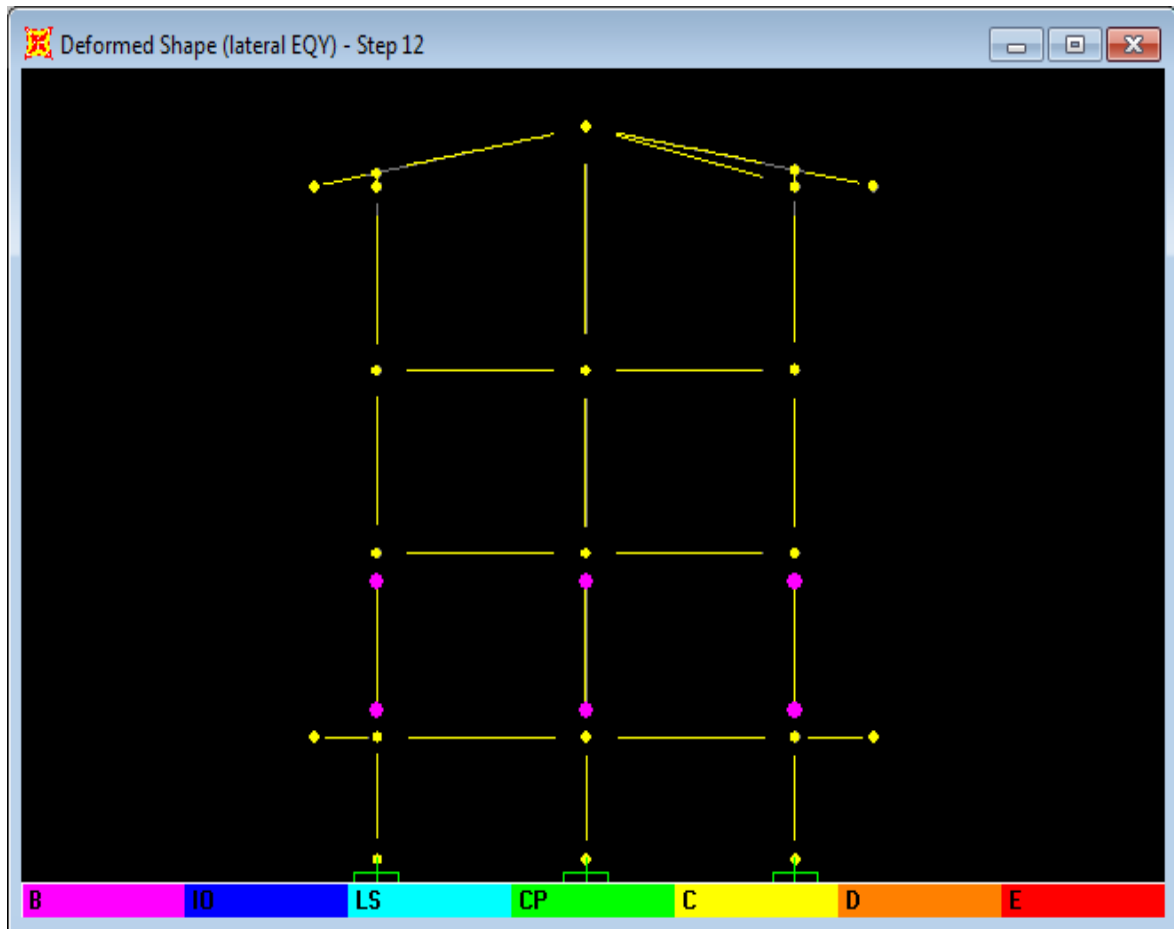
Case Study 01

X direction loading

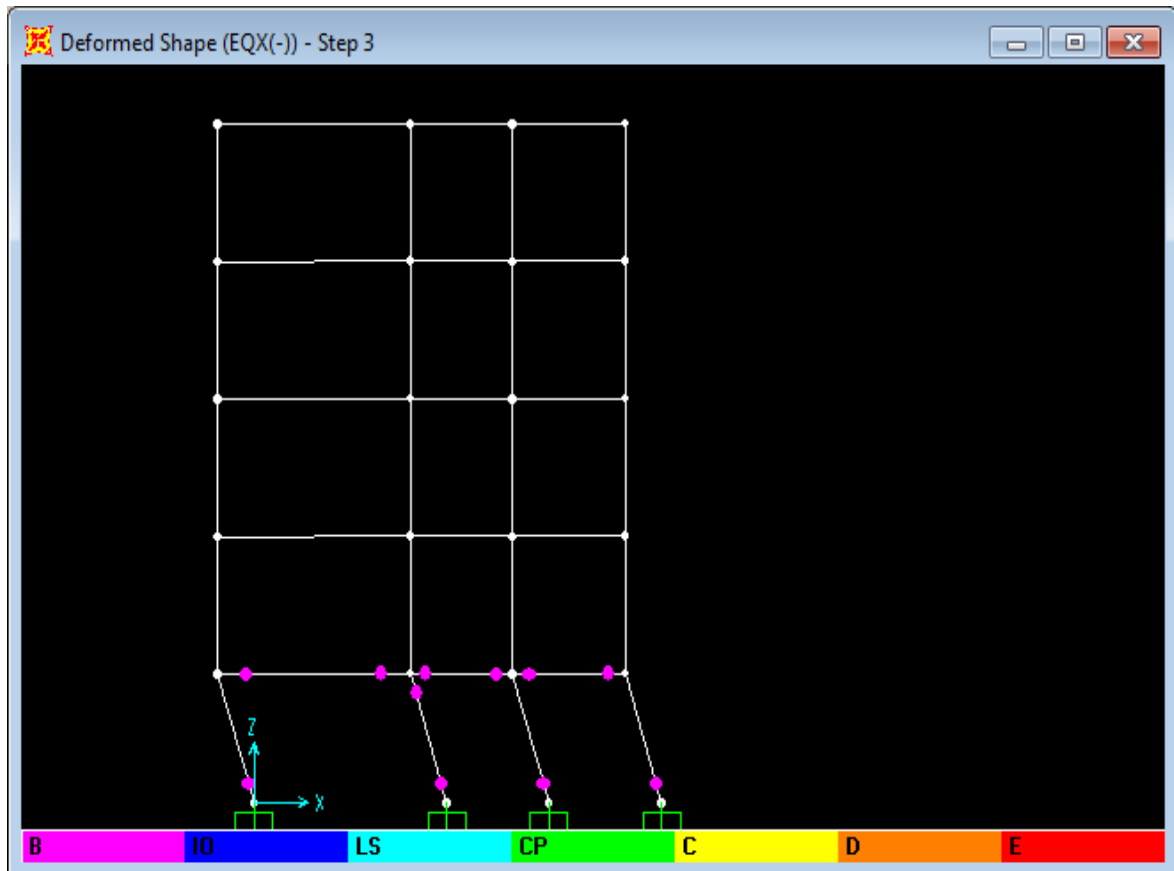


Hinge state at X direction loading on Axis 4

Case Study 01 Y-Direction Loading



SAP 2000 Hinge state of case study 02



Appendix C- PUSHOVER ANALYSIS STEPS

Step-1 Linear Analysis & Unlock the Model

- Design the building as per linear analysis using expected strength and modified partial safety factors.
- After completing the model of the building Unlock the model as shown in Figure

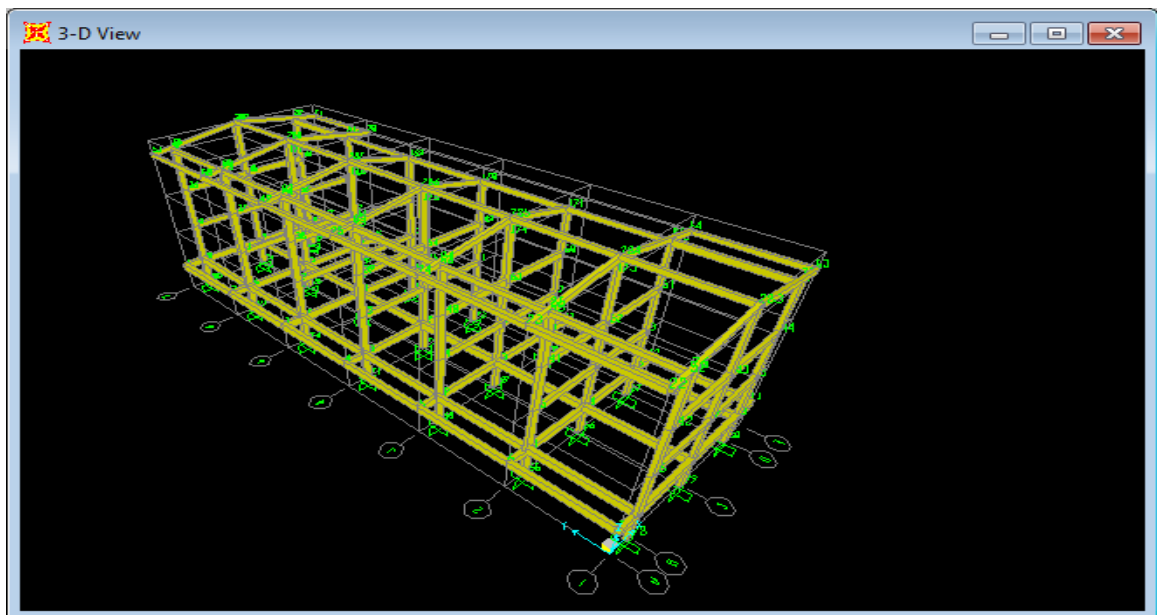


Figure B.1 Unlock the model form

Step-2 Define Load Case of Gravity Loads

- Go to **Define > Load Case> Add New Load Case** consisting of Gravity loads (i.e. dead load and % of live load).

Step-3 Assignment of Hinges to Frame Elements

- Select all the beams in the model. Go to **Assign > Frame > Hinges**
- The hinges should be assigned at both the ends which means at the relative distance of 0 and 1.
- In similar manner assign hinges to all columns by repeating steps as previously Carried out for beams the only difference is that column should be assigned P-M2-M3 hinges instead of M3 hinges
- The SAP 2000 non-linear pre-defined non-linear hinge properties corresponding to Caltrans hinge model and FEMA 356 hinge model

Step-4 Define PUSHOVER load Case

- Go to **Define > Load Case> Add New Load Case >PUSH** consisting of load in proportion to the fundamental mode. This load case is deformation controlled load.
- Select **Load Case Type> Static, Analysis Type> Nonlinear** and Geometric Non-linearity Parameters as P-Delta.
- This load case should be started from a previous load case Gravity since gravity load will always be acting on the structure.
- Load Application should be Displacement control with Monitored Displacement. Generally, the monitored displacement is kept equal to 2% of the height of the building.

Step-5 RUN Analysis

- While running the analysis it is important to Run the Modal and Gravity analysis with pushover load case since Pushover load case takes stiffness from gravity load case and mode shape from Modal load case
- The pushover analysis is a non-linear static analysis so depending upon system configuration it takes time to complete the analysis

Step-6 Graphically Review the Pushover Analysis Results

- To see the pushover curve **Go to > Display > Show Static Pushover Curve**, the display static pushover curve
- The Static Pushover Curve Go to > File > Display Tables it will show the results of Pushover analysis in Tabular Form.