JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIROMENTAL ENGINEERING

STRUCTURAL ENGINEERING STREAM

SEISMIC PERFORMANCE OF STIFFNESS IRREGULAR MOMENT RESISTING FRAME DESIGNED FOR HIGH STRENGTH CONCRETE

A thesis submitted to School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Structural Engineering.

BY: YADATA BEKELE BALCHA

MARCH, 2022

JIMMA, ETHIOPIA

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SEISMIC PERFORMANCE OF STIFFNESS IRREGULAR MOMENT RESISTING FRAME DESIGNED FOR HIGH STRENGTH CONCRETE

YADATA BEKELE BALCHA

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ABSTRACT

Damage observed after previous earthquakes indicates that a large number of existing buildings are vulnerable to seismic hazard. The damage in the structures generally initiates at location of the structural weak planes present in the building systems. These weaknesses trigger further structural deterioration which leads to the structural collapse. The weaknesses often occur due to presence of the structural irregularities in stiffness, strength and mass in a building system. Buildings with irregular configurations demonstrated more vulnerability in the past during earthquakes. It has been observed after previous earthquakes that the buildings having irregularities have higher seismic demands. The structural irregularity can be broadly classified as plan and vertical irregularities.

The focus of the present study is to assess the performances of soft storey at first storey level (stiffness irregular) buildings designed for normal strength concrete (NSC) and high strength concrete(HSC). A building is said to be stiffness Irregular (Soft Story) when there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above. Accordingly in this study the average stiffness of first storey are 68.75 % and 69.14% of the storey above it for NSC-SI and HSC-SI respectively.

For the purpose of this study four 12- Storey moment resisting frames (MRF) building were designed and detailed using CSI ETABS v18.1 according to new Ethiopian building code ES EN 1998-1 2015 guidelines. The study program grouped into two: One with ***regular configuration*** *designed for both* ***NSC*** *and* ***HSC*** *and the other group as* ***soft storey at first storey*** *level designed for both* ***NSC*** *and* ***HSC****. Following the design and detailing, Seismic performance assessment were conducted using nonlinear static pushover analysis and Incremental dynamic analysis by Sesimostruct (2018) software.*

Static pushover analysis indicates that the ultimate base shear capacity is increased by 2.88% for HSC-R, decreased by 20.37% and 13.55% for NSC-SI and HSC-SI respectively when compared to NSC-R. The value of roof displacement at ultimate base shear capacity is reduced by 13.92%, 5.65% and 14.11% for HSC-R, NSC-SI and HSC-SI respectively compared to NSC-R. Similarly, the results from IDA shows the relative increase in ultimate base shear capacity by 6.65% for HSC-R, 23.97% decrease for NSC-SI and 7.018% decrease for HSC-SI compared to NSC-R. Roof drift at ultimate base shear capacity is decreased by 8.71% for HSC-R, 1.68% increase for NSC-SI and 13.8% increase for HSC-SI in comparison to NSC-R.

The study is extended to the seismic vulnerability analysis using the result from IDA analysis as the input for seismic fragility analysis. These curves depict probability of exceeding limit state capabilities identified as immediate occupancy (IO), life safety and collapse prevention limit states under different levels of seismic intensity.

The fragility analysis indicates that the NSC-R moment resisting frame shows 90.01%, 77.73% and 43.39% probability of exceeding the IO, SD and NC performance levels respectively. For HSC-R, it was observed that 84.54%, 81.53% and 59.29% probability of exceedance of IO, SD, and NC performance level of exceedance. Regarding the stiffness irregular (soft-story) MRF, the results indicates that 96.34%, 85.04% and 53.72% of probability of exceedance of IO, SD and NC respectively whereas, for HSC-SI the probability of exceedance of IO, SD and NC were 92.92%, 91.12% and 77.37% respectively.

The results from both pushover analyses and Incremental Dynamic analysis shows that collapse shear capacity increase slightly over wide ranges of concrete strength. Increase in concrete strength does not improve seismic performance to desired level in seismic zone as structural member fail early prior reaching their full capacity due brittleness nature of the concrete. Fragility analysis further confirms that the frames designed for high strength (HSC) perform generally well at immediate occupancy (IO) performance level whereas, frames designed for normal strength concrete (NSC) are relatively better at Significant damage (SD) and near collapse (NC) for both regular and stiffness irregular (Soft-storey) MRF.

Keywords: Static pushover analysis, IDA, Soft-storey, High strength concrete Normal strength concrete.

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ACRONYMS

ATC: Applied Technology Council

CP: Collapse Prevention

FEMA: Federal Emergency Management Agency

HSC-SI: Stiffness irregular high strength concrete

HSC-R: Regular high strength concrete

IDA: Incremental Dynamic Analysis

IO: Immediate Occupancy

LS: Life Safety

NSC-R: Regular normal strength concrete

NSC-SI: Stiffness irregular normal strength concrete

PGA: Peak ground acceleration

RC: Reinforced concrete

SPO: Static Push over

CHAPTER ONE

1.1 GENERAL

An important aspect of seismic response of buildings is the regularity of the structural system. The damage in a structure generally initiates at location of the structural weak planes present in the building systems. These weaknesses trigger further structural deterioration which leads to the structural collapse. The weaknesses often occur due to presence of the structural irregularities in stiffness, strength and mass in a building system. The structural irregularity can be broadly classified as plan and vertical irregularities.

In most of situations, buildings become vertically irregular at the planning stage itself due to some architectural and functional reasons for instance basements for commercial purposes created by increasing ground floor columns height. Also, reduction of size of beams and columns in the upper storeys to fulfill functional requirements leading the engineer to utilize high strength concrete (HSC). This various requirements to fulfill for the buildings results in vertically irregular as well as use of high strength concrete in the building to meet the specified demand. Vertical irregularities such as mass irregularity, stiffness irregularity and strength irregularity (weak story) in buildings are very common feature in urban area.

The focus of present study is to assess the relative performances of typical soft story (stiffness irregular) buildings in designed for high strength concrete.

Stiffness Irregularity - Soft Story: is defined to exist when there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above

Buildings with irregular configurations demonstrated more vulnerability in the past earthquakes. It has been observed after previous earthquakes that buildings having irregularities have higher seismic demands. These buildings suffer significantly more damage than those that have regular floor plans, without discontinuities. Typical soft story irregular frame buildings which suffer during past earthquake are shown in Figure 1.1a and 1.1b as follows.

Figure 1.1a Failure of Olive View Medical State Center (Moehle and Mahin, Nisee Berkeley)

Figure 1.1b Failure of buildings due to soft storey in Turkey during Sumatra earthquake 2004 (Kirac et al. 2011)

1.2 STATEMENT OF PROBLEM

Nowadays due to growing population and limited space in urban areas the demand for the designing of the structures with irregular configurations especially stiffness irregular structures are becoming common. On the other hand, structural engineers often recommend concretes with the high compressive strength when the space constraints are inevitable to increase the gravity load carrying capacity of the structural members. For architectural reasons or space restrictions many multistory buildings are designed with High strength concrete (HSC) which leads to smaller size of the reinforced concrete beams and columns in the lower stories. This is associated with a change in the lateral resistance or stiffness of the structures when subjected to ground motions that in turns has impact on the overall stability of the structures partly because the concrete tends to be brittle rather than ductile as the compressive strength of concrete increases. The Combining effects of structural stiffness irregularity and the less ductility of concrete material pose the significant effect on the seismic performance of the building.

Most of the available studies related to the seismic behaviour of reinforced concrete frame buildings with irregularity in stiffness are designed with Normal Strength Concrete (NSC). Hence, to know the behavior of the moment resisting frame buildings with stiffness irregularity configuration designed for HSC under earthquake loading, such study is paramount important for it will provide detailed analytical investigations into structural behavior of the building.

1.3 RESEARCH QUESTION

The study intended answer the following question:

1. What is the effect of a soft-story (Stiffness irregularity) on the seismic performance of moment resisting (MRF) reinforced concrete building?

2. What is effect of concrete with high compressive strength on seismic performance of regular moment resisting frame (MRF) reinforced concrete building?

3. What is effect of concrete with high compressive strength on seismic performance of stiffness irregular (soft-story) moment resisting frame (MRF) reinforced concrete building?

4. What is the probability of failure of the reinforced concrete MRF building due to introduction soft story (stiffness irregularity) at the first story level which designed for normal strength concrete (NSC)?

5. What is the probability of failure of the reinforced concrete MRF building due to introduction soft story (stiffness irregularity) at the first story level which designed for High strength concrete (HSC)?

1.4 OBJECTIVE OF THE STUDY

1.4.1 GENERAL OBJECTIVE

The objective of this Study is to investigate the seismic performance of medium height soft-story (stiffness irregular) moment resisting (MRF) frame building with high strength concrete subjected to seismic lateral load in addition to gravity loads as compared to the regular MRF designed for normal strength (NSC-R).

1.4.2 SPECIFIC OBJECTIVES

The specific objectives of the present study include:

- To assess the seismic performance of buildings with soft-storey (stiffness irregular) irregularity designed for normal strength concrete (NSC) using static pushover analysis.
- To assess the seismic performance of buildings with soft-storey (stiffness irregular) irregularity designed for high strength concrete (HSC) using static pushover analysis.
- Performing the incremental dynamic analysis (IDA) to check the response of the soft-storey (stiffness irregular) irregularity designed for normal strength concrete (NSC) and high strength concrete (HSC) under seismic loading.

- Performing seismic vulnerability assessment in the form of fragility curves for reinforced concrete moment resisting frames buildings
- Performing Comparative analysis for soft-story reinforced concrete moment resisting frames designed for both concrete strength classes with the baseline design case which is regular MRF designed for normal strength (NSC-R)

1.5 SIGNIFICANCE OF THE STUDY

The output of the study will provide important insight into the effect of stiffness irregularity on seismic performance of moment resisting frame. In addition, it attempts to address the effect of using high compressive strength in combination with stiffness irregularity (soft-storey) on moment resisting frame (MRF), as the utilization of material with high strength is now common in building construction. Hence the study is expected to shed some light on the general understanding of seismic response of stiffness irregular building designed with high strength concrete.

Finally, it can also serves engineers and designers as guidelines to improve design code provisions related to the stiffness irregular building with high strength concrete.

1.6 SCOPE AND LIMITATION OF THE STUDY

The scope of the current research is limited to the following investigation:

i) Review of previous literature; and seismic codes and standards: The review of previous literature on seismic performance and seismic fragility analysis of reinforced concrete buildings will be conducted.
   - Literature review on code high strength concrete with regular and irregular structural layouts.
   - Literature review on irregular buildings specifically those related to the stiffness irregularity.
   - Review of Seismic design and detailing requirements of new Ethiopian building codes and Standard ES EN 1998-1 2015.

ii) Selection of buildings: Four Reinforced concrete Moment resisting frame buildings with 12-storeys are designed for normal strength concrete (NSC) and high strength concrete (HSC) with Addis Ababa, Ethiopia as their locations

iii) Seismostruct (2018) is used for non-linear static pushover analysis and incremental dynamic analysis. The weight of the slab, partition and floor finishing is calculated and applied to the beam as the uniformly distributed load.
   - Modeling of buildings for Seismostruct (2016) analysis: Three dimensional building models were generated in Seismostruct (2018) based on the design and detailing according to ES-EN 1998-1 2015. The models consisted of beam and column elements with potential plastic hinges at their ends.
   - Selection of site-specific earthquake records compatible with the Uniform Hazard Spectra (UHS) specified in the ES EN 1998-1 2015 for use in dynamic analysis: A

total of 20 synthetic earthquake records were selected for location in Addis Ababa for use in incremental dynamic analysis(IDA).

- Incremental Dynamic Analysis (IDA): IDA was conducted to generate data for fragility analysis under incrementally increasing seismic intensity.
- Selection of target building performance objectives: Building performance levels were selected as i) Immediate Occupancy, ii) Life Safety, and iii) Collapse. These target building performance levels were quantified based on roof drift calculated from nonlinear static pushover analysis as the damage indicator.
- Statistical analysis of IDA results to develop fragility curves: Fragility curves were developed for four sets of buildings; a) Normal strength concrete buildings with regular structural layouts (NSC-R) –Baseline case building, b) High strength concrete buildings with regular structural layouts (HSC-R) and c) Normal strength concrete buildings with first storey soft- storey (stiffness irregular) structural layouts (NSC-SI) c) High strength concrete buildings with first storey soft- storey (stiffness irregular) structural layouts (HSC-SI)

CHAPTER TWO
LITERATURE REVIEW

2.1 INTODUCTION

This chapter present the review of literature related to this research work. The literature review concentrates on the current state of the art in the seismic performance evaluation of existing buildings, and discusses overview of the relevant published literatures related to the current study. The discussion starts with the literatures on the seismic performance evaluation of stiffness irregularity of reinforced concrete and the review on pushover analysis and Incremental dynamic analysis (IDA) followed by a review of published literatures on the fragility curves.

The performance criteria of Moment resisting frame RC building as defined by Federal Emergency Management Agency (FEMA), Applied Technology Council (ATC-40) and different researchers will also be reviewed.

2.2 SEISMIC PERFORMANCE OF BUILDING

Performance based approach requires that response of a structure should match the demands to which it is subjected. The demands are referred to as structural requirements and response level is said to be performance expectations. When these two are equated we can quantify hazard and evaluate different losses which will occur if a desired performance level is not achieved. Performance of a building structure includes both structural and non-structural damages and is quantified in terms of a limit state known as damage state or performance limit state. Level of demand and response is defined for every limit state. A performance limit state is said to be achieved when defined level of demand and response is matched. Different codes define discrete limit states out of many expected damage states to be experienced by the building structure when subjected to an anticipated level of ground shaking. Different performance limit states have been defined by different methods and codes. FEMA 273 and FEMA 440 uses global roof drift ratio levels of 0.7%, 2.5% and 5% for immediate occupancy (IO), life safety (LS) and collapse prevention (CP) limit states for RC multistory building. Same performance parameters are also defined in FEMA 356.

Recent interests in the development of performance based codes for the design and rehabilitation of buildings in seismically active areas show that the inelastic procedure commonly referred to as “pushover analysis” and Incremental dynamic analysis (IDA) are viable method to assess the damage vulnerability of buildings.

2.3 PUSHOVER ANALYSIS

The static pushover procedure was firstly presented by Saiidi and Sozen (1981), and was later used in many seismic analysis studies. In push over analysis the building is pushed incrementally until control node, which is normally the roof of a building structure, reaches a target displacement. Alternatively the structure can be pushed up to a level of base shear which is expected to be achieved during a design earthquake. The gravity loads are considered as constant during the procedure. The target displacements are selected as desired performance levels to be achieved during a design earthquake (Bruneau et al., 1998). The procedure can determine the collapse mechanism and point out the sequence of yielding and failure of components. The ductility and strength demands at the target displacement or target base shear are used to ensure the acceptance of the structural design. The capacity spectrum is basic output of pushover analysis and describes overall performance of a building. FEMA 273 also includes this method and recommends its use for analysis of new and old structures. The procedure is now acceptable and considered as an easy solution which could give estimate of deformation demands.

2.3.1 LIMITATIONS OF PUSHOVER ANALYSIS

Although pushover analysis has pros over elastic analysis procedures, underlying assumptions, the accuracy of pushover predictions 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 vibrations are important issues that affect the accuracy of pushover analysis result. The pushover analysis is a useful, but not impeccable tool for assessing inelastic strength and deformation demands and for exposing design weaknesses. It must be emphasized that the pushover analysis is approximate in nature and is basic on static loading. As such, it cannot represent dynamic phenomena with a large degree of accuracy. Thus, in the present study IDA also is performed.

2.4 INCREMENTAL DYNAMIC ANALYSIS

Vamvatsikos and Cornell (2002) proposed a new method that meets the requirements of performance based earthquake engineering. The procedure is called Incremental Dynamic Analysis (IDA), which involves performing nonlinear dynamic analyses of the structural model under a suite of selected ground motion records, each scaled to several intensity levels designed to force the structure all the way from elasticity to final global dynamic instability (Vamvatsikos D. et al. 2002). The structural response is presented by IDA curves plotted between Damage Measure (DM) and Intensity Measure (IM). DM can be peak roof drift or inter story drift. IM is ground motion intensity for example, peak ground acceleration or 5%-damped first-mode spectral acceleration $S_a(T_1; 5\%)$. The IDA curves summarize the distribution of demand DM for given intensity IM. The probability of exceedance of a limit-state, such as Immediate Occupancy or Collapse Prevention (FEMA-350, 2000a) can be defined on each IDA curve. The output of IDA can be easily superimposed with conventional hazard curves to calculate annual rates of exceeding of a certain limit-state capacity or a certain demand. The IDA has the following relative advantages:

- Thorough understanding of the range of response or demands versus the range of potential levels of a ground motion record,
- Better understanding of the structural implication of rarer/more severs ground motion levels,

- Better understanding of the changes in the nature of the structural response as the intensity of ground motion increases (e.g. changes in peak deformation patterns with height, onset of stiffness and strength degradation and their patterns and magnitudes),
- Producing estimates of the dynamic capacity of the global structural system,
- And finally, given a multi-record IDA study, understanding how stable (or variable) all these items are from one ground motion record to another.

2.5 STRUCTURAL IRREGULARITY

Nowadays different floors of buildings are used for purposes like car parking, storing heavy mechanical appliances, for observatory towers at top etc. this results in variation of mass, strength and stiffness at different storeys. Current earthquake codes define structural configuration as either regular or irregular in terms of size and shape of the building, arrangement of the structural and nonstructural elements within the structure, distribution of mass in the building etc. A regular structure can be designed to have uniformly distributed mass, stiffness, strength and structural form. When one or more of these properties is non-uniformly distributed, either individually or in combination with other properties in any direction, the structure is referred to as being irregular. Structural irregularity may occur for many reasons. Some irregularities are *architecturally planned*.

Examples of these structural irregularities are:

- A factory with heavy machinery, or an educational institution with a library at one floor level that leads to irregular distribution of mass.
- A residential building having a car park in the basement producing a first soft- storey.
- A shopping complex with setbacks to accommodate boundary offset requirements as shown in the plan.
- Buildings with flexible, rigid or no diaphragms at a floor level, or structural plan having different lateral load resisting systems (resulting in torsion)

A structure can also be irregular because of unplanned effects, which include rearrangement of loadings, as well as material strength and stiffness variations.

For the above reasons, structures are never perfectly regular and hence the designers routinely need to evaluate the likely degree of irregularity and the effect of this irregularity on a structure during an earthquake.

Soft storey (Stiffness irregularity): The stiffness of the seismic-force-resisting-system in any storey is not lessnthan 70% of the seismic-force-resisting system stiffness in an adjacent storey above or less than 80% of the average seismic-force-resisting-system stiffness of the three stories above. Soft storey usually occurs in commercial buildings with open fronts at ground floor storefronts and hotels or office buildings with particularly tall first stories. Soft stories usually result in an abrupt change in storey drift. A tall story or a change in the type of seismic-force-resisting-system is an obvious indication that a soft storey might exist. A gradual reduction of seismic-force-resisting elements as the building increases in height is typical and is not considered a soft-storey condition. The difference between "soft" and "weak" stories is the difference between stiffness and strength. A change in column size can affect strength and stiffness, and both need to be considered.

Weak Storey (Strength irregularity): The sum of the shear strengths of the seismic-force-resisting system in any storey in each direction is not less than 80% of the strength in the adjacent storey above. The storey strength is the total strength of all the seismic force-resisting elements in a given storey for the direction under consideration (the shear capacity of columns or shear walls or the horizontal component of the capacity of diagonal braces). If the columns are flexure controlled, the shear strength is the shear corresponding to the flexural strength. Weak stories are usually found where vertical discontinuities exist or where member size or reinforcement has been reduced. Weak storey induces in a concentration of inelastic activity which may result in partial or total collapse of the storey.

In previous years large numbers of research works have been carried out in relation to mass irregularity

Esteva L., (1992) studied the nonlinear response of buildings with excessive stiffness and strength above the first story. It is stated that the response of a building is quite sensitive to the stiffness variation along the height of the structure and the p-delta effects are significant on the response. The use of a safety factor to meet the local ductility demands in a soft story, which is dependent to the natural period of a structure, is offered.

Chintanpakdee and Chopra (2004) evaluated the effects of strength, stiffness and combination of strength and stiffness irregularity on seismic response of multistory frames. For analytical study, different 12 storey frames were modeled based on strong column – weak beam theory. The irregularity in strength and stiffness were introduced at different locations along height of the building models. The building models were analyzed using time history analysis by subjecting the building model to 20 different round motion data. From analytical study it was concluded that irregularities in strength and stiffness when present in combination had the maximum effect on the seismic response. Further maximum variation in the displacement response along height was observed when irregularities were present on the lower storeys.

Fragiadakis et al. (2005) determined the seismic response of building systems with irregular distribution of strength and stiffness in vertical direction. After conducting the analytical study it was concluded that seismic performance of the structure depended on type and location of irregularity and on intensity of seismic excitation.

Ellingwood et al. (2007) developed fragility response for RC frame due to the potential impact of earthquake in low-to moderate seismicity region where building design and construction followed gravity load design without providing provision for earthquake resistance.

Three-storey and six-storey frame buildings were designed according to ACI 318 (ACI 1989) for gravity with load combination 1.4D + 1.7L with no consideration of seismic resistance. Open source program Opensees (Opensees 2007) was used to model the frames where fibre approach nonlinear uniaxial constitutive concrete and steel model were used to develop element/section. Synthetic uniform hazard ground motions for Central and Eastern United States were selected from Mid-America Earthquake Center. 10 ground motions were generated using attenuation models from each 10%/50 year and 2%/50 year hazard level ground motions developed by WenWu (2001) for both hard rock and representative soil sites. 10 ground motions were also generated using attenuation models from each 10%/50 year, 5%/50 year and 2%/50 year hazard level ground motions developed by Rix-Fenandez (Fernenandez and Rix 2006).

Athanassiadou (2008) studied multi storey reinforced concrete (R/C) frame buildings with irregularity in elevation. Two ten-storey two-dimensional plane frames with two and four large setbacks in the upper floors and another building which was regular in elevation, all designed based on the provisions of the 2004 Eurocode8 (Eurocode 2005) for high ductility (DCH) and medium ductility (DCM) were considered. The same peak ground acceleration (PGA) and material characteristics were used for both cases. SAP2000 generated models were subjected to inelastic static pushover analysis and inelastic dynamic time-history analysis for assessment of seismic performance based on both global and local criteria. The authors presented the results and showed the effects of vertical irregularities.

Kappos and Stefanidou (2010) proposed a new deformation design method based on inelastic analysis for the setback frames. From analysis results, adequate seismic performance of the setback frames designed as per the proposed method was observed.

Celik and Ellingwood [2010] studied the effects of uncertainties in material, structural properties and modeling parameters for gravity load designed RC frames. It was found that damping, concrete strength, and joint cracking have the greatest impact on the response statistics. However, the uncertainty in ground motion dominated the overall uncertainty in structural response. The study concluded that fragility curves developed using median (or mean) values of structural parameters may be sufficient for earthquake damage and loss estimation in moderate seismic regions.

Kim and Hong (2011) determined the collapse resisting capacity of the building models with stiffness and strength irregularity. The irregularity in the building models was created by removal of column in the intermediate storey. However, analysis results suggested minor variation in the collapse potentials of regular and irregular structures.

Rajeev and Tesfamariam (2012) developed vulnerability curves for RC buildings, which have either poor quality of construction on different irregularities such as weak storey, soft storey, plan irregularities. Soft storey (SS) and quality of construction (CQ) was taken into consideration to develop fragility based seismic vulnerability of structures.

Varadharajan et al. (2013) studied the behaviour of RC moment resisting frames with setbacks, proposing ‘irregularity index’ based on the dynamic characteristics of the frames to quantify the setback irregularity. They modeled 2D Frames with different arrangements of setbacks designed in accordance with the current European standard code, Eurocode 8. They analyzed the buildings using ETABS and 13 ground motions scaled to different intensities to obtain different performance levels as prescribed by SEAOC (SEAOC 1995). They observed significant influence of beam–column strength ratio, number of stories, and geometric irregularity on inelastic seismic demands. Maximum deformations occurred in the vicinity of setbacks and at tower portions of the setbacks.

CHAPTER THREE
METHODOLOGY

3.1 GENERAL

The seismic evaluation of concrete buildings poses a great challenge for the owners, architects engineers and building officials. The inherent complexity of concrete building and their performance during earthquakes compounds the uncertainty. The traditional design and analysis procedures developed primarily for new construction are not wholly adequate tools for meeting this challenge. This section presents general methodology that will be used specifically to address the seismic evaluation of concrete building.

3.2 STUDY AREA

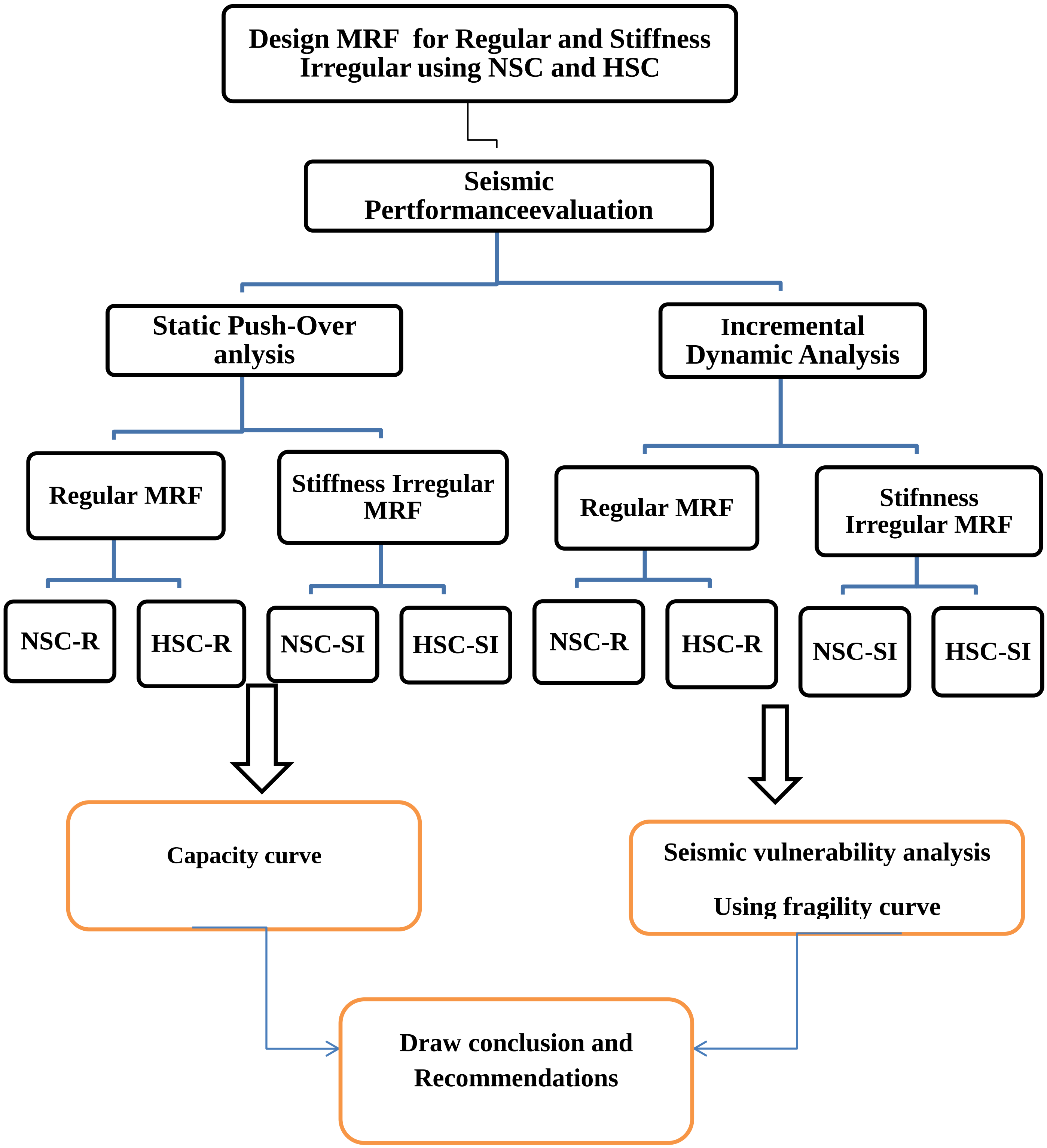
The study was conducted in Addis Ababa city, which is located in central Ethiopia. The geographical locations are 9° 0' 19.4436" N latitude and 38° 45' 48.9996" E latitude and longitude respectively. The altitude is 2350m above sea level with the estimated area of. 527 km². The seismic zonation for the study area is selected based on ES EN 1998-1: 2015 seismic hazard zonation of Ethiopia.

3.3 RESEARCH DESIGN

The research is exploratory research which will address the effect of stiffness irregularity (soft story) on seismic performance of moment resisting frame (MRF) for different concrete strength. First the frame is designed using ETABS V.18.1 as per new Ethiopian building code and standards (ES EN 1998-1-2015) for all cases. Following the design and detailing of frame, seismic performance assessment of moment resisting frame will be performed by Seismostruct (2018) using FEMA 356 and ATC 40 guidelines. Nonlinear static (Static push over) and Incremental dynamic analysis (IDA) are employed for performance assessment. The former method of analysis is a static nonlinear analysis under permanent vertical loads and gradually increasing lateral loads whereas, the Incremental dynamic analysis (IDA) involves performing a series of nonlinear dynamic analyses in which the intensity of the evaluated ground motion is monotonically increased until the structure's collapse limit state is reached. A measure of the ground motion intensity is then plotted against a selected engineering demand parameter (EDP), such as maximum interstory drift ratio.

The paper also investigates Building vulnerability to seismic hazard by performing probabilistic seismic exceedance of certain damage state using the predefined performance level. Three performance levels were considered. Immediate Occupancy (IO) performance level describes the damage state where structure is safe to be re-occupied having suffered minor damage to the structural elements with minor spalling and flexural cracking. Life Safety (LS) performance level describes the damage state where significant damage has occurred to the structure with extensive cracking and hinge formation in primary structural elements. Collapse Prevention (CP) performance level describes the damage state where the lateral-force resisting system loses most of its pre- earthquake strength and stiffness. Substantial degradation of structural elements occurs, including extensive cracking and spalling of masonry and concrete elements, and buckling and fracture of steel elements.

3.3.1 SCHEMATIC DIAGRAM OF RESEARCH DESIGN



3.4 STUDY VARIABLES

3.4.1 DEPENDENT VARIABLES

- Story base shear
- Story displacement
- Pushover capacity curve
- IDA capacity curve
- Fragility curve

3.4.1 INDEPENDENT VARIABLES

- Story stiffness
- Ductility class
- Story height
- Ground motion input (PGA)
- Concrete strength

3.5 POPULATION AND SAMPLING METHOD

3.5.1 SAMPLE SIZE

Six reinforced concrete frame buildings of 12-storey height, with a regular and stiffness irregularity (soft-story) were selected for analysis. The buildings have a floor plan of 4 bays in x-direction and 3 bays in y-direction with a 5 m span length floor. The story height is 3.0 m for all floor in stiffness regular building and 5.5 m for first story (ground floor) and 3m for the remaining story height in the soft story (stiffness irregular) building. The buildings are designed and detailed according to the provisions of ES EN 1998-1 2015 located in Addis Ababa city, Ethiopia.

3.6 SAMPLING PROCEDURE

Study program is broadly classified into two namely, a Regular Moment Resisting Frame and Soft- Story (Stiffness Irregular) Moment Resisting Frame (MRF). Both cases are designed and detailed for normal strength concrete (NSC) and high strength concrete (HSC) to understand the effect of soft-story (stiffness irregularity) and concrete strength variation on performance of Moment Resisting Frame (MRF) building. The soft-story (stiffness Irregularity) is introduced by increasing the height of ground floor from 3m to 5.5m The building samples are designated as NSC-R (regular normal strength), HSC-R (Regular high strength),for Regular MRF and NSC-SI (Stiffness irregular normal strength concrete), HSC-SI (Stiffness Irregular high strength concrete) for stiffness irregular MRF as indicated in the above Schematic diagram.

3.7 DATA COLLECTION PROCEDURE

This section sets a general procedure of data collection methods and the strategies that may be used for the analysis, design, and detailing and seismic performance evaluation of MRF. The necessary data shall be collected from literature and different codes of practice adopted including new Ethiopian building code and standards ES EN-1998.1 2015).

The configuration of the structural system, as well as the type, detailing, connectivity, material strength and condition of the structural elements comprising the building shall be determined in accordance with Ethiopian building code and standard. Data shall also be obtained for all nonstructural elements of the building that affect the forces and deformations experienced by the structural elements during response to earthquake ground motion.

The information required regarding the properties of material for MRF building design may be obtained from specification and guidelines set in ES EN 1998-1 2015 and also from a previously conducted laboratory test by different researchers. The stiffness of the frame is calculated from the mechanical properties such as modulus of elasticity and cross sectional properties of frames structure.

For all designs cases the gravity load and seismic loads are calculated in accordance to the ES EN 1991, 1-1: 2015.

The data collection process for seismic performance evaluation includes: acquisition of available documents including the building geometry and configuration, mass and element of seismic load path such as frames and diaphragm. Building configuration and layout of structural members comprises size of members, size of reinforcing, tie spacing, splice locations and concrete cover. The numerical model is created in the nonlinear analysis software Seismostruct (2018), according to the design and detailing considerations regarding materials, sections and elements definition based on ES EN 1998-1 2015. The required input information for non-linear analysis is obtained from structural design and detailing documents, including design drawings, and material test records and material constitutive model developed by different researcher for non – linear material properties**.**

In nonlinear static (static pushover) analysis pseudo-static loads (forces or displacements) that are incrementally varied at any step defined by the user. The magnitude of a load at any step is given by the product of its nominal value defined by user and the load factor which is updated in automatic or user-defined fashion. Incremental loads are employed in pushover type of analyses, generally used to estimate horizontal structural capacity.

The assessment of performances and demands of regular and Stiffness irregular moment resisting frames requires a set of acceleration time histories with amplitude, frequency content, and duration. However, in most cases, using time histories from actual earthquake data has many limitations for many reasons. ES EN 1998-1:2015 stipulates that artificial accelerograms shall be generated so as to match the elastic response spectra used in the design for 5% viscous damping. Accordingly, sets of artificial earthquake records will be generated with help of Seismoartif (2018) as suitable candidates for incremental dynamic analysis (IDA). The detail structural design data, input ground motion and material constitutive model for the nonlinear analysis are presented in the chapter five of this thesis.

3.8 DATA ANALYSIS

Seismic Design of proposed RC buildings is performed on ETABS 2018.2.1 following the new Ethiopian Building code analysis and design approach. All building model cases are analyzed both for gravitational loads and earthquake loads by situating the proposed building in Addis Ababa area (earthquake zone-III) using response spectrum method.

The numerical values found from the design section are then used for nonlinear l modeling of RC frames on finite element software package (SeismoStruct 2016). Pushover and nonlinear time history analysis (incremental dynamic analysis) are performed on all model cases.

Incremental dynamic analysis (IDA) is conducted using software seismostruct (2018) to perform nonlinear dynamic analysis and evaluate the inelastic performance of structural components. Three-dimensional analytical models of the regular frame and soft story (stiffness irregular) structures with different concrete strengths (i.e. NSC and HSC) designed earlier are developed for dynamic analysis using Seismostruct (2018). A set of 20 artificial earthquake records are generated using Seismoartif (2018) and used as an input in the software (Seismostruct) to perform incremental dynamic analysis (IDA). Artificial earthquake records compatible with the Uniform Hazard Spectra (UHS) specified in ES EN 1998-1 2015 developed for Addis Ababa site. IDA method will be conducted for each seismic record with incrementally varying intensity levels, resulting in an IDA curve providing a relationship between earthquake intensity and a structural deformation quantity.

Finally, using the IDA results the fragility curve will be developed to assess the structure probability of exceeding the damage levels for the given ground motion intensity level.

3.8.1 PERFORMANCE LEVELS

Three performance levels are considered. Immediate Occupancy (IO) performance level describes the damage state where structure is safe to be re-occupied having suffered minor damage to the structural elements with minor spalling and flexural cracking. Life Safety (LS) performance level describes the damage state where significant damage has occurred to the structure with extensive cracking and hinge formation in primary structural elements. Collapse Prevention (CP) performance level describes the damage state where structure is at the onset of partial or total collapse with extensive cracking, hinge formation and reinforcement buckling in structural elements.

3.8.2 DEVELOPMENT OF FRAGILITY RELATIONSHIPS

A fragility function represents the probability of exceedance of the selected Engineering Demand Parameter (EDP) for a selected structural limit state (DS) for a specific ground motion intensity measure (IM). These curves are cumulative probability distributions that indicate the probability that a component/system will be damaged to a given damage state or a more severe one, as a function of a particular demand. Fragility curve can be obtained for each damage state and can be expressed in closed form as using Eq. E.1

$$P(C\text{-}D)/IM = 1- \left(\emptyset \left(\frac{ln\frac{S_d}{S_c}}{\sqrt{\beta^2{}_{\frac{d}{IM}} + \beta^2{}_C + \beta^2{}_M}} \right) \right) \quad \text{……………………………..……..…….3.1}$$

Where, D is the drift demand, C is the drift capacity at chosen limit state, S_d is the median of the demand and S_c is the median of the chosen damage state (DS) respectively. $\beta_{d/IM}$, β_c and β_M are dispersions in the intensity measure, capacities and modelling respectively. Eq. E.1 can be rewritten as Eq. E.2 for component fragilities (Nielson, 2005) as,

$$P\ (DS/IM) = 1- \emptyset \left(\frac{lnIM - lnIM_m}{\beta_{comp}} \right) \quad \text{………………………...…………………….. 3.2}$$

Where, $IMm = exp^{\left(\frac{ln\ s_C + \ln(a)}{b} \right)}$ a and b are the regression coefficients of the probabilistic seismic demand model (PSDM) and the dispersion component, $\boldsymbol{\beta}_{comp}$ is given,

$$\beta_{comp} = \sqrt{\frac{\beta^2{}_{d/IM} + \beta^2{}_c}{b}} \quad \text{………………………………………………………..... 3.3}$$

The dispersion in capacity, βc is dependent on the building type and construction quality. In this study, dispersion in capacity has been assumed as 0.25. It has been suggested by Cornell et. al (2002) that the estimate of the median engineering demand parameter (EDP) can be represented by a power law model, which is called a Probabilistic Seismic Demand Model (PSDM) as given in Eq. E.4.

$$EDP = a\ (IM)^b \quad \text{…………………………………………………….……….3.4}$$

Where: “a” and “b” are the regression coefficients of the PSDM. In this Paper, Roof drift (d_i) is taken as the engineering damage parameter (EDP) and peak ground acceleration (PGA) as the intensity measure (IM). Hence, Eq. E.1 can be rewritten for system fragilities as follows:

$$P\ (D \geq C/M) = 1 - \emptyset \left(\frac{\ln(S_c) - \ln(aIM^b)}{\sqrt{\beta^2{}_{\frac{d}{IM}} + \beta^2{}_C + \beta^2{}_M}} \right) \quad \text{………………………………......3.5}$$

Accordingly, the roof drift calculated by IDA is taken as EDP and PGA values are taken as intensity measure and summarized in the following table for each case.

$$\beta_{D/IM} = \sqrt{\sum_{i=1}^{n} \frac{\left(\ln(di) - \ln(a(IM)^{b})\right)^{2}}{n-1}} \qquad 3.6$$

To caluclate the value of a and b, Curve fitting principle is used. This method is used when we often have “y” data, that is a function of some independent variable “x”, but the underlying relationship is unknown. Hence, curve fiiting principle that determine a function (i.e., a curve) that “best” describes relationship between x and y.

3.9 DATA PRESENTATION

Finally, the performance of the model structures at different performance levels has been investigated and their results are discussed in terms of the response parameters, such as total base shears, inter-story drifts, lateral displacements, and seismic fragility curves. Seismic fragility curve is the relationship between maximum roof drift and spectral acceleration is used in this paper.

CHAPTER FOUR
ANALYSIS AND DESIGN OF MOMENT RESISTING FRAME

4.1 DESCRIPTION OF BUILDING

A twelve storey RC moment resisting frame (MRF) building with regular and stiffness irregular (soft-storey) is considered in the study. The building has overall plan dimension of 20m × 15m, with 4 bays in the X- direction and 3 bays in the Y- direction and designed according to the new Ethiopian Building Code (ES EN 1998-1 2015). To represent the likely variations of concrete strength that might be used for most building in the city, two classes of concrete strengths are considered; these concrete strengths are normal strength concrete (NSC) and high strength (HSC). First the building are designed as regular moment resisting frame using both normal strength concrete (NSC) and high strength concrete (HSC) and then the building are designed by introducing the soft story (stiffness irregularity) at the first floor level by increasing story height for both normal strength concrete and high strength concrete. Seismic Design of proposed RC buildings is performed on ETABS 2018.2.1 following the new Ethiopian Building code analysis and design approach. All building model cases are analyzed both for gravitational loads and earthquake loads by locating the proposed study site in Addis Ababa (Earthquake zone-III) using response spectrum method.

4.2 ANALYTICAL MODELS

Figure 4.1 through 4.4 shows the plan and elevation view of the frame model created by CSI ETABS 18.1.

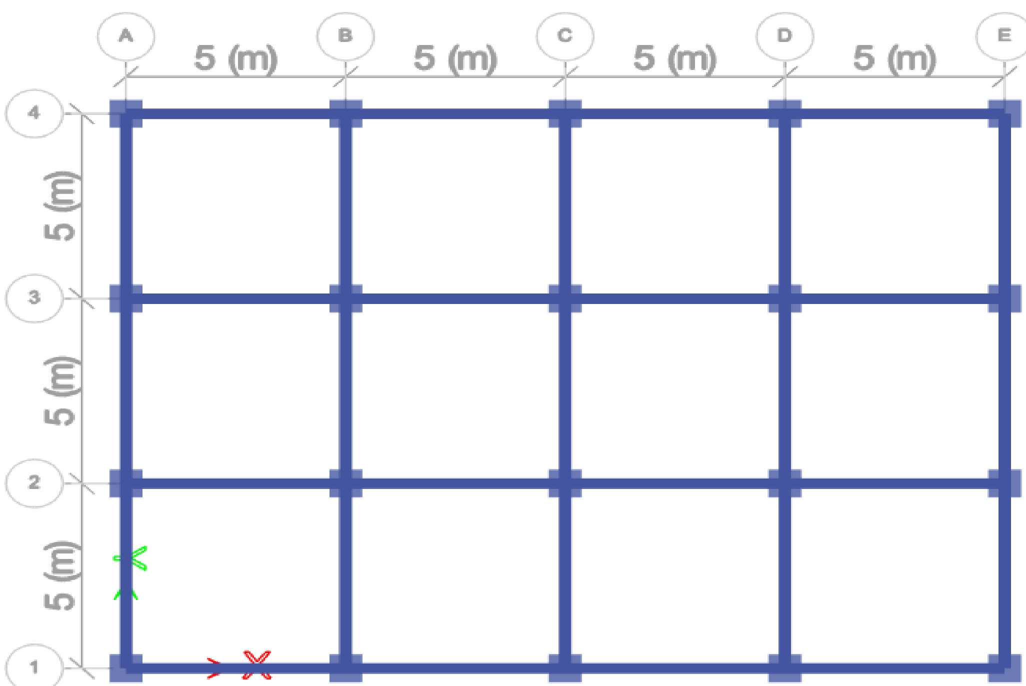


Figure 4.1 : Typical stiffness regualr MRF model –Plan view

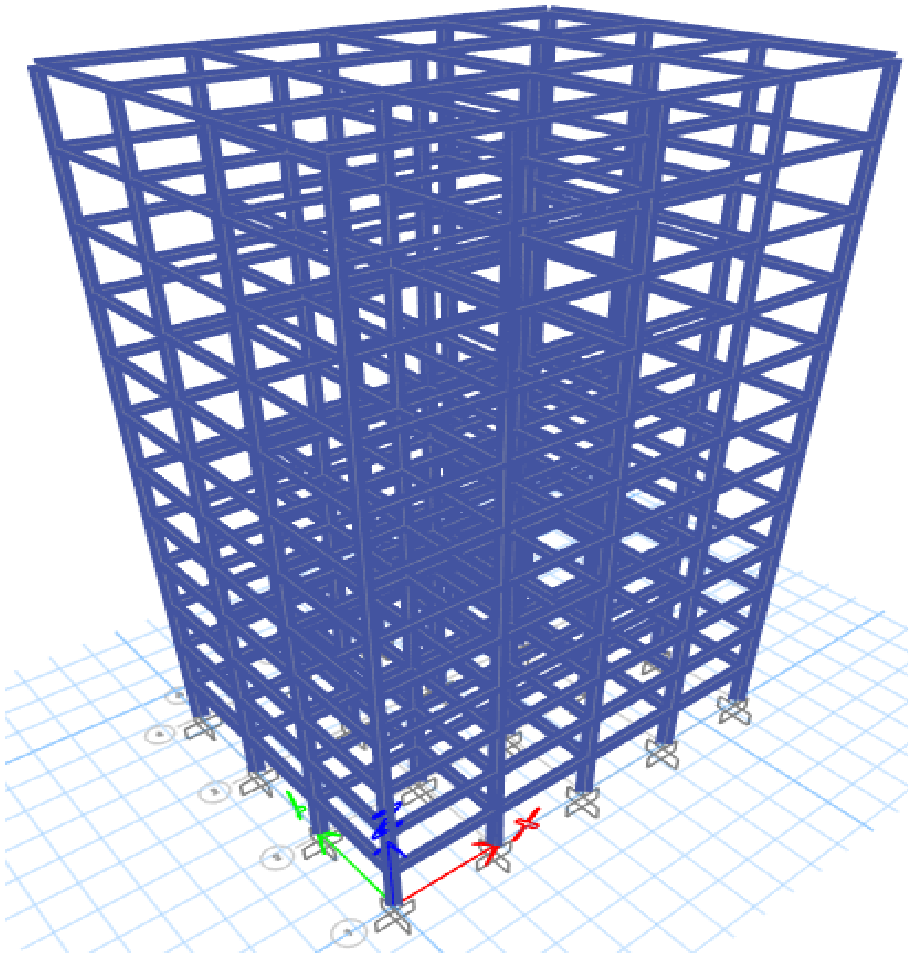


Figure 4.2: Typical stiffness regualr MRF model -3D view

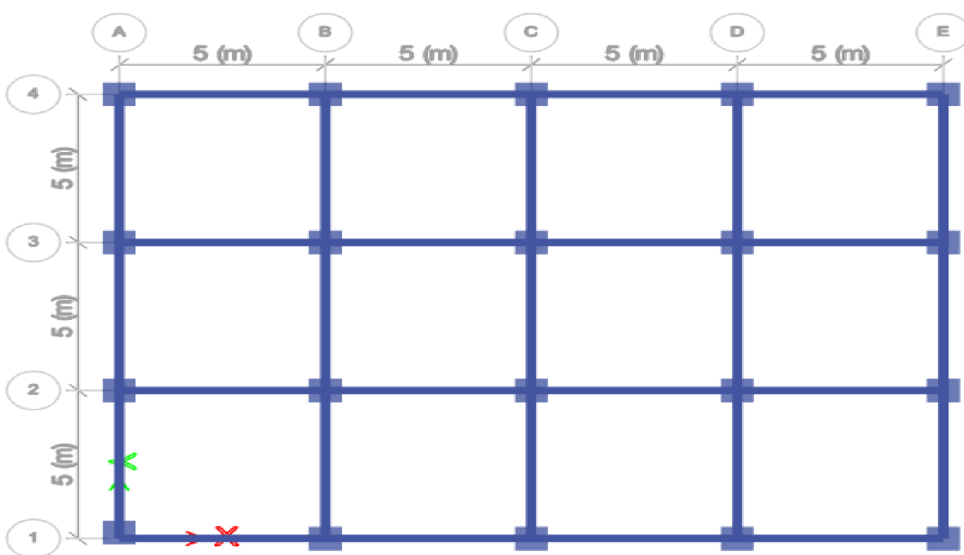


Figure 4.3 : Typical stiffness irregualr MRF model –Plan view

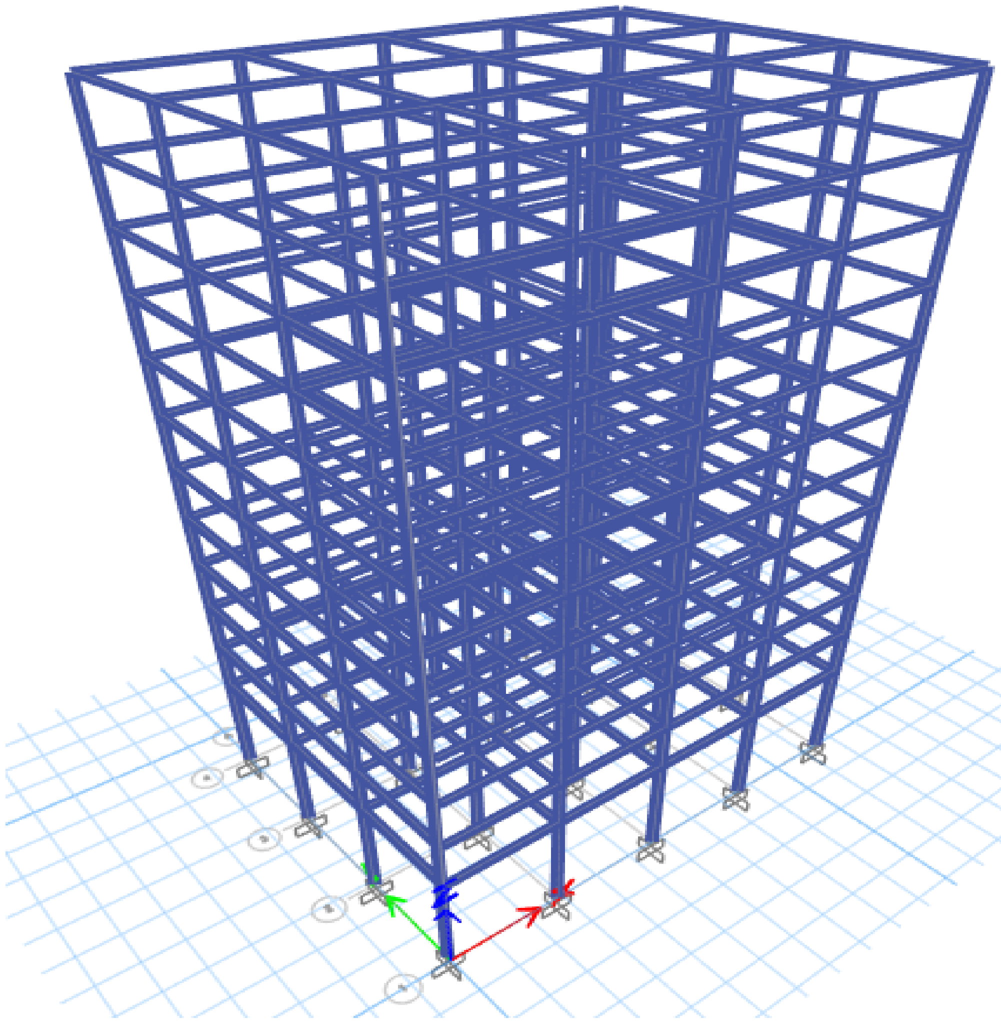


Figure 4.4 : Typical stiffness irregualr MRF model -3D view

4.3 STRUCTURAL DESIGN

The design process consists of preparing a basic structural analysis model of the building with the dimensions and details obtained from preliminary design strategies. Then apply design lateral forces, perform structural analysis, and design structural elements based on results obtained from structural analysis. Seismic action is used as governing lateral force on the building structures and the analysis for the lateral action followed modal response spectrum method. These steps include preiliminary analysis and member sizing, detialed analysis, design and detialing of reinforcement.

Material propeties required for calculation of the loads on the structure is estimated accordignt to the ES N 1998-1-1 2015 for all matreials including Concrete, Steel, Cement screeding , Floor finishing, plastering and Hollow concrtete block (HCB) shown in the table the 4.1,4.2 and 4.3.

4.4 MATERIAL PROPERTIES

Table 4.1: Material properties

Item	Material	Unit Weight
1	Cement Screed	23 kN/m^3
2	Ceramic floor finish	23 kN/m^3
3	Plastering	23 kN/m^3
4	HCB	14 kN/m^3
5	Concrete strength	**1. Normal strength concrete (NSC)** Grade C-25 (Columns, Beams, slabs) *Partial Safety Factor = 1.5* f_{ctk} =1.5 Mpa E_{cm} = 30 Gpa f_{cd} = 0.85[25/1.5] = 14.33 Mpa f_{ctd} = 1.5/1.5 = 1.00 Mpa 2. **High strength concrete (HSC)** Grade C-85 (Columns, Beams, slabs) Partial Safety Factor = 1.5 F_{ctk} =3.045 Mpa E_{cm} = 40.74 Gpa f_{cd} = 0.85[85/1.5] = 48.166 Mpa f_{ctd} =3.045/1.5 = 2.03 Mpa
6	Reinforcing Steel Strength	Partial Safety Factor =1.15 fy=400 Mpa f_{yd} = 400/1.15 = 347.82 Mpa Es = 200Gpa

4.5 MEMBERS CROSS-SECTIONAL PROPERTIES

The layout and size of members are very often controlled by architectural details, limit states requirements, structure cross-section properties such as: material strength and ductility requirement. The Preliminary dimension for structural design is given as in the following table.

Table 4.2: Cross sectional properties of the MRF Members

<table>
<tr><td colspan="4">i. Regular Moment Resisting Frame</td></tr>
<tr><td colspan="2">Section Dimension</td><td>NSC-R</td><td>HSC-R</td></tr>
<tr><td rowspan="3">Beam Dimension</td><td>Floor 1-6</td><td>500mm x350mm</td><td>450mm x300mm</td></tr>
<tr><td>Floor 6-10</td><td>500mm x300 mm</td><td>450mm x250mm</td></tr>
<tr><td>Floor 11 and 12</td><td>250mm x 200mm</td><td>250mm x200mm</td></tr>
<tr><td rowspan="3">Column Dimension</td><td>Floor 1-6</td><td>700mm x700mm</td><td>5500mm x550mm</td></tr>
<tr><td>Floor 6-10</td><td>500mm x500mm</td><td>400mm x400mm</td></tr>
<tr><td>Floor 11 and 12</td><td>400mm x400mm</td><td>350mm x350mm</td></tr>
<tr><td colspan="2">Slab Thickness</td><td>200 mm</td><td>150 m</td></tr>
<tr><td colspan="4">ii. Soft-Story (Stiffness Irregular) Moment Resisting Frame</td></tr>
<tr><td colspan="2">Section Dimension</td><td>NSC-SI</td><td>HSC-SI</td></tr>
<tr><td rowspan="3">Beam Dimension</td><td>Floor 1-6</td><td>500mm x350mm</td><td>450mm x300mm</td></tr>
<tr><td>Floor 6-10</td><td>500mm x300mm</td><td>400mm x250mm</td></tr>
<tr><td>Floor 11 and 12</td><td>350mm x350mm</td><td>250mm x200mm</td></tr>
<tr><td rowspan="3">Column Dimension</td><td>Floor 1-2</td><td>850mm x850mm</td><td>550mm x550mm</td></tr>
<tr><td>Floor 3-6</td><td>700mm x700mm</td><td>400mm x400mm</td></tr>
<tr><td>Floor 7-12</td><td>500mm x500mm</td><td>350mm x350mm</td></tr>
<tr><td colspan="2">Slab Thickness</td><td>200mm</td><td>150mm</td></tr>
</table>

4.6 LOAD CALCULATION

The actions (loads) on the structure are divided into two types: Permanent actions, and variables (imposed) actions, Permaneent action are those which are normally constant during the structure life. Permanent action include the weight of the structure itself and all archetectural componets such as exterior cladding, partions and ceilings and permanent fixtures. Variable actions, on the other hand, are transient and not constant in magnitude as for example those due to wind or human ocupant and seismic loads. Reccomendation for laods on the building are given in ES EN 1998-1-8-2015 and summarized in the table as follows.

Table 4.3 : Dead load and live load calcualtion

1. Dead load		
1.1 self-weight	The self-weight of the beam column , and slab is automatically calculated and applied by the program itself	
	i) Wall load on beam For 20cm thick HCB (approximate values)	14kN/m3* 0.2 * 2.9 + 0.04 * 2.9 * 23 = 10.8kN/m
	ii) Partition load on floors (average value)	((14kN/m3* 3.1 * 0.2 * 5) + (0.04 * 3.1 * 5 * 23)) / (4*5) = 2.35kN/m2
	Floor finish	0.03 * 23 = 0.69kN/m2
	Plastering	0.02 * 23 = 0.46kN/m2
	Cement screed	0.03 * 23 = 0.69kN/m2
	Total finishing load on floors:	0.69 + 0.46 + 0.69 = 1.84kN/m2
3. Live load		3 kN/m^2

4.7 SEISMIC ACTIONS

The most common and significant cause of earthquake damage to buildings is ground shaking; thus, the effects of ground shaking are the basis for most building code requirements for seismic design. The consideration of seismic hazards by the building codes is performed in a highly qualitative manner. The codes contain seismic hazard maps that divided the hazards into a series of zones of equivalent seismicity.

The classification of sites within the various zones was based on the historic seismicity of the region. If there were no historic reports of damaging earthquakes in a region it was classified as zone 0. If there were many large damaging earthquakes in an area, it was classified as zone 3, or zone 4.

For purpose of this study the building is decided to be located in Addis Ababa city, Ethiopia. Most of the applications of ES EN 1998, describe the hazard in terms of a single parameter, i.e. the value of the reference peak ground acceleration a_{gR}, derived from zonation maps found in the National Annex. The reference peak ground acceleration, chosen for each seismic zone, corresponds to the reference return period T_{NCR} of the seismic action for the no-collapse requirement (or equivalently the reference probability of exceedance in 50 years, P_{NCR}). An importance factor γ_I equal to 1.0 is assigned to this reference return period.

In cases of low seismicity, reduced or simplified seismic design procedures for certain types or categories of structures may be used. The selection of the categories of structures, ground types and seismic zones for which the provisions of low seismicity apply is found in the National Annex. It is recommended to consider as low seismicity cases either those in which the design ground acceleration on type A ground, a_g, is not greater than 0.08 g (0.78 m/s2), or those where the product $a_g \cdot S$ a is not greater than 0.1 g (0.98 m/s2).

The design ground acceleration $a_g = \gamma_I \cdot a_{gR}$).

Where: αg= the design ground/bed rock acceleration

αgR (αo)= the ratio of design bed rock acceleration to acceleration due to gravity

γ1 (I) =importance factor assigned to the reference return period

4.7.1 IMPORTANCE CLASSES AND IMPORTANCE FACTORS

Buildings are classified in 4 .4 importance classes, depending on the consequences of collapse for human life, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse.

Table 4.4 Importance classes for buildings

Importance class	Buildings
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II	Ordinary buildings, not belonging in the other categories
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

The value of γ_1 for importance class II shall be, by definition, equal to 1.0 (for ordinary building of reference return period associated with no-collapse requirement).

4.7.2 SEISMIC ZONE

For earthquake analysis, the country has been subdivided into different seismic zones depending on the local hazard. The hazard map is preliminary and is processed from an instrumentally recorded earthquake catalog. The seismic hazard map is divided into 5 zones, where the ratio of the design bedrock acceleration to the acceleration of gravity for the respective zone is indicated in table D1 of ES EN 1998-1: 2015.

Table 4.5: Bedrock acceleration ratio, a_o

Zone	1	2	3	4	5	0
$ao = \frac{\alpha g}{g}$	0.20	0.15	0.10	0.07	0.04	0

Thus for Addis Ababa (seismic zone 3), αo=0.1

The design ground/bed rock acceleration, α_g= 0.1*1.0 = 0.10

4.7.3 GROUND TYPES

Ground types A, B, C, D, and E, described by the stratigraphic profiles and parameters given in Table 4.1 and described hereafter, may be used to account for the influence of local ground conditions on the seismic action. This may also be done by additionally taking into account the influence of deep geology on the seismic action.

According the literatures review on seismic site specific hazard assessment done in Addis Ababa city is, the ground is taken as type C which described in table 3.1 of the code as “Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters as shown in the table 4.1.

Table 4.6: Ground Types

Ground Types	Description of stratigraphic profile	Parameters		
		vs,30 (m/s)	NSPT (blows /30cm)	cu (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800		
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand gravel or stiff clay with thickness from several tens to many hundreds of meters.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with vs values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with vs > 800 m/s.	<100 (indicative)		
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content		-	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S1			

4.7.4 RESPONSE SPECTRUM

A response spectrum is a plot of the peak or steady state response (displacement, velocity or acceleration) of a series of ground motion of varying natural frequency.

An elastic response spectrum is a function of frequency or period, showing the peak response of the building that is subjected to a ground motion. Response spectrum analysis is the method to estimate the structural response of short, non-deterministic, transient seismic event. The design response spectra (site specific response spectra) are an envelope over known geographical region used as an input to the response spectrum analysis.

Development of site-specific response spectra shall be based on the geologic, seismologic, and soil characteristics associated with the specific site. Response spectra should be developed for an equivalent viscous damping ratio of 5%. Additional spectra should be developed for other damping ratios appropriate to the indicated structural behavior

Within the scope of ES EN1998 the earthquake motion at a given point on the surface is represented by an elastic ground acceleration response spectrum, henceforth called an "elastic response spectrum".

For the horizontal components of the seismic action, the elastic response spectrum *S*e (*T*) is defined by the following expressions.

$$0 \leq T \leq T_{B:} \quad S_d(T) = a_{g.S}.\left[\frac{2}{3} + \frac{T}{TB} \cdot \left(\frac{2.5}{q} - \frac{2}{3}\right)\right]$$

$$T_B \leq T \leq T_C: \quad S_e\,(T) = a_g\,.S\,.\frac{2.5}{q}$$

$$T_C \leq T \leq T_D: \quad S_d(T) = \begin{bmatrix} ag.S.\frac{2.5}{q}\left(\frac{TC}{T}\right) \\ \geq \beta\,.ag \end{bmatrix}$$

$$T_D \leq T: \quad S_d(T) = \begin{bmatrix} ag\,.S\,.\frac{2.5}{q}\left(\frac{TC}{T^2}\right. \\ \geq \beta\,.ag \end{bmatrix}$$

Where:

$S_d(T)$ = is the design spectrum

q = is the behavior

β=is the lower bound factor for the horizontal design spectrum (0.2)
T = is the vibration period of a linear single-degree of freedom system

α_g= is the design ground acceleration on type A ground (ag= γαgR)
T_B= is the lower limit of the period of the constant spectral acceleration branch
T_C= is the upper limit of the period of the constant spectral acceleration branch
T_D= is the value defining the beginning of the constant displacement response range of the spectrum

S = is the soil factor

ɳ=is the damping correction factor with a reference value of ɳ=1 for 5% viscous damping.

Table 4.7: Values of the parameters describing the recommended type 2 elastic response spectra

Ground type	S	T_B(s)	T_C(s)	T_D(s)
A	1	0.05	0.25	1.2
B	1.35	0.05	0.25	1.2
C	**1.5**	**0.1**	**0.25**	**1.2**
D	1.8	0.1	0.3	1.2
E	1.6	0.05	0.25	1.2

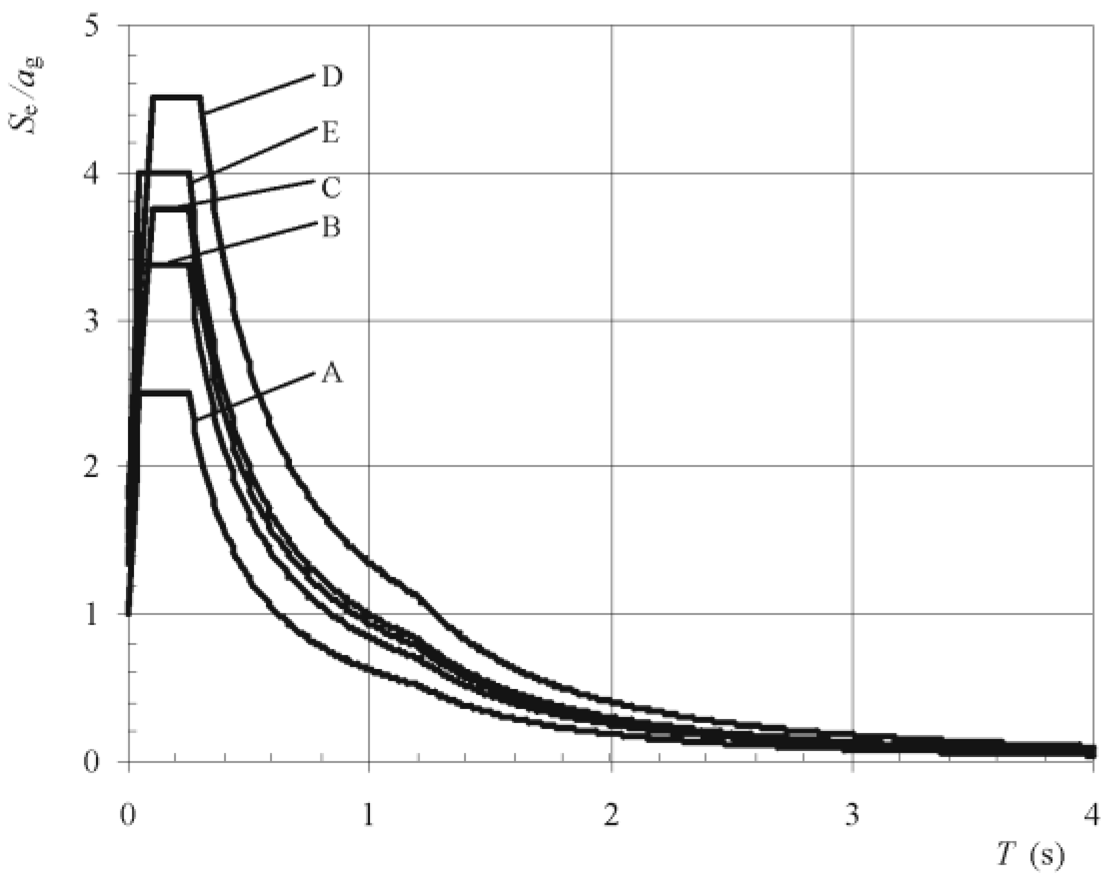


Figure 3.3: Recommended Type 2 elastic response spectra for ground types A to E (5% damping)

4.8 LOAD COMBINATION

Various load combinations of the characteristic values of permanent G_k,variable actions Q_k and seismic action and their partial factors of safety must be considered for the loading of the structure. The contribution of gravity loads to the effective seismic weight is obtained by the combination rule established in equation 3-17 of EN 1998-1 (2004) - section 3.2.4.

$$\sum G_{k,j} + \sum \psi_{Ei} . Q_{k,i}$$

$$\psi_{Ei} = \varphi . \psi_{2i}$$

Where:

ψ_{Ei} is the combination coeffcient of variable action (from table 4.2 ES EN 1998-1,2015)

The combination coefficients $\psi E,i$ take into account the likelihood of the loads Qk,i not being present over the entire structure during the earthquake. These coefficients may also account for a reduced participation of masses in the motion of the structure due to the non-rigid connection between them.

CHAPTER FIVE
PERFORMANCE ASSESSMENT OF THE MOMENT RESISTING FRAME

5.1 GENERAL

In the chapter four of this study the structural design and detailing of 4-different samples of MRF have developed, and this section discusses seismic performance assessment of MRF to simulate the seismic performance of buildings subjected to earthquakes. Performance assessment is the process used to determine the performance capability of a given building design. In performance assessment, engineers conduct structural analyses to predict building response to earthquake hazards, assess the likely amount of damage, and determine the probable consequences of that damage. Following performance assessment, engineers compare the ***predicted performance capability*** with the *desired performance objectives*. If the assessed performance is equal to or better than the stated performance objectives, the design is adequate. If the assessed performance does not meet the performance objectives, the design must be revised or the performance objectives altered, in an iterative process, until the assessed performance and the desired objectives match.

The paper focused on influence of soft-story (stiffness irregularity) by comparing the nonlinear static pushover and Incremental dynamic analysis (IDA) response of the Regular Moment resisting frame (NSC-R) as reference to those with soft-story (Stiffness Irregular). The paper study further discusses effect of different concrete strength on the seismic performance of the both regular and soft-storey MRF. The compressive concrete strength of 25Mpa is considered for NSC and 85Mpa for the HSC. Some modification of dimensions and reinforcements is made in order to take into account the contribution of increase in concrete strength. Finally seismic Vulnerability of RC MRF buildings is done by developing the fragility curve that provide probability of exceeding pre-determined performance levels as a function of earthquake intensity for a given region and a building type .

5.2 STRUCTURAL MODEL

The following section includes the modeling of the building according to the design and detailing given in the chapter four in the selected software-seismostruct (2018).

The structural design and detailing of the typical 12-story RC moment resisting frames shown in the Fig 5.1 are considered for the nonlinear analysis. The buildings have plan dimensions of 15m x 20m.

The column sections vary between smaller dimensions of 400mmx400 mm and larger column dimensions of 850x850 mm for regular NSC and smaller dimensions of 350mm x350 mm and larger column dimensions of 550 x550 mm for HSC regular and Irregular frame.

The beams sections vary between smaller dimensions of 250mm x200 mm and larger beams dimensions of 500 x350 mm for regular and Irregular NSC and smaller dimensions of 250mm X 200 mm and larger beams dimensions of 450 x300 mm for HSC regular and irregular frame. Different longitudinal reinforcement is introduced at the middle of the beam and at its two edges, while the transversal reinforcement consists of stirrup diameter 10 mm and with different stirrup spacing along the member length. The summary of the column and beam dimension and detailing dimension are given in the following figures from 5.1 through 5.4.

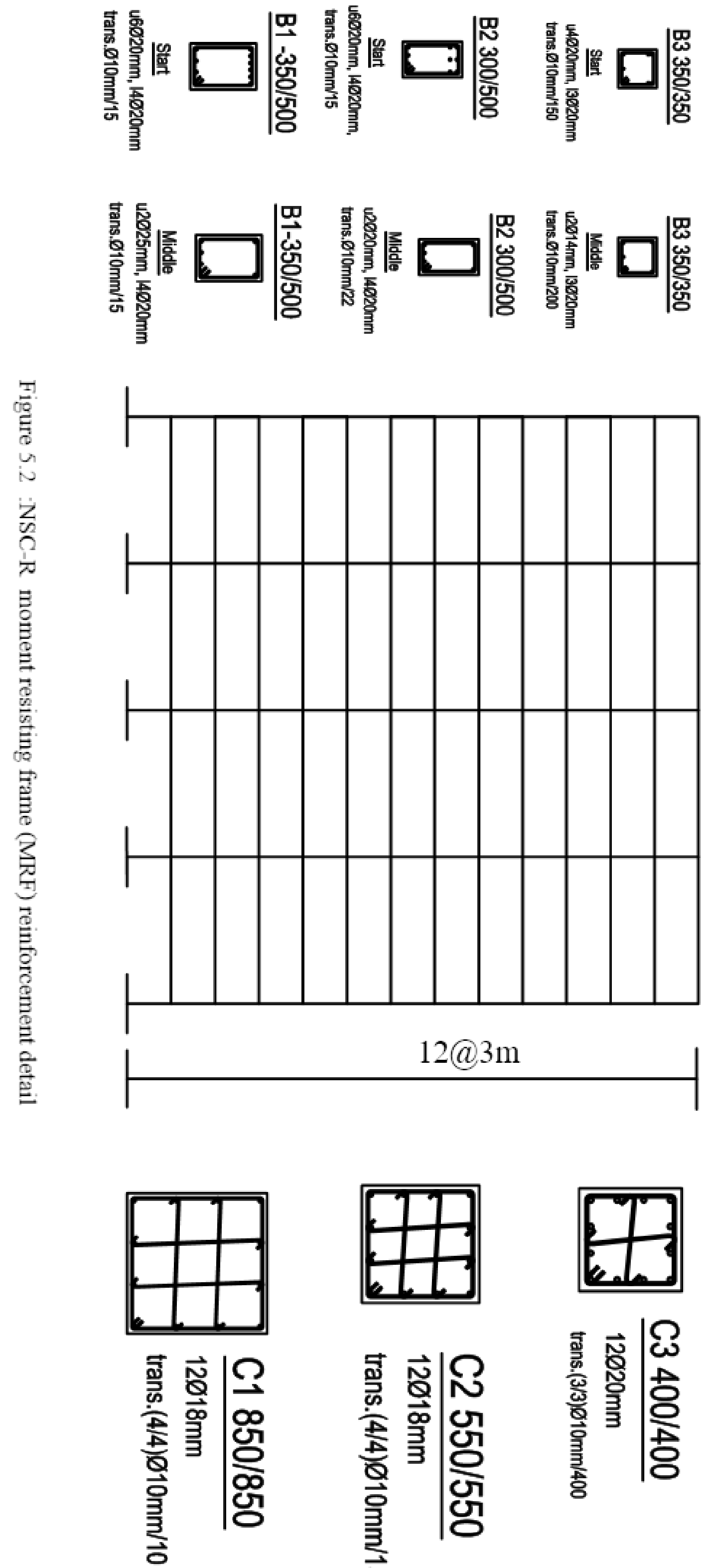


Figure 5.1: NSC-R moment resisting frame (MRF) reinforcement detail

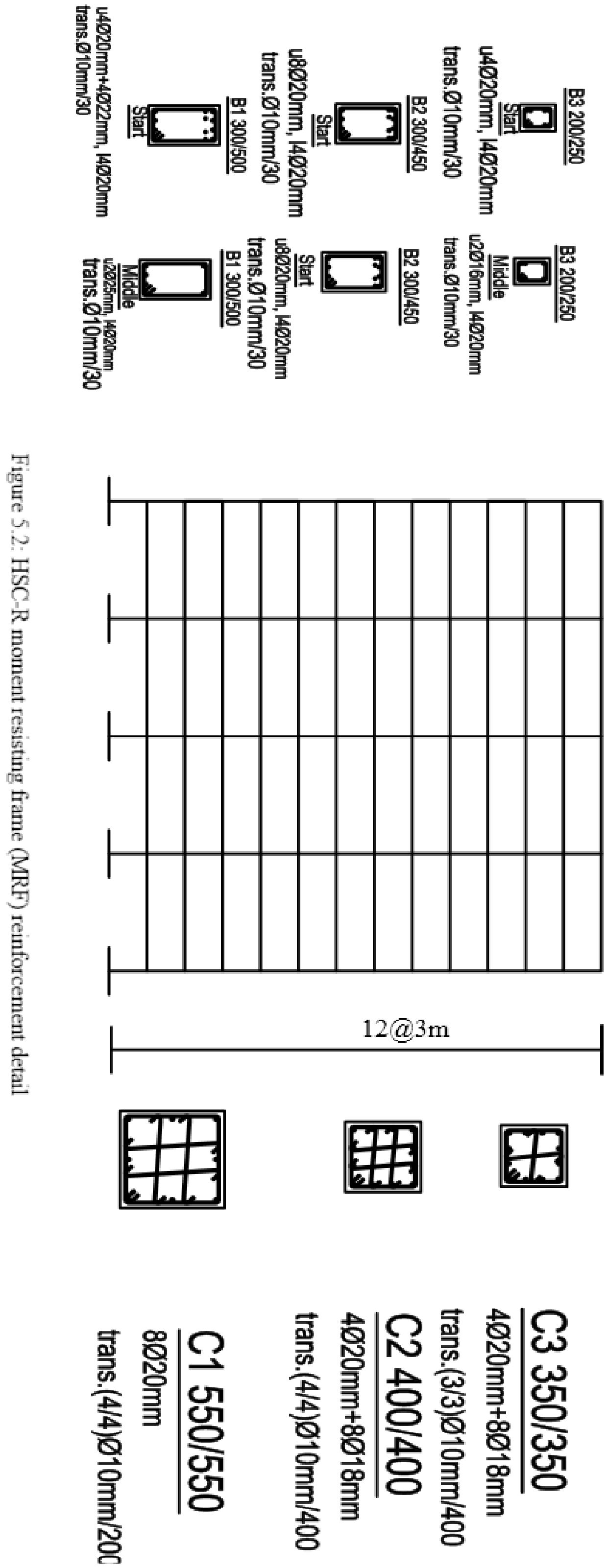


Figure 5.2: HSC-R moment resisting frame (MRF) reinforcement detail

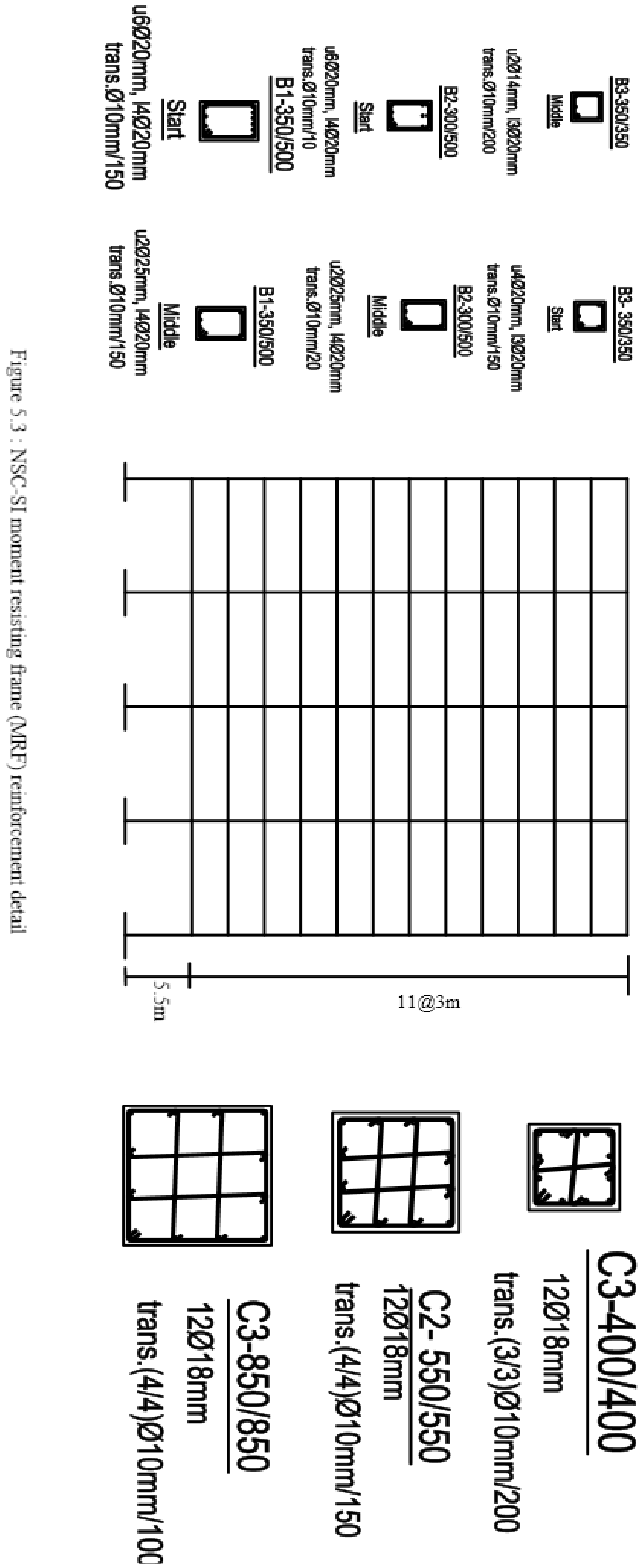


Figure 5.3: NSC-SI moment resisting frame (MRF) reinforcement detail

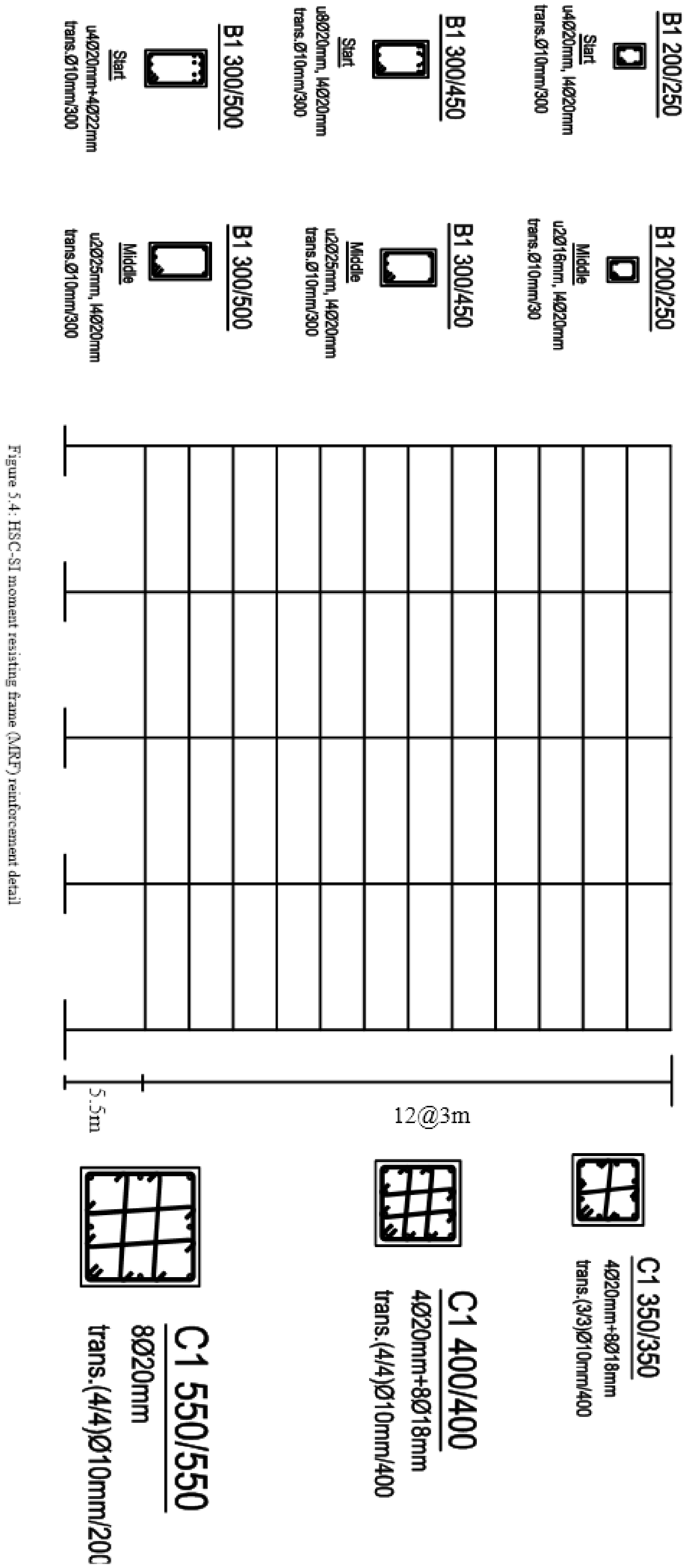


Figure 5.4: HSC-SI moment resisting frame (MRF) reinforcement detail

5.3 PERFORMANCE CRITERIA

It is paramount that analyst and engineers are capable of identifying the instants at which different performance limit states (non-structural damage, structural damage) collapse are reached. This can be efficiently carried out in the seismostruct through the definition of performance criteria module, whereby the attainment of given threshold value of material strain, sectional curvature, element curvature, element-chord rotation or element shear during the analysis of the structure is automatically monitored by the program.

The type of criteria to be used depends on the objective of the user. However, within the context of a fiber-based modeling approach such as its implemented in seismostruct, material strain do usually constitute the best parameter for identification of the performance state of given structure. The available materials strains are:

- **Cracking of structural elements** can be detected by checking for (positive) concrete strains larger than the ratio between the tension strength and the initial stiffness of the concrete material.
- **Spalling of cover concrete** can be recognized by checking for (negative) cover concrete strains larger than the ultimate crushing strain of unconfined concrete material. [typical value: -0.002]
- **Crushing of core concrete** can be verified by selecting the “Check the Core Only” check-box and checking for (negative) core concrete strains larger than the ultimate crushing strain of confined concrete material.
- **Yielding of steel** can be identified by checking for (positive) steel strains larger than the ratio between yield strength and modulus of elasticity of the steel material.

5.4 MATERIALS

Concrete materials are modeled according to the model developed by Mander et al (1988) for normal Strength Concrete (NSC). For high strength concrete (HSC) model developed by Kappos and konstantis are used as specified in the seismostruct software material definiton. The effective confinement effect is a function of section geometry, longitudinal and transversal reinforcement layout (Mander et al, 1988), which is automatically estimated by the software (Seismostruct) upon introduction of the longitudinal and transverse reinforcement.

The constitutive model used for the reinforcement steel is the one proposed by Menegotto and Pinto (1973) coupled with the isotropic hardening rules proposed by Filippou et al. (1983). The yield strength steel (rebar) of 400MPa considered for all buildings.

5.5 NONLINEAR *STATIC ANALYSIS (STATIC* PUSHOVER ANALYSIS)

The nonlinear static analysis is the method used in performance assessment of existing buildings. It is based on pushover analyses carried out under constant gravity loads and increasing lateral forces, applied at the location of the masses to simulate the inertia forces induced by the seismic action. As the model may account for both geometrical and mechanical nonlinearity, this method can describe the expected plastic mechanisms and structural damage.

The introduced vertical loads applied to the 3D model, in addition to incremental loads, are equal to 1.00G+0.30Q. The self-weight of the beam and column elements is automatically computed through their specific weight. The slabs' permanent and live loads are introduced as beams' additional mass automatically.

The Procedure for performing static non- linear analysis (Static pushover analysis) by Seismostruct software will be given as follows:

i. Creating model of the structure in the Seismostruct according to design and detailing using ETABS.
ii. Defining member behaviors such as: Beams: moment-rotation relations, Columns: moment-rotation and interaction diagrams. Beam-column joints: assume rigid and special links to extra members.
iii. Defining Gravity load as the predefined lateral load pattern, which is consists of dead load plus a portion of live load. (1.00G+0.30Q)
iv. Applying incremental Lateral loads until numerical collapse occurs.
v. Plot the Roof displacement against base shear is (capacity curve) at the stage when numerical collapse occurred.

5.6 NON-LINEAR TIME-HISTORY ANALYSIS

In this method, the seismic response of the structure is evaluated using step-by-step time history analysis. In nonlinear dynamic analyses, the detailed structural model subjected to a ground-motion record produces estimates of component deformations for each degree of freedom in the model. The main methodology of this procedure is almost similar to the static method of analysis. However, this approach differs in a concept that the design displacements are not established using the target displacement; but, is estimated through dynamic analysis by subjecting the building model to a series of the ground motions. Accordingly, the non-linear dynamic analysis method is adopted for the analytical study due to its accuracy and efficiency in determining the inelastic seismic response of a system subjected to the ground motion data. One method non-linear dynamic method is Incremental dynamic analysis.

5.6.1 INCREMENTAL DYNAMIC ANALYSIS (IDA)

IDA involves performing a series of nonlinear dynamic analyses in which the intensity of the evaluated ground motion is monotonically increased until the structure's collapse limit state is reached. A measure of the ground motion intensity is then plotted against a selected engineering demand parameter (EDP), such as roof displacement, roof drift and maximum interstory drift ratio. In the current investigation the maximum Top Storey drift, is used as a damage measure (EDP) and 5% damped spectral acceleration is used as an intensity measure (IM).The global collapse capacity is reached when the curve in this plot approaches a horizontal slope, indicating that a small increase in the ground motion intensity generates a large increase in the structural response. The paper also extend to developing fragility response of reinforced concrete frame to assess vulnerability of the structures for both regular structural layout and Stiffness Irregular (Soft-Story) moment resisting frame based on the results of incremental dynamic analysis (IDA).

To perform IDA 20 synthetic earthquake records representative of Addis Ababa site were generated using Seismoatif (2018). An increment of 0.05g, in PGA, is used to generate the earthquake records in seismoartif (2018). Using these generated earthquake record; IDA was carried out with an increasing level of IM till dynamic instability occurs with increasing intensity measure (IM).

5.6.2 GROUND MOTION

The assessment of performances and demands for moment resisting frames (MRF) necessitated the availability of a set of acceleration time histories with amplitude, frequency content, and duration enclosed into certain limits in order to reduce the dispersion of the corresponding demand parameters. However, in most cases, using time histories from actual earthquake data has many limitations for many reasons. Hence, artificial time history sets, generated from response spectra, are widely used instead. ES EN 1998-1:2015 recommends using artificial accelerograms for seismic motion input depending on the information available and nature of application. It stipulates that artificial accelerograms shall be generated so as to match the elastic response spectra used in the design for 5% viscous damping. Also it has been stated that with the absence of site specific data, the minimum duration of stationary part of the accelerograms should be equal to 10sec; and a minimum of 3 accelerograms should be used.

Accordingly, twenty artificial earthquake records compatible with the Addis Ababa soil is generated with help of Seismoartif (2018) for the Incremental dynamic analysis and shown in figure 5.5

The Procedure for performing Incremental dynamic analysis (IDA) by Seismostruct software will be given as follows:

i. Creating model of the structure in the Seismostruct according to design and detailing using ETABS.
ii. Defining member behaviors such as: Beams: moment-rotation relations, Columns: moment-rotation and interaction diagrams. Beam-column joints: assume rigid and special links to extra members.
iii. Define and select an appropriate ground motion record compatible with the selected site
iv. Define a multiple scale factors applied to the selected ground motion record.
v. Scale the selected ground motion record in order to create a set of ground motion records that will be applied to the structure starting from elastic range response to collapse.
vi. Perform response history analysis of the structural model subjected to the scaled ground motion record
vii. Estimate the DM corresponding to the scaled IM

CHAPTER SIX
RESULT AND DISCUSSION

6.1 INTRODUCTION

This chapter presents analytical results evaluated by nonlinear static (push over) analysis, incremental dynamic analysis (IDA) and fragility curves to check the performance of MRF buildings.

Nonlinear static analysis and Dynamic inelastic response history analyses were conducted using computer software Seismostruct (2018). Incremental dynamic analyses were performed under artificial earthquake records compatible with Addis Ababa site response spectra with increasing seismic intensity, as the intensity measure. From Nonlinear static analysis (pushover analysis) Capacity curves are developed for each case that was previously designed and for incremental dynamic analysis IDA curves are developed to check the collapse capacity of the building under seismic excitation.

Vulnerability assessment of reinforced concrete frame buildings with regular and stiffness irregular will be presented as probability of exceedance of given performance levels for given intensity levels. Incremental dynamic analysis results used as input data to generate fragility curves that can be used as analytical tools for further seismic vulnerability assessment. Accordingly, fragility curves were developed and presented for all MRF cases, and compared to each other to learn how the stiffness irregularity and compressive concrete strength variations affect the seismic performance of the RC moment resisting frame (MRF).

6.2 NON-LINEAR STATIC ANALYSIS (PUSHOVER) RESULTS

Results from Push-Over analysis are presented as Base shear and Top drift (capacity curve) for both Regular and stiffness Irregular as follows in figure 6.1 to 6.12. Capacity curves (base shear versus roof displacement) are the load - displacement envelopes of the structures and represent the global response of the structures. The capacity curve gives an insight of the maximum base shear that the structure can resist. In the present study capacity curves of buildings were obtained from the pushover analyses using SeismoStruct software. Firstly the curves are analyzed separately in order to get the behavior of each one of the considered buildings and to obtain the limit state capacities. Then a comparative analysis is done for all models in order to check seismic performance of the RC moment resisting frame building during an earthquake (under seismic excitation).

6.2.1 REGUALR MOMENT RESISTING FRAME (MRF)

Figures 6.1 to 6.5 present the deformed shape and hinge formation and pushover (load - displacement curve) results of the regular moment resisting frame for both Normal strength (NSC) and high strength concrete (HSC).

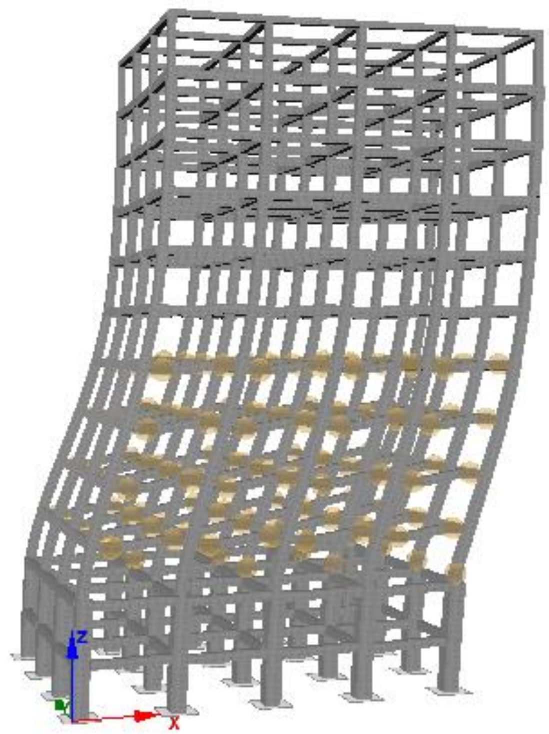
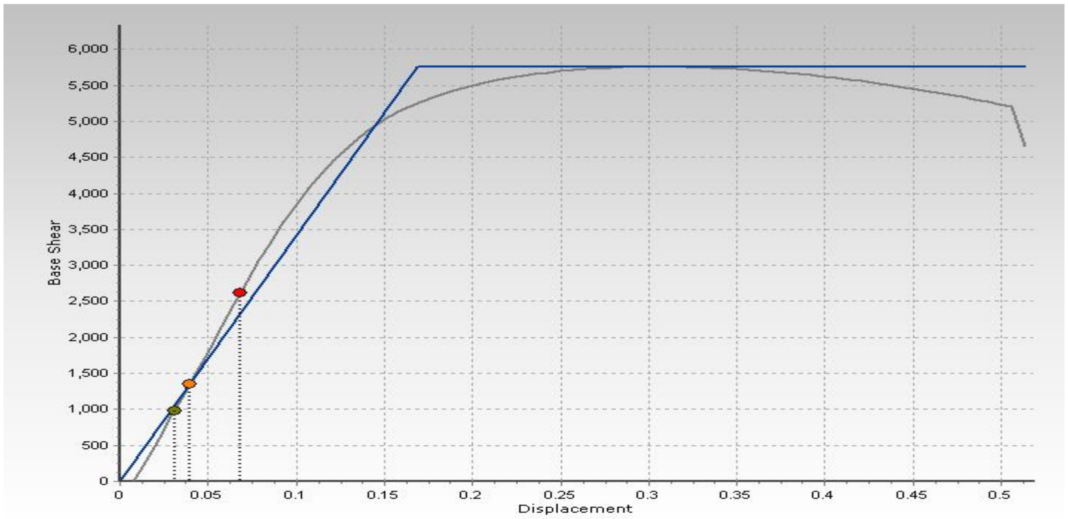


Figure 6.1: Deformed shape and plastic formation for NSC-R



a) Figure 6.2 : Pushover –load –diplacement curve for NSC-R MRF

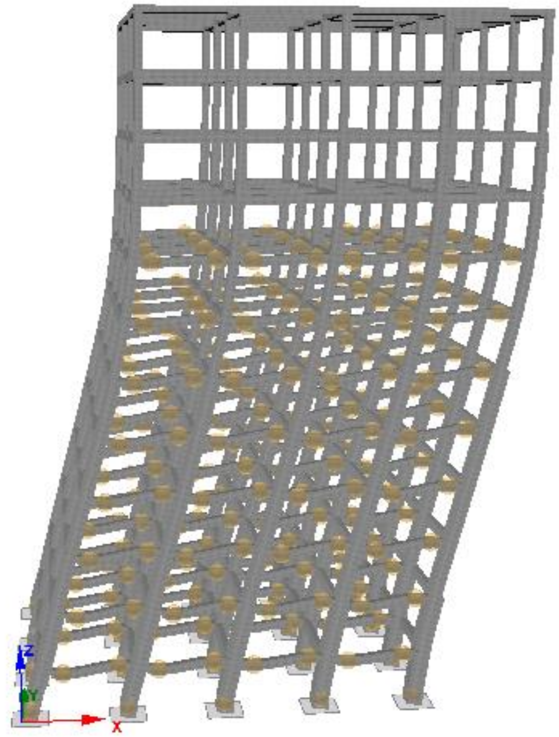
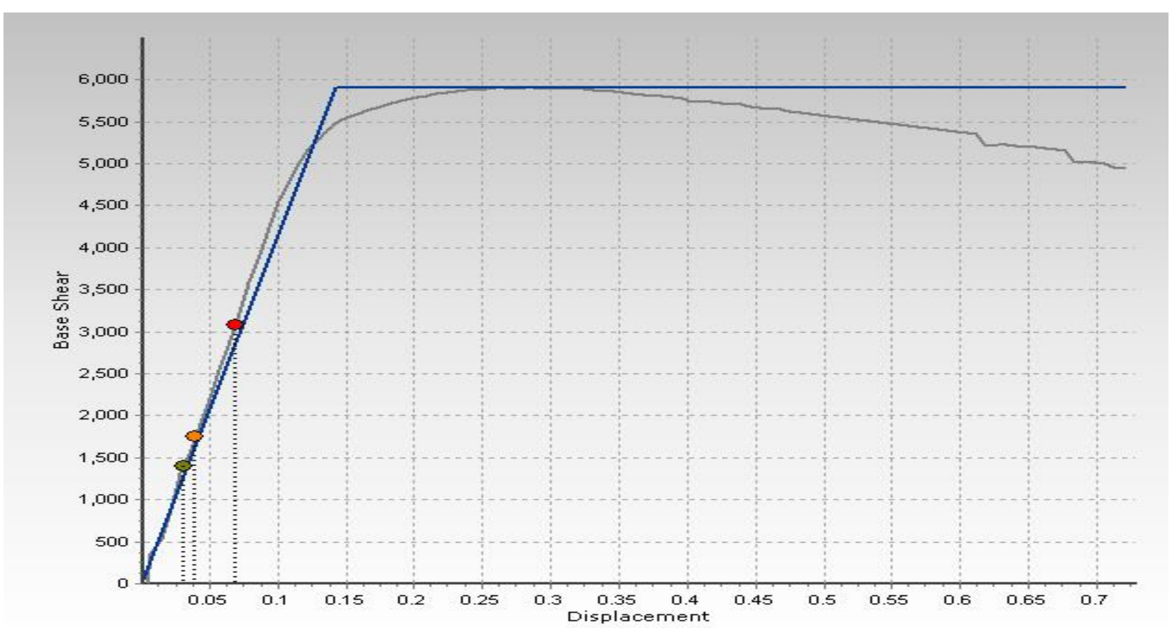
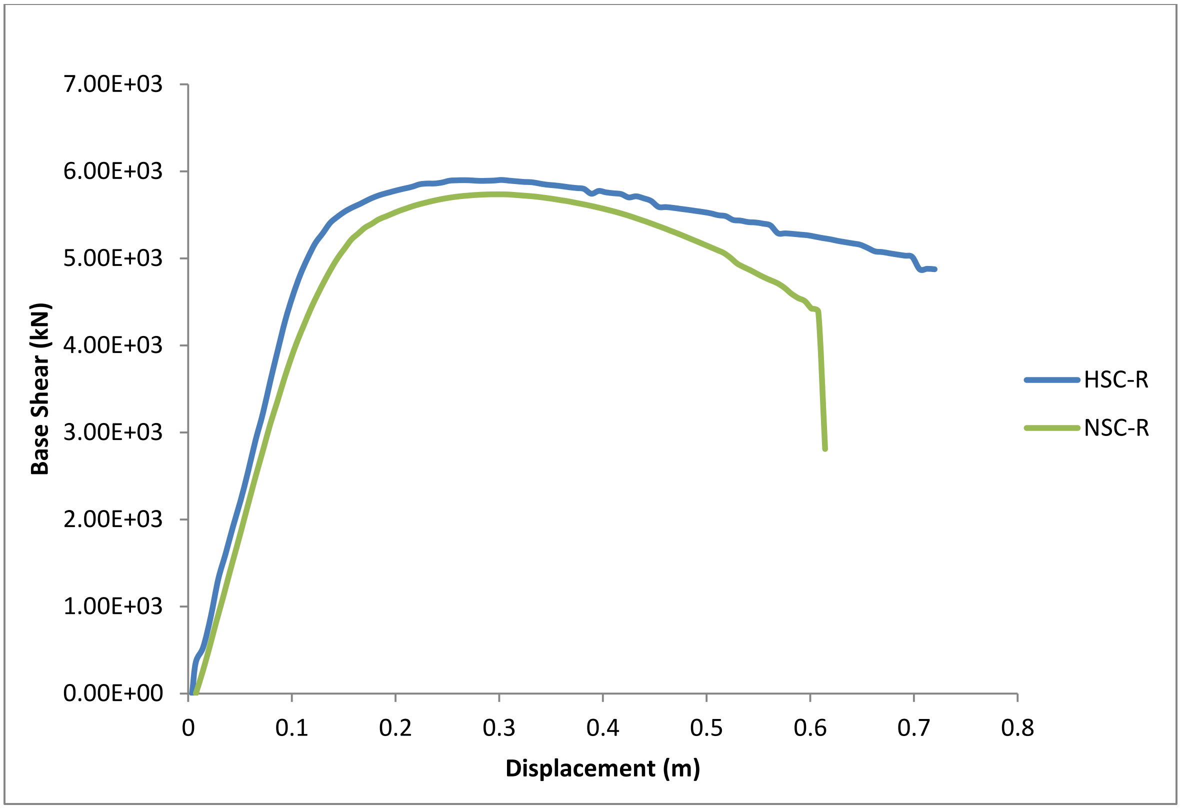


Figure 6.3: Deformed shape and plastic formation for HSC-R



b) Figure 6.4 : Pushover –load –diplacement curve for HSC-R MRF



c) Figure 6.5 : Pushover result:- load –diplacement curve for NSC-R and HSC -R on one graph.

The maximum base shear and the corresponding top displacement are summarized in the table 6.1 for regular moment resisting frames using both normal strength (NSC) and high strength concrete(HSC).

Table 6.1 maximum Top displacement vs. Maximum Base shear of regular MRF

Type of MRF	REGULAR MRF		Deviation of yield displacement from Baseline yield displacement in percent	Deviation of Maximum base shear from Baseline case in percent
	Top Drift	Yield Strength		
NSC-R	0.1645	5735.86	0	0
HSC-R	0.1416	5900.88	-13.92097264	2.876987932

Is it can be seen from the table 6.1 as well as from the figure 6.1,6.2 and 6.3 the ultimate collapse base shear capacity of regular MRF for high strength concrete (HSC-R) increases by 2.88% compared to NSC-R MRF. The value roof displacement at ultimate collapse shear capacity reduced by 13.92% for HSC-R compared to NSC-R. The decrease in collapse displacement shows that when using HSC, the frames **fail earlier** than the NSC frame because the concrete is less ductile as its strength increases.

6.2.2 SOFT-STORY (STIFFNESS IRREGULAR FRAME)

Figures 6.6 to 6.10 present the pushover (load –displacement curve) results of the stiffness irregular (soft-story) moment resisting frame for both Normal strength (NSC) and high strength concrete (HSC).

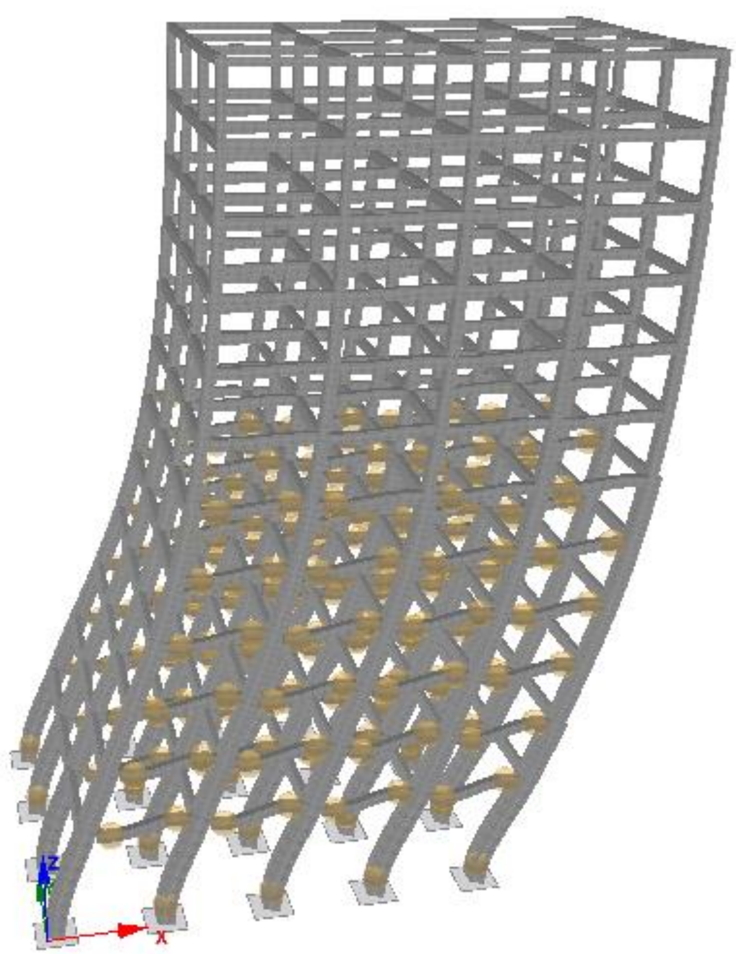
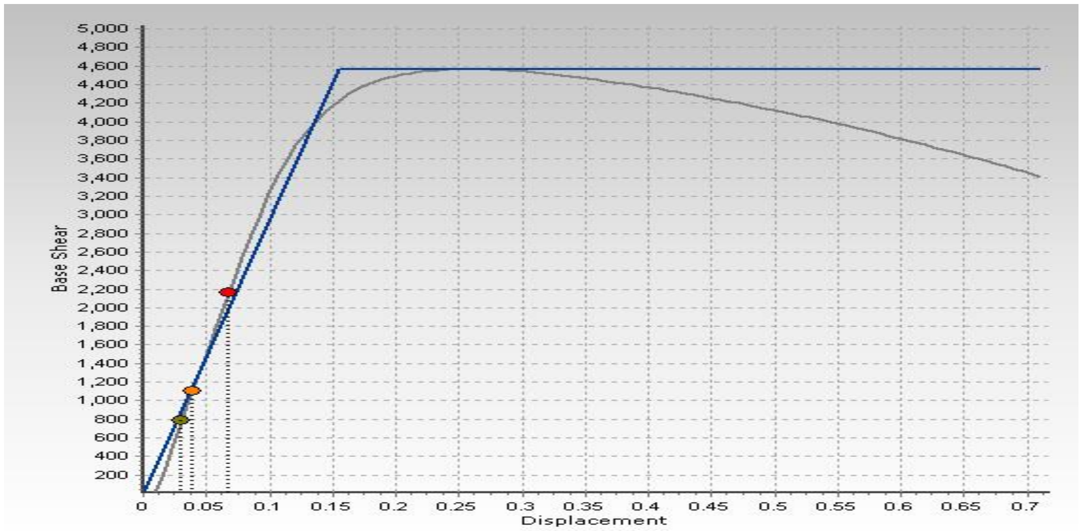


Figure 6.6: Deformed shape and plastic formation for NSC-SI



a) Figure 6.7 :Pushover –load –diplacement curve for NSC-SI MRF

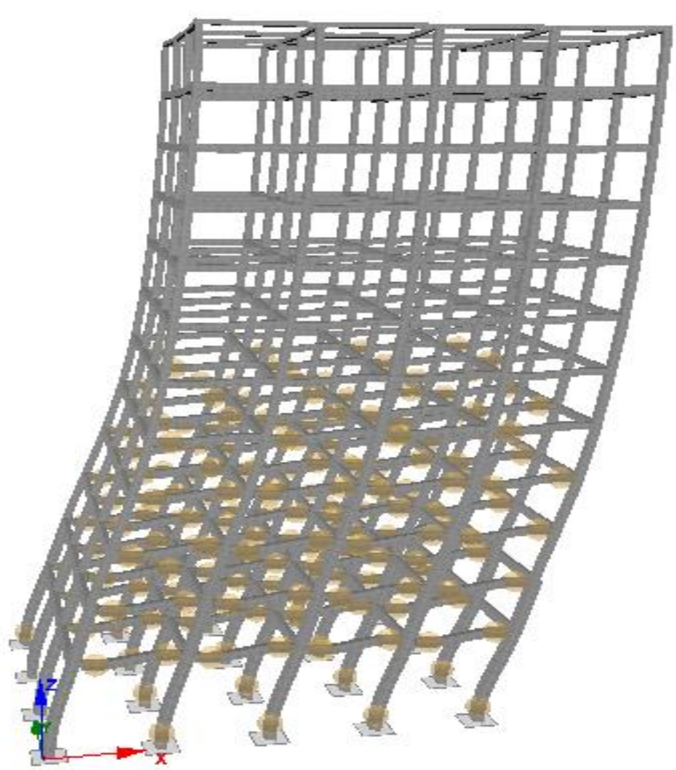
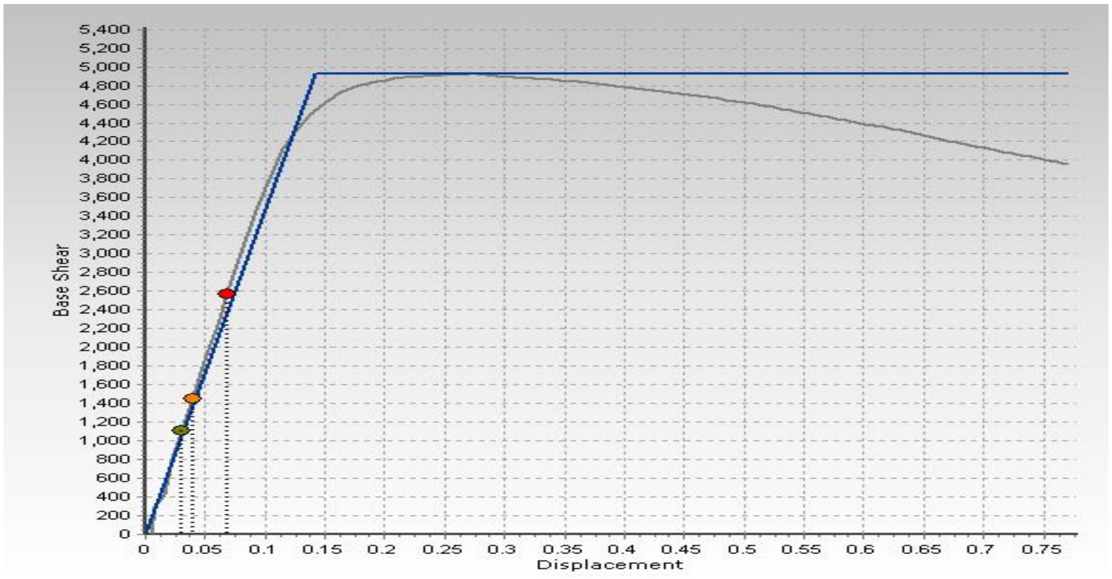


Figure 6.8: Deformed shape and plastic formation for HSC-SI



b) Figure 6.9: Pushover –load –diplacement curve for HSC-SI MRF

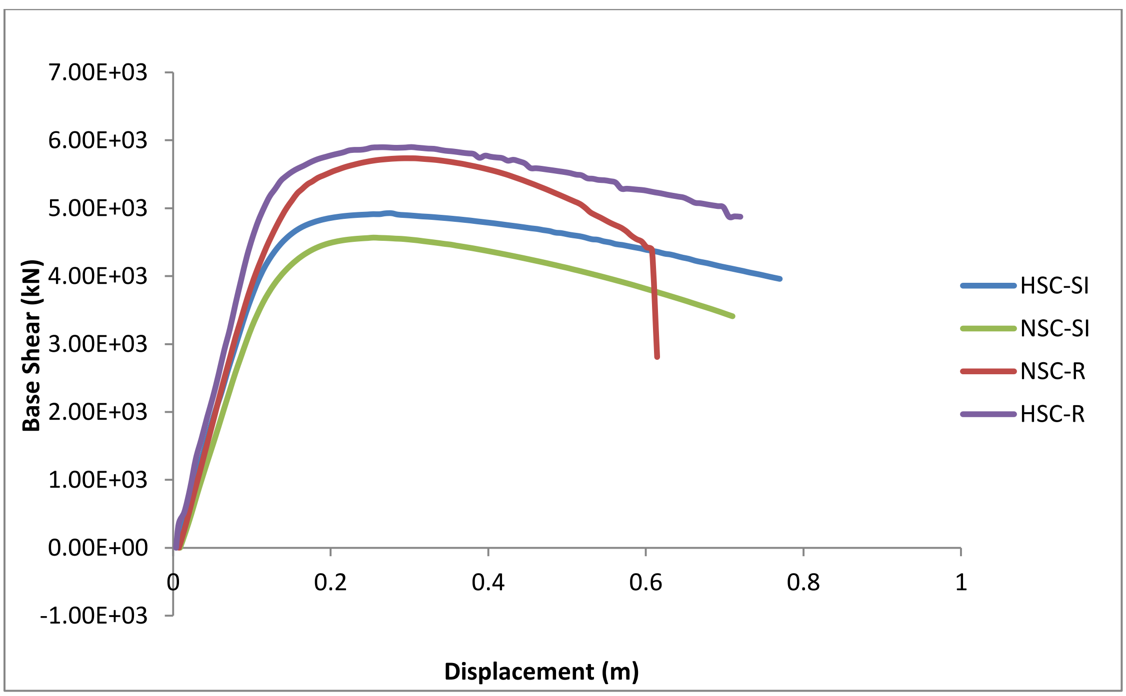


Figure: 6.10: Pushover results of Regular and soft-story MRF

Table 6.2: Maximum Base Shear vs. Top drift for all cases

Type of MRF	REGULAR MRF		Deviation of yield displacement from Baseline yield displacement in percent	Deviation of Maximum base shear from Baseline case in percent
	Top Drift	Yield Strength		
NSC-R	0.1645	5735.86	0	0
HSC-R	0.1416	5900.88	-13.92097264	2.876987932
SOFT- STORY MRF				
NSC-SI	0.1552	4567.48	-5.653495441	-20.36974403
HSC-SI	0.1422	4926.53	**-13.556231**	**-14.11000268**

***Note: Negative sign shows decrease in values from the baseline case*.**

Figure 6.5, 6.6, 6.7, and 6.8 show the pushover curves of soft story (stiffness irregular) moment resisting frame. From table 6.2 it can be understood that the collapse shear capacity of NSC-SI MRF is decreased by 20.37 % compared to NSC-R MRF. Figure 6.8 and table 6.2 shows that the collapse shear capacity of the HSC-SI is decreased by 14.11 % whereas the roof displacement at the collapse shear capacity is reduced by 13.56% compared to the NSC-R. The value of roof displacement at ultimate collapse shear capacity reduced by 7.91% for HSC-SI compared to NSC-SI whereas the shear collapse capacity of the HSC-SI is increased by the 6.26%.

6.3 INCREMENTAL DYNAMIC ANALYSIS (IDA) RESULTS

Incremental dynamic analysis (IDA) was also used to compare the performance of the building. Figure 6.8 to 6.12 shows the comparison of IDA results for both regular and stiffness irregular (soft story) moment resisting designed for NSC and HSC. The IDA results from Seismostruct (2018) model are summarized in the figures below.

6.3.1 IDA CURVE REGUALR MOMENT RESISTING FRAME (MRF)

The peak roof drift vs. maximum base shear is given in the figure 6.8 for NSC-R Moment resisting frame. The peak roof displacement of 0.14288m occurred at the maximum Base Shear of 5808.461 kN.

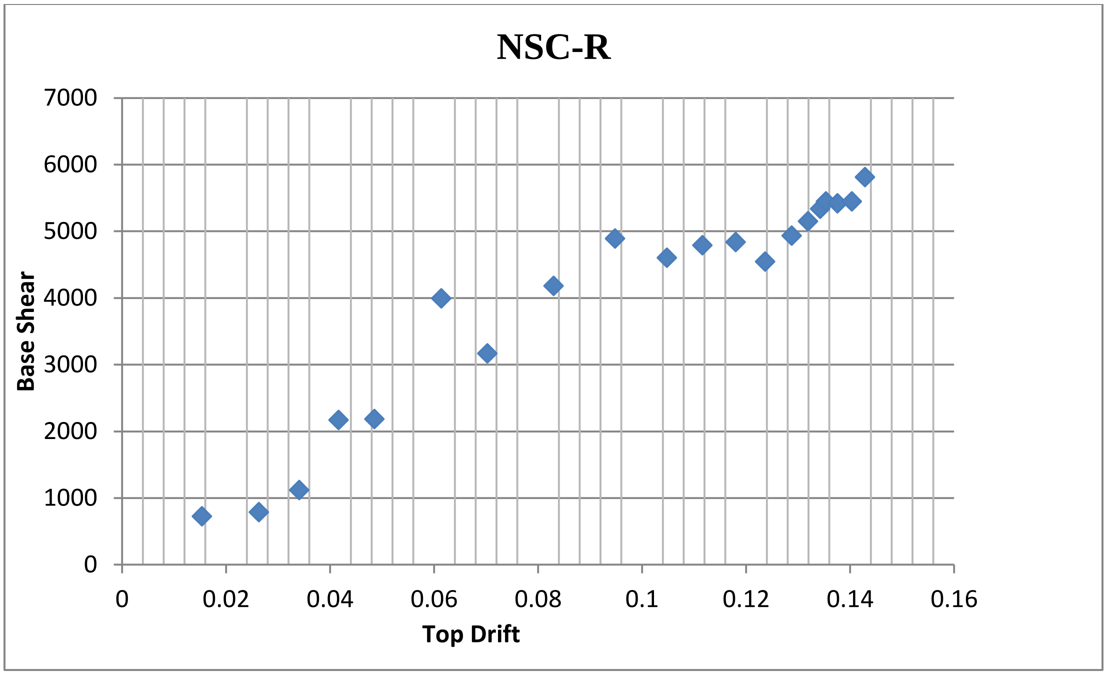


Figure 6.11: IDA capacity curve for NSC-R MRF

The peak roof drift vs. maximum base shear is given in the figure 6.9 for HSC-R Moment resisting frame. The peak roof displacement of 0.15533m occurred at the maximum Base Shear of 6194.93kN. The maximum base shear has increased by 6.653% for HSC-R compared to NSC-R, whereas the peak roof drifts increase by 8.71%.

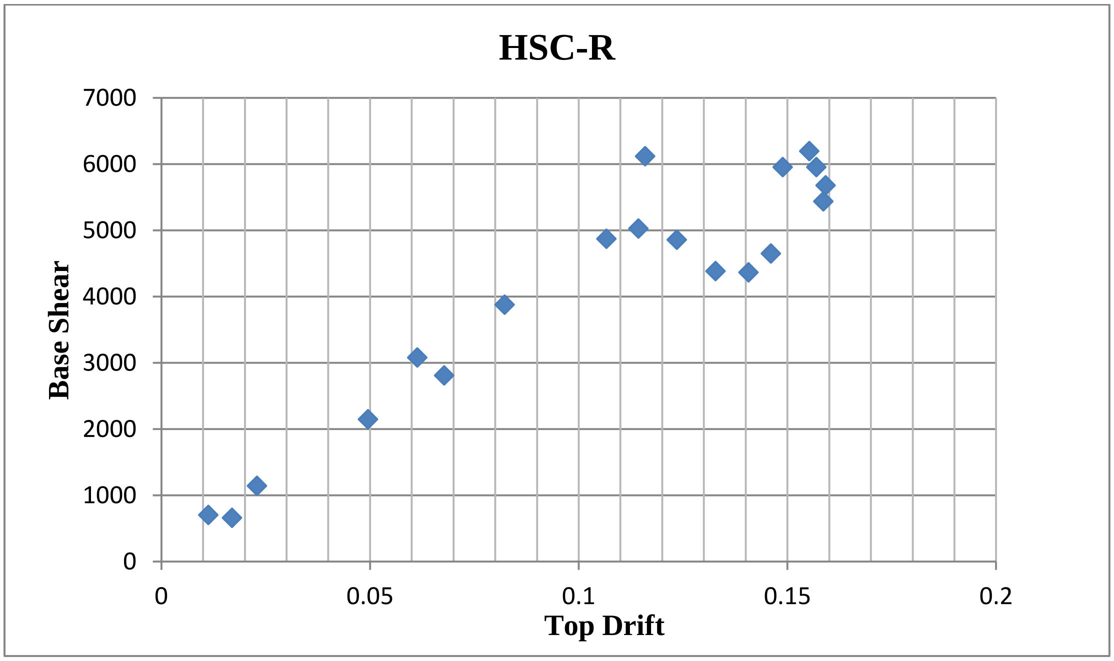


Figure 6.12: IDA capacity curve for HSC-R MRF

6.3.2 IDA CURVE FOR SOFT-SRORY (STIFNESS IRREGULAR) MRF

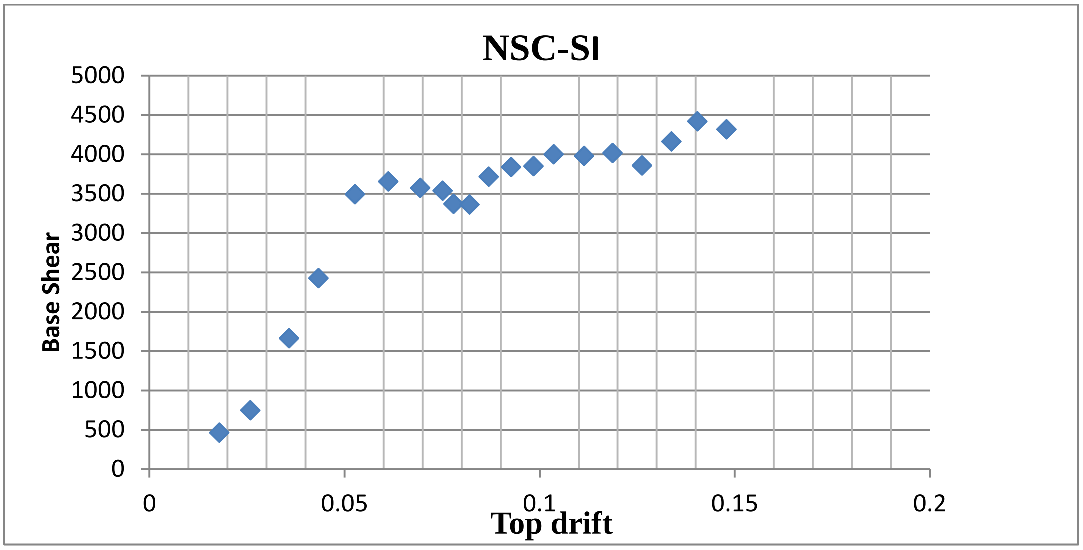


Figure 6.13: IDA capacity curve for NSC-SI MRF

The peak roof drift vs. maximum base shear is given in the figure 6.9 for NSC-SI Moment resisting frame. The peak roof displacement of 0.14051436m occurred at the maximum Base Shear of 4416.173 kN. The maximum base shear has decreased by 23.97% for NSC-SI compared to NSC-R, whereas the peak roof drifts increase by 1.684%.

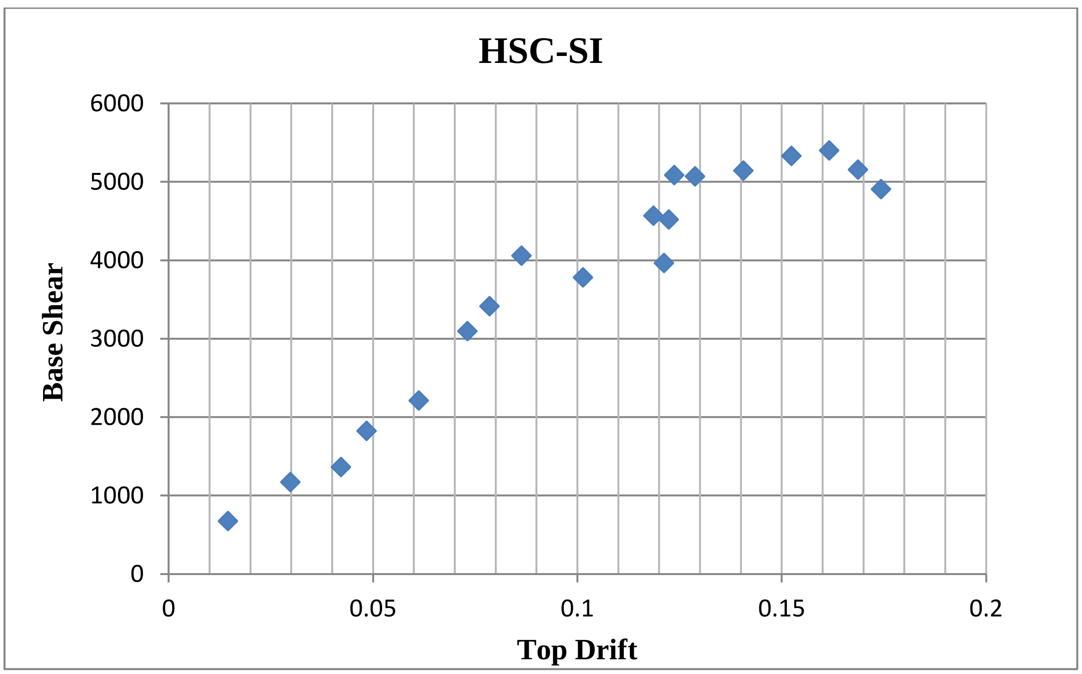


Figure 6.14: IDA capacity curve for HSC-SI MRF

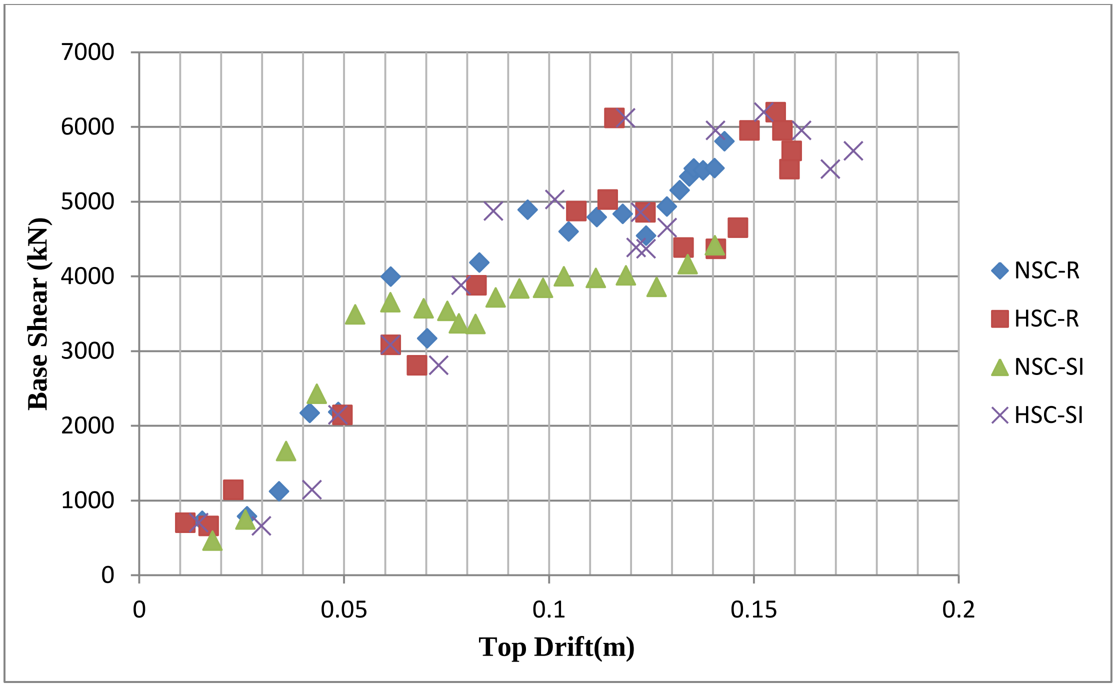


Figure 6.15: IDA capacity curve for all moment resisting (MRF)

The peak roof drift vs. maximum base shear is given in the figure 6.10 for HSC-SI Moment resisting frame. The peak roof displacement of 0.16165m occurred at the maximum Base Shear of 5401.3 kN. The maximum base shear has decreased by 7.01 % for HSC-SI compared to NSC-R, whereas the peak roof drifts increased by 13.134%.

6.4 PROBABILISTIC SEISMIC DEMAND MODEL (PSDM)

The relationship between ground motion intensities (Peak ground accelerations) and engineering demand parameter (EDP) which is global roof in this study is developed as a Probabilistic Seismic Demand Model (PSDM) and given in the figures 6.12 to 6.15. The maximum roof drifts (roof demand) for each 20 input ground motions at each roof level of MRF are obtained from IDA are shown below in Table 6.3. Regression analysis is conducted to obtain the best fit curve that represents the PSDM and the Constants ‘a’ and ‘b’ of the equation 3.4 are obtained from the best fit curve.

Table 6.3: Roof drift vs. peak ground acceleration of each MRF

PGA(g)	Roof Drift (mm)			
	NSC-R	HSC-R	NSC-SI	HSC-SI
0.1	20.39438	11.30293	17.92783	14.58471
0.15	26.56994	16.98234	25.92791	29.86732
0.2	31.24001	22.95948	35.86264	42.22463
0.25	32.25455	49.62522	43.35534	48.49101
0.3	37.89753	61.43132	52.72131	61.30022
0.35	43.69615	67.83461	61.28715	73.14883
0.4	52.05109	82.31842	69.47446	78.58481
0.45	59.4921	106.6707	75.25196	86.43741
0.5	67.00634	114.387	78.02411	101.4489
0.55	71.45906	115.9538	82.09646	118.7204
0.6	80.76322	123.5949	87.05603	122.4135
0.65	88.27391	132.8379	92.79989	121.3023
0.7	93.30343	140.7363	98.53968	123.7437
0.75	101.034	146.1245	103.6722	128.8817
0.8	111.3641	148.9105	111.4338	140.6867
0.85	120.0554	155.3286	118.771	152.4558
0.9	128.4194	156.9968	126.2795	161.652
0.95	135.0269	158.6978	133.8982	168.7279
1	142.6366	159.2452	140.5144	174.3659

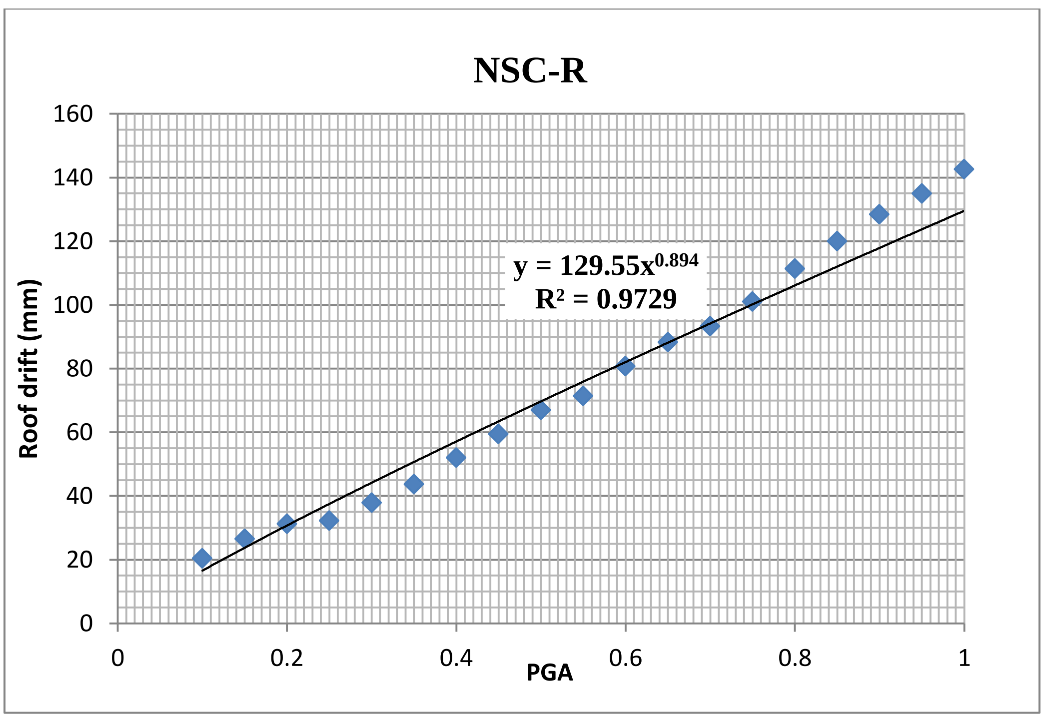


Figure 6.16: Probabilistic Seismic Demand Model of NSC-R

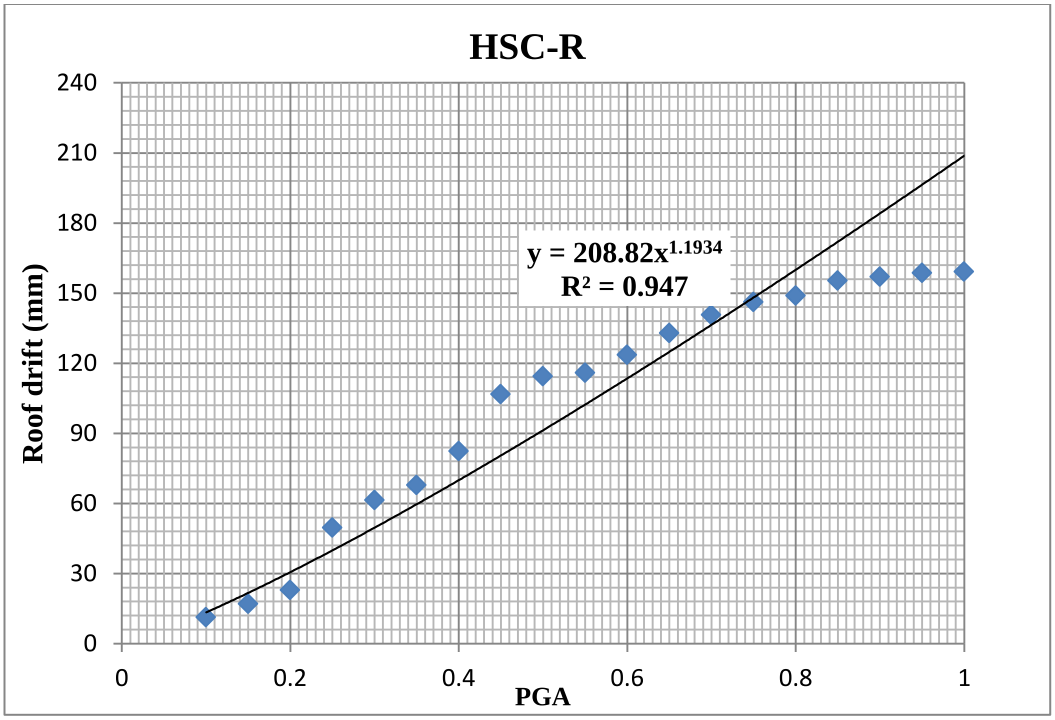


Figure 6.17: Probabilistic Seismic Demand Model of HSC-R

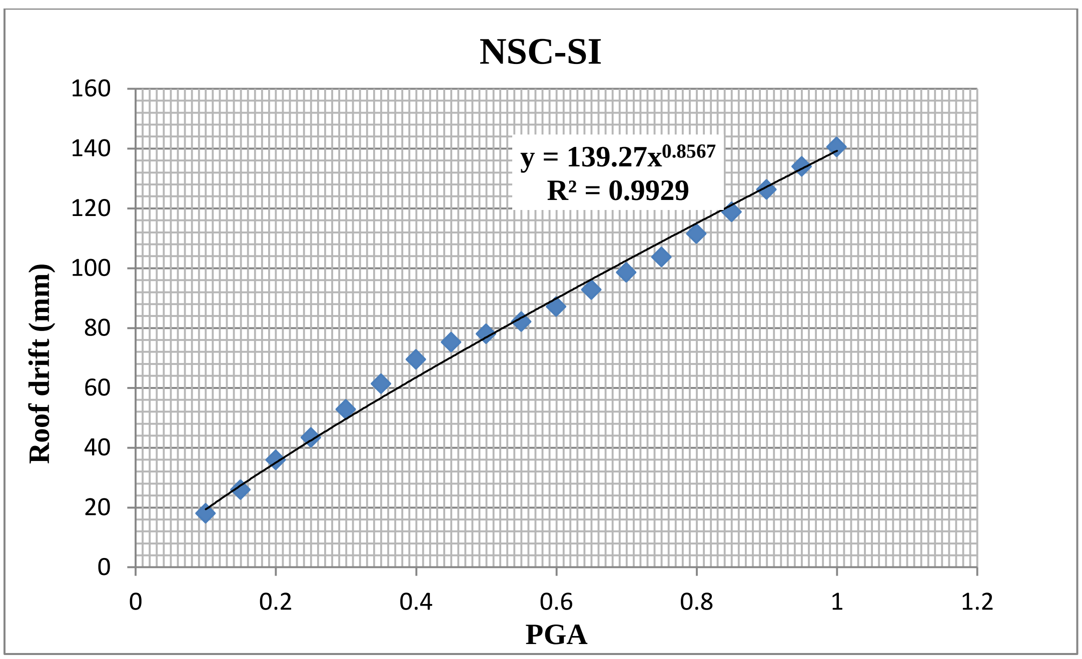


Figure 6.18: Probabilistic Seismic Demand Model of NSC-SI

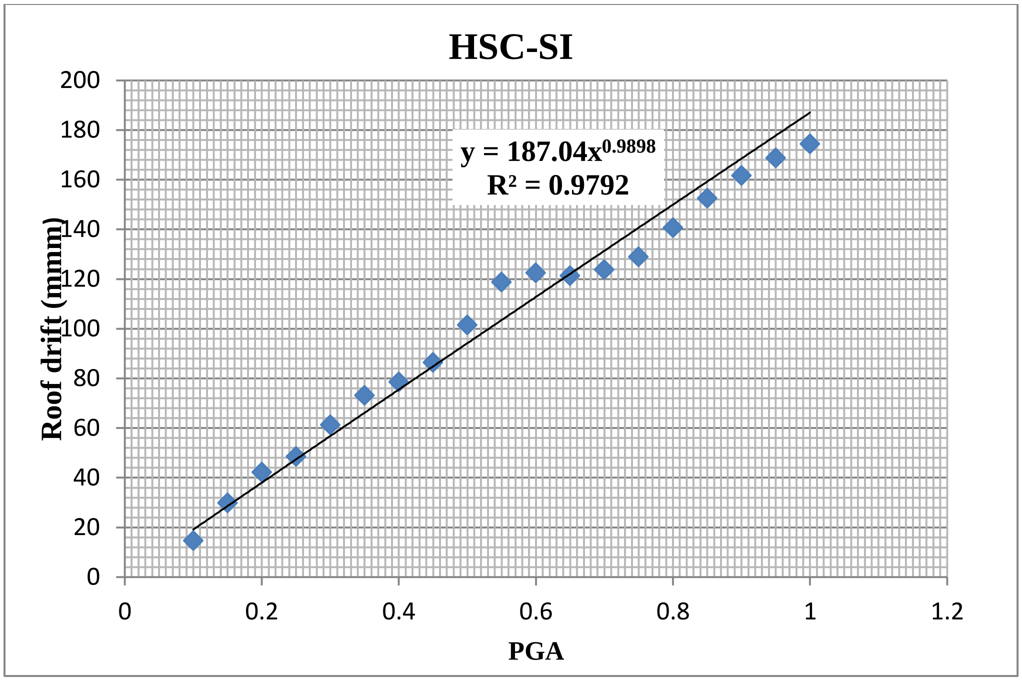


Figure: 6.19: Probabilistic Seismic Demand Model of HSC-SI

Mean and Standard deviation (dispersion) of the engineering demand parameters are calculated as shown in the table 6.4.

Table 6.4 Mean and stand deviation (dispersion) of (EDP)

Building Type	Mean ln (EDP)	$\beta_{EDP/PGA}$
NSC-R	4.177884	0.63316986
HSC-R	4.425488	0.787423197
NSC-SI	4.278861	0.552022204
HSC-SI	4.471609	0.642235056

6.5 FRAGILITY CURVE

After the relation between ground motion intensities (Peak ground accelerations) and engineering demand parameter (EDP) is established and dispersions ($\beta_{D/PGA}$, β_c, and β_m) for all the building models are computed, fragility curves for various performance levels are developed using the equation 3.5. Accordingly, fragility curves of all building cases are presented in the figures 6.16, 6.17 and 6.18 considering for different IO, SD and NC performance levels respectively.

Limit state capacities of moment resisting frames are obtained from the pushover analysis and presented in the table 6.5. Finally by combining the limit state capacities (d_c), dispersion and the mean engineering demand the percentages probability of exceedance of the given limit state capacities are calculated and shown in table 6.6.

Table 6.5 Limit state capacity of each MRF

Performance level (limit state capacities	NSC-R	HSC-R	NSC-SI	HSC-SI
Damage Limitation (IO)	29.002	34.90564	25.06958	30.7376
Significant Damage (SD)	39.5	39.64679	25.06958	30.7376
Near Collapse (NC)	72.76	68.73352	25.06958	30.7376

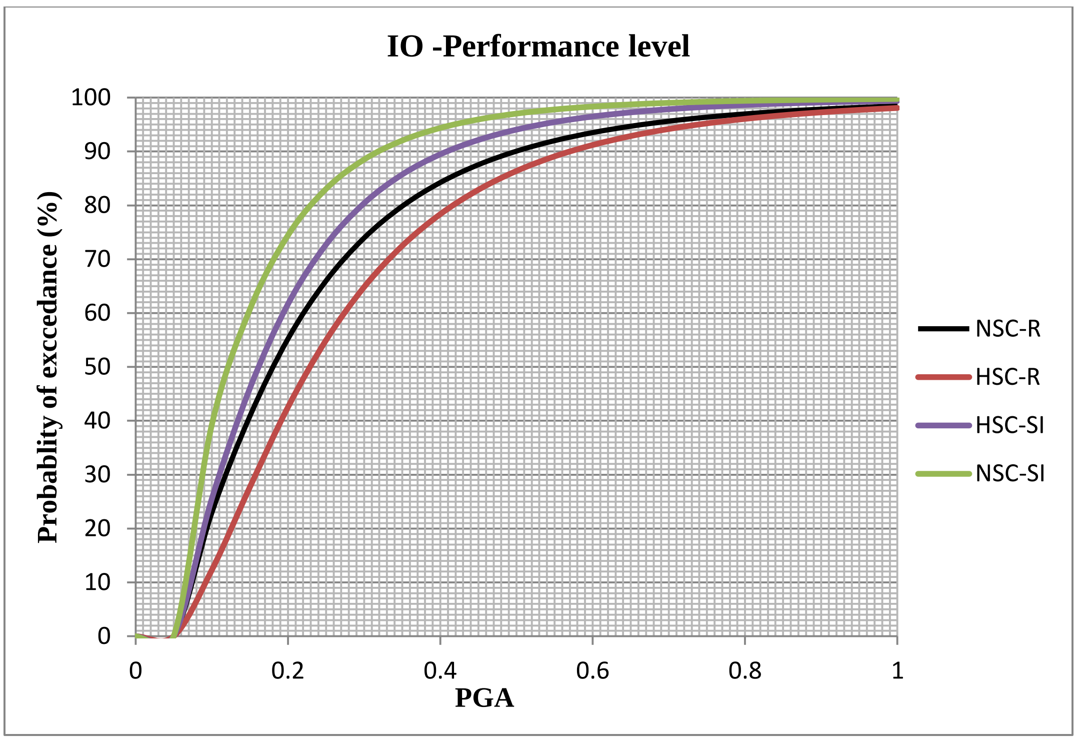


Figure 6.20: Fragility curve of both regular and stiffness irregular for IO Level

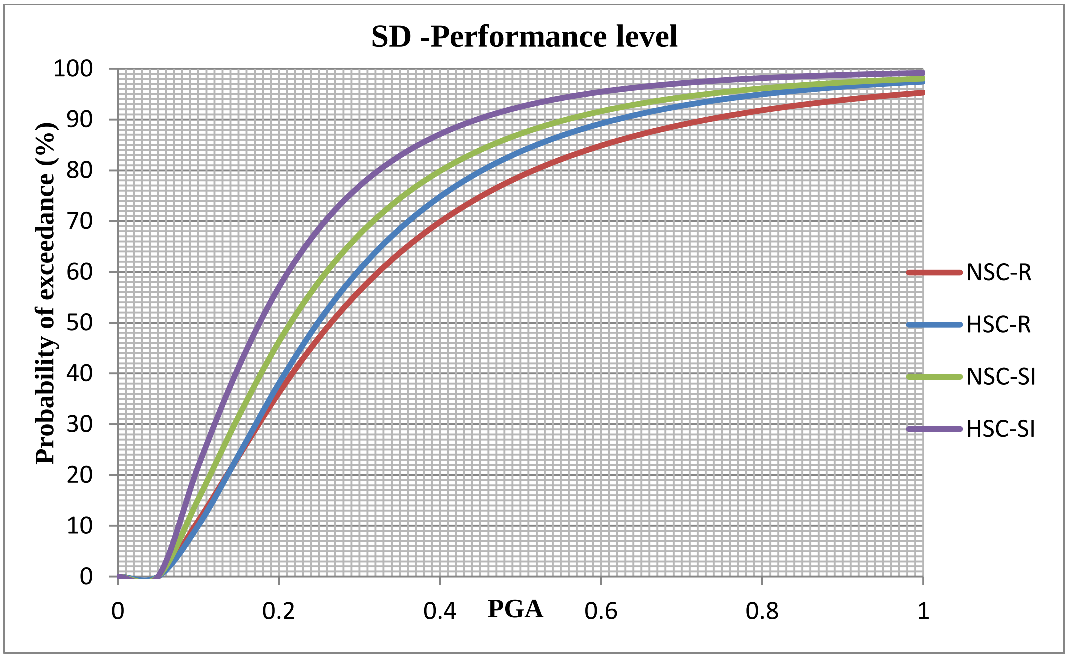


Figure 6.21: Fragility curve of both regular and stiffness irregular for SD Level

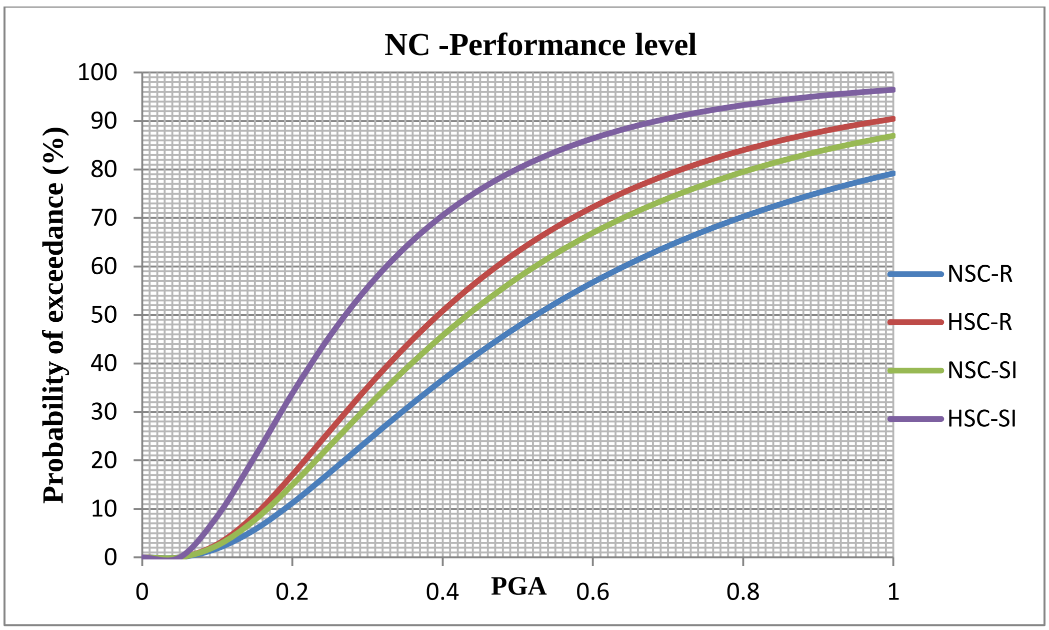


Figure 6.22: Fragility curve of both regular and stiffness irregular for NC Level

The percentage probabilities of exceeding given performance levels are summarized in the table 6.6.

Table 6.6: Percentage probability of Exceedance of limit state capacities of MRF

Performance level (limit state capacities)	**NSC-R**	**HSC-R**	**NSC-SI**	**HSC-SI**
Damage Limitation (IO)	90.08765	84.54315	96.34112159	92.92289
Significant Damage (SD)	77.73104	81.53226	85.04757128	91.12425
Near Collapse (NC)	43.39728	59.29405	53.72145076	77.37085

The fragility analysis indicates that the NSC-R building shows 90.087%, 77.73101% and 43.387% probability of exceeding the IO, SD and NC performance levels respectively.

A similar comparison is shown in the Table 6.6, for HSC-R. It was observed that 84.543%, 81.532% and 59.294% probabilities of exceedance of performance levels IO, SD and NC are occurred respectively.

Regarding the stiffness irregular (soft-story), the result from table 6.6 indicate that 96.341%, 85.047% and 53.721% probability of exceedance of IO SD and NC performance levels are shown respectively for stiffness irregular (soft-story) designed for normal strength concrete (NSC-SI) whereas, for stiffness irregular MRF designed for high strength concrete (HSC-SI) 92.923%, 91.124% and 77.3708% probability exceedance of IO, SD and NC performance levels are occurred.

From the fragility it is observed that HSC-R building shows better performance at the IO performance level but its performance decreased as it jumps to the SD and NC performance levels. Similarly, frames designed for normal strength concrete (NSC) are relatively have better performance at Significant damage (SD) and near collapse (NC) regardless of the of the stiffness irregularity. This clearly shows that increase in concrete strength improves performance of building at lowest performance level but it’s performance decrease because the concrete becomes less ductile and more brittleness as its strength increase at larger displacement.

CHAPTER SEVEN
SUMMARY AND CONCLUSION

7.1 SUMMARY

Nonlinear static analysis and Dynamic inelastic response history analyses were conducted using computer software Seismostruct (2018). From Nonlinear static analysis (pushover analysis) Capacity curves are developed for each case that was previously designed and from incremental dynamic analysis IDA curves are developed to check the collapse capacity of the building under seismic excitation.

The results are evaluated for different performance levels as indicated by corresponding damage indices, where story drift was adopted as the damage indicator. Commonly accepted performance levels, defined as immediate occupancy (IO), life safety (SD) and collapse prevention (DL) are adopted. Vulnerability assessment of reinforced concrete frame buildings with regular and soft story (stiffness irregular) were presented using fragility curve as probability of exceedance of predetermined performance levels for given intensity levels. Accordingly, fragility curves were developed and presented for all MRF cases, and compared to each other to learn how the stiffness irregularity and compressive concrete strength affect the seismic performance of the RC moment resisting frame (MRF). By analyzing the results from pushover analysis, Incremental dynamic analysis and fragility curve the following conclusion were drawn regarding the seismic performance of the high strength concrete soft story MRF building

7.2 CONCLUSION

The capacity curves, limit state capacities, probabilistic seismic demand models and fragility curves are developed for all considered buildings. Accordingly the following major conclusions are drawn from the investigation of the seismic performance assessment of stiffness irregular (soft-story) reinforced concrete buildings designed for NSC and HSC evaluated on the basis of capacity curves and the fragility curves.

- The shear capacity for regular moment resisting frame designed for high strength (HSC-R) is increased compared to that of regular normal strength concrete frames (NSC-R). This indicates that the increase in concrete strength improves shear capacity of regular moment resisting frames.
- The roof displacement of the regular moment resisting frame designed for the high strength (HSC-R) at collapse shear capacity is reduced when compared with that of regular MRF designed for normal strength (NSC-R). The decrease in roof displacement shows that when using HSC, the frames **fail earlier**, though the shear capacity of the frame is improved. This is because the concrete is less ductile and tends to fail without utilizing its full capacity under seismic loading as its compressive strength increases.
- Fragility analysis indicate that HSC-R moment resisting frame shows better performance at Immediate occupancy (IO) performance level and it performance decrease at Significant damage (SD) and Near collapse performance level.

- NSC-R moment resisting frame have relatively better performance at significant damage (SD) and near collapse performance (NC) level when compared to the HSC-R moment resisting frame.

For soft-story building the following conclusion is drawn for both high strength and normal strength concrete.

- The base shear capacity of the stiffness irregular MRF designed for high strength concrete (HSC-SI) is increased slightly compared to that of stiffness irregular frames designed for normal strength concrete (NSC-SI) but decreased compared to the Regular frame designed for the normal strength concrete (NSC-R).
- The roof displacement of the soft story (stiffness irregular) MRF designed for high strength concrete (HSC-SI) at the collapse shear capacity is reduced compared to the both NSC-R and NSC-SI. This agrees with the assumption that increases in the concrete strength results rigid frames.

Generally, the results from both pushover analyses and Incremental Dynamic analysis shows that the performance of the Moment resisting frames building (MRF) have similar trend though they have varying values of different damage states

From the fragility analysis frames designed for high strength (HSC) perform generally well at immediate occupancy (IO) performance level whereas, frames designed for normal strength concrete (NSC) are relatively better at Significant damage (SD) and near collapse (NC) regardless of the of the stiffness irregularity. Hence, results indicate that the HSC do not achieve the expected seismic performance because high strength concrete decreases their ductility under earthquake excitation and tends to be more brittle under large displacement.

Regarding the stiffness irregular frame one can observe that the use of the high strength concrete for soft-story (HSC-SI) at the first story level in a building results the collapse shear capacity performance to increase slightly compared to similar frames designed for normal strength concrete (NSC-SI). Therefore, It can be understood that the change in the collapse shear capacity of the Stiffness irregular MRF building designed for the high strength concrete is not significant when compared to the to the stiffness irregularity effect on the same moment resisting frame (i.e. stiffness irregular (soft-story) building.

7.3 RECOMMENDATIONS

The following recommendations are made for future research.

- High-rise moment resisting frame having more than 12 story height were not considered in the study. Further seismic performance evaluation have to be undertaken for high rise moment resisting frame with more than 12-storeis
- In this study only soft story irregularity at first story level is considered for practical reason , for the future study other types of vertical irregularities can be considered as part of research work and additional assessment can be developed for the building .
- The building with high strength steel reinforcement can be considered in addition to the high strength concrete for further seismic performance assessment of the building.

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APPENDIX A

STIFFNESS IRREGULARITY CHECK FOR MOMENT RESISTING FRAME

According to ESEN-1998 (2015) if the stiffness of a story is less than 70% of the stiffness of the storey above it or less than 80% of the average stiffness of three stories above it is called soft storey. Using the value of the stiffness estimated from the ETABS the soft-story stiffness irregularity can be checked as follows.

$$k_i < 0.7\, k_{i+1} \rightarrow \frac{ki}{ki+1} < 0.7 \quad \text{or}$$

$$k_i < 0.8 \left(\frac{ki+1 + ki+2 + ki+3}{3}\right) \rightarrow \frac{3ki}{ki+1 + ki+2 + ki+3} < 0.8$$

Table A.1 stiffness irregularity check for NSC-R MRF

	Stiff X	Stiff Y	Stiffness Irregular check-along x- axis $\left(\frac{ki}{ki+1}\right)*100$	Stiffness Irregular check-along y- axis $\left(\frac{ki}{ki+1}\right)*100$
Story	kN/m	kN/m		
Story12	248200.409	224128.738	0	0
Story11	264200.756	244172.762	106.44	108.94
Story10	278200.967	264148.397	105.29	108.18
Story9	304377.179	298548.425	109.40	113.02
Story8	309475.596	307832.888	101.67	103.11
Story7	323388.737	324248.959	104.49	105.33
Story6	426972.764	430660.033	132.03	132.81
Story5	445515.52	450731.01	104.34	104.66
Story4	459430.29	466114.388	103.12	103.41
Story3	488158.191	496147.857	106.25	106.44
Story2	581050.351	590556.047	119.02	119.02
Story1	1181066.529	1197145.301	203.26	202.71

Table A.2 Stiffness irregularity check for HSC-R MRF

Story	Stiff X kN/m	Stiff Y kN/m	Stiffness Irregular check-along x-axis $\left(\frac{ki}{ki+1}\right)*100$	Stiffness Irregular check-along y- axis $\left(\frac{ki}{ki+1}\right)*100$
Story12	221451	224135.45	0	0
Story11	249231	240387.4	112.54	107.25
Story10	278200.967	264148.397	111.62	109.88
Story9	304377.179	298548.425	109.40	113.05
Story8	309475.596	307832.888	101.67	103.06
Story7	323388.737	324248.959	104.49	105.33
Story6	426972.764	430660.033	132.03	132.81
Story5	445515.52	450731.01	104.34	104.66
Story4	459430.29	466114.388	103.12	103.41
Story3	488158.191	496147.857	106.25	106.44
Story2	581050.351	590556.047	119.02	119.02
Story1	1181066.529	1197145.301	203.26	202.71

Table A.3 Stiffness irregularity check for NSC-SI MRF

Story	Stiff X kN/m	Stiff Y kN/m	Stiffness Irregular check-along x- axis $\left(\frac{ki}{ki+1}\right)$	Stiffness Irregular check-along y- axis $\left(\frac{ki}{ki+1}\right)$
Story12	214350.839	225128.738	0	0
Story11	234200.834	242172.762	109.26	107.57
Story10	250246.787	204348.511	106.85	84.38
Story9	313100.354	290896.82	125.11	142.35
Story8	316959.143	299531.643	101.23	102.96
Story7	398514.319	378842.964	125.73	126.47
Story6	421193.933	411376.52	105.69	108.58
Story5	433848.345	434343.536	103.04	105.58
Story4	445089.218	450056.962	102.59	103.61
Story3	470251.481	477300.597	105.65	106.53
Story2	589817.235	587619.614	125.42	123.11
Story1	404272.051	405156.855	68.54	68.95

Table A.4 Stiffness irregularity check for HSC-SI MRF

Story	Stiff X kN/m	Stiff Y kN/m	Stiffness Irregular check-along x-axis $\left(\frac{ki}{ki+1}\right)$*100	Stiffness Irregular check-along y- axis $\left(\frac{ki}{ki+1}\right)$*100
Story12	321555.367	2944503.629	0	0
Story11	341096.756	314612.671	106.07	106.84
Story10	360736.755	337368.003	105.75	107.23
Story9	398327.065	387011.416	110.42	114.71
Story8	405945.359	400978.617	101.91	103.60
Story7	420473.619	419253.936	103.57	104.55
Story6	545430.717	546942.228	129.71	130.45
Story5	565782.097	569547.876	103.73	104.13
Story4	573138.62	579179.19	101.30	101.69
Story3	578657.409	586584.822	100.96	101.27
Story2	598383.036	599240.648	103.40	102.15
Story1	416868.002	411246.52	69.66	68.62

APPENDIX B

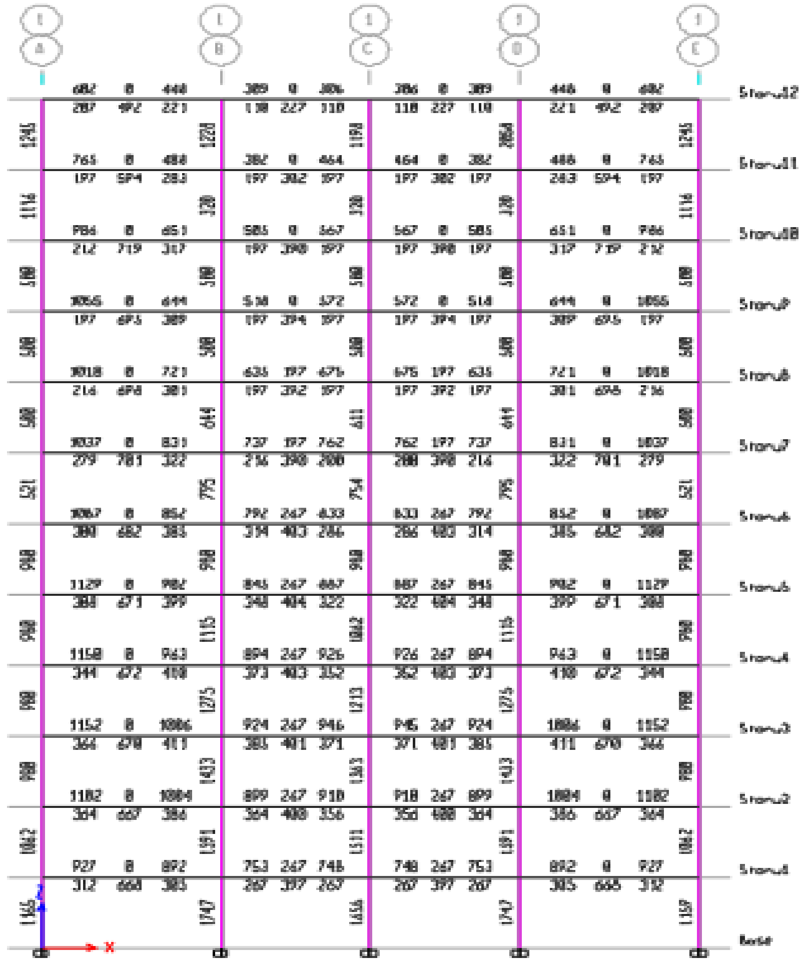

Figure B.1 NSC- R frame desiign values from ETABS v-18.1

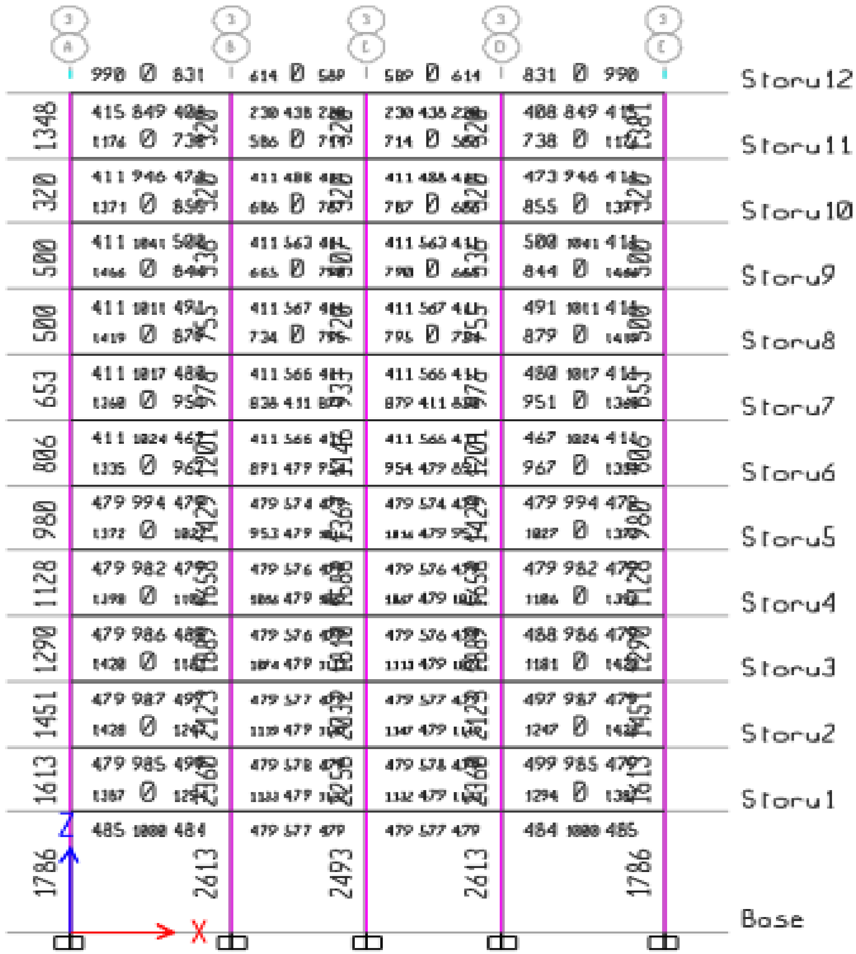

Figure B.2 HSC-Rframe desiign values from ETABS v-18.1

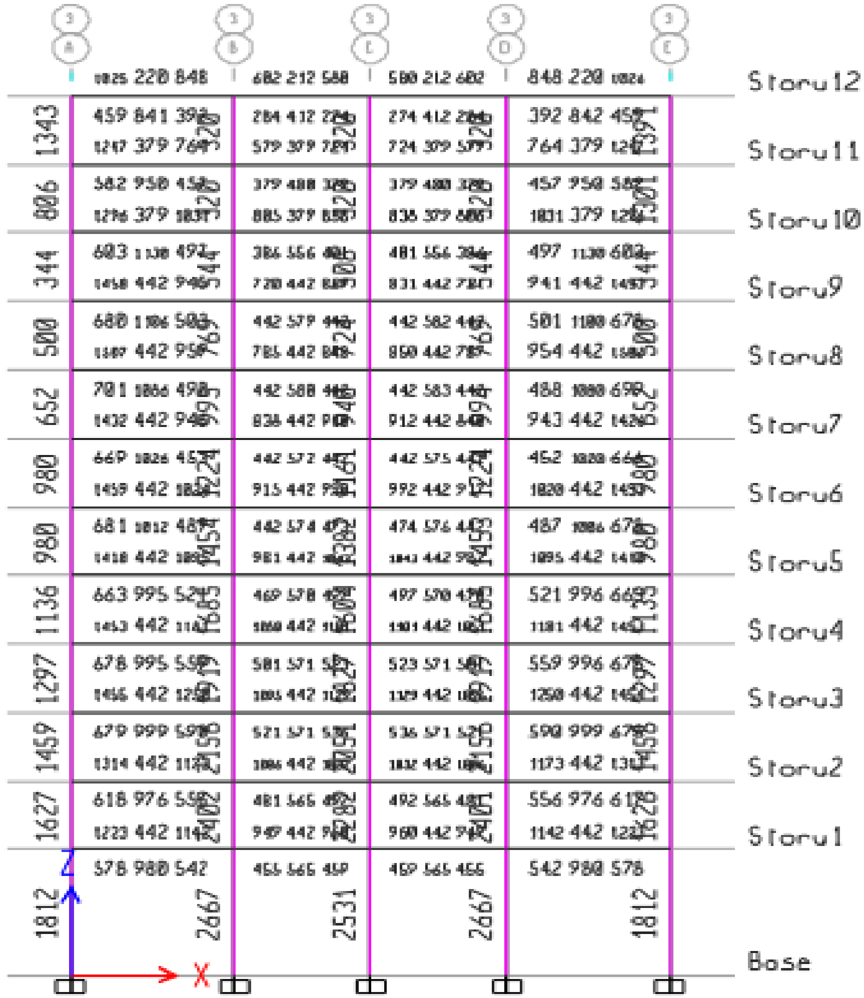

Figure B.3 NSC-SI frame desiign values from ETABS v-18.1

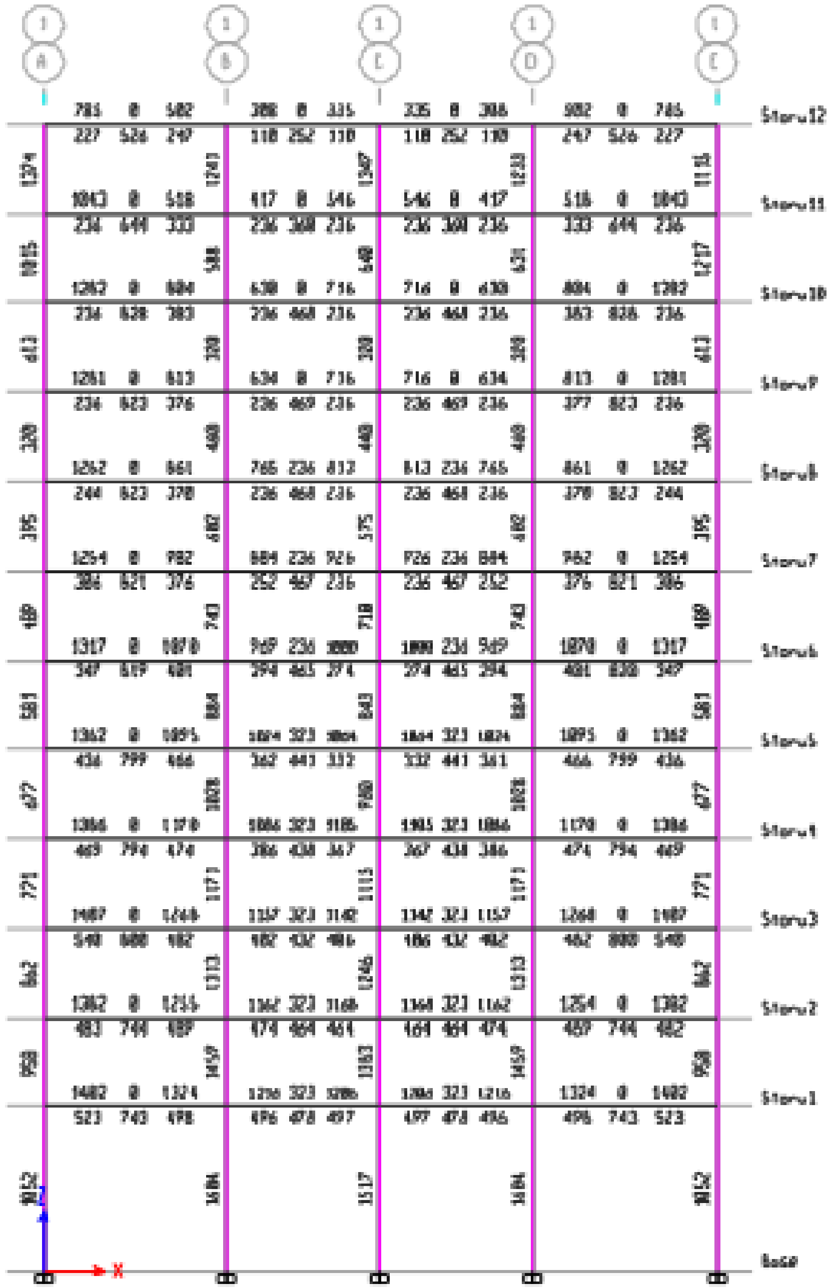

Figure B.4 HSC-SI frame design values ALONG SECTION 1-1

APPENDIX C

Three dimension models created using the Seismostruct (2018)

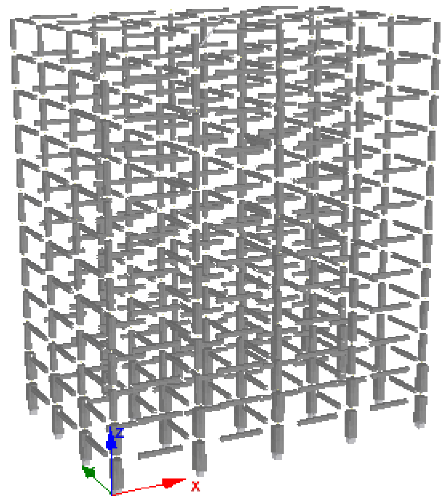

Figure C.1: 3D Seismostruct model for NSC-R MRF

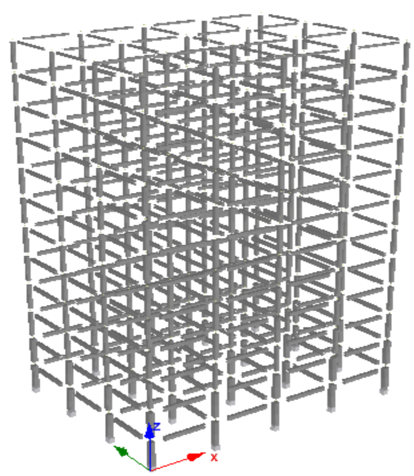

Figure C.2: 3D Seismostruct model for HSC-R MRF

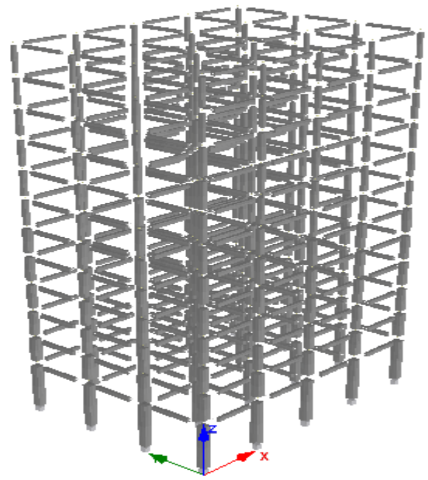

Figure C.3: 3D Seismostruct model for NSC-SI MRF

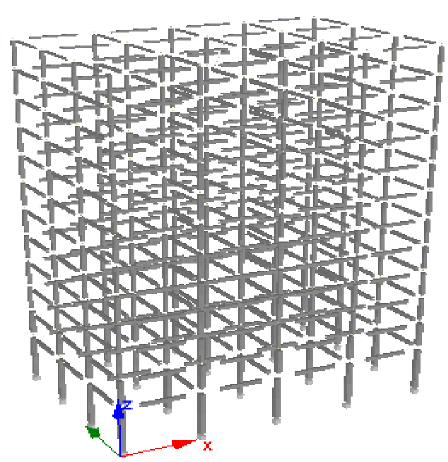

Figure C.4: 3D Seismostruct model for HSC-SI MRF

APPENDEX D

CALCULATION OF THE STANDARD DEVIATION AND MEAN OF EDP FOR MRF

Using the equation given in 3.6 the dispersion in engineering demand parameter $\beta_{EDP/PGA}$ and Mean are calculated as in the table E.1, E.2, E.3 and E.4

Table E.1: Calculation of the Standard deviation and mean of EDP for NSC-R

PGA	Top Drift	EDP =a(PGA)^b	ln(di)-ln(EDP)
0.1	15.40478	16.53626477	1.269202547
0.15	26.32888	23.76090637	1.805191131
0.2	34.11542	30.72969384	2.064274177
0.25	41.66808	37.51420826	2.264260064
0.3	48.57603	44.15539956	2.417654895
0.35	61.43612	50.67972492	2.652522632
0.4	70.27648	57.1056461	2.786961873
0.45	83.05347	63.4467535	2.954009313
0.5	94.84033	69.71345408	3.086719437
0.55	104.8375	75.91396263	3.186936419
0.6	111.6617	82.05492166	3.249998729
0.65	118.0598	88.1418087	3.305715802
0.7	123.6886	94.17921476	3.35229157
0.75	128.8189	100.1710409	3.39293216
0.8	131.9373	106.1206413	3.416851354
0.85	134.3047	112.0309289	3.434635646
0.9	135.3735	117.9044565	3.442562321
0.95	137.6438	123.7434786	3.45919403
1	140.4051	129.55	3.479056654

Mean of ln(EDP)	**4.177884418**
Standard deviation	**0.574015108**

Table D.2: Calculation of the Standard deviation and mean of EDP for HSC-R

PGA	Top Drift	EDP =a(PGA)^b	ln(di)-ln(EDP)
0.1	11.30293	13.37741	0.892934158
0.15	16.98234	21.70298	1.300046155
0.2	22.95948	30.59295	1.601603097
0.25	49.62522	39.92765	2.372371346
0.3	61.43132	49.63278	2.585791977
0.35	67.83461	59.6572	2.684944711
0.4	82.31842	69.96333	2.878467072
0.45	106.6707	80.52225	3.137618693
0.5	114.38701	91.31096	3.207459698
0.55	115.95384	102.3107	3.221064355
0.6	123.59487	113.5057	3.284881214
0.65	132.83787	124.8829	3.357001536
0.7	140.73633	136.4307	3.414760314
0.75	146.12445	148.1393	3.45233083
0.8	148.91045	159.9999	3.471217293
0.85	155.32857	172.0048	3.513414854
0.9	156.99676	184.1472	3.524097342
0.95	158.69776	196.4208	3.534873687
1	159.24515	208.82	3.538317013

Mean of ln(EDP)	**4.425488093**
Standard deviation	**0.766252382**

Table D.3: Calculation of the Standard deviation and mean of EDP for NSC-SI

PGA	Top Drift	EDP =a(PGA)^b	ln(di)-ln(EDP)
0.1	17.92783	19.37125	1.399254
0.15	25.92791	27.41669	1.76822
0.2	35.86264	35.07922	2.092596
0.25	43.35534	42.46907	2.28233
0.3	52.72131	49.64864	2.47792
0.35	61.28715	56.65793	2.62847
0.4	69.47446	63.52466	2.753859
0.45	75.25196	70.26916	2.833742
0.5	78.02411	76.90688	2.869918
0.55	82.09646	83.44999	2.920795
0.6	87.05603	89.90829	2.979452
0.65	92.79989	96.28984	3.043345
0.7	98.53968	102.6013	3.103359
0.75	103.67222	108.8485	3.154134
0.8	111.43378	115.0363	3.22633
0.85	118.77101	121.1688	3.290097
0.9	126.27946	127.2498	3.351397
0.95	133.8982	133.2826	3.40998
1	140.51436	139.27	3.458209

Mean of ln(EDP)	**4.278861**
Standard deviation	**0.550066**

Table D.4: Calculation of the Standard deviation and mean of EDP for HSC-SI

PGA	Top Drift	EDP =a(PGA)^b	ln(di)-ln(EDP)
0.1	14.58471	19.14849	1.145349
0.15	29.86732	28.60419	1.86214
0.2	42.22463	38.02717	2.208379
0.25	48.49101	47.42589	2.346754
0.3	61.30022	56.80533	2.581159
0.35	73.14883	66.16877	2.757872
0.4	78.58481	75.51852	2.829554
0.45	86.43741	84.85633	2.924796
0.5	101.4489	94.18354	3.08493
0.55	118.7204	103.5012	3.242147
0.6	122.4135	112.8103	3.27278
0.65	121.3023	122.1114	3.263661
0.7	123.7437	131.4052	3.283588
0.75	128.8817	140.6922	3.32427
0.8	140.6867	149.973	3.411911
0.85	152.4558	159.2478	3.49225
0.9	161.652	168.517	3.550821
0.95	168.7279	177.781	3.593663
1	174.3659	187.04	3.626531
		Mean of ln(EDP)	**4.471609**
		Standard deviation	**0.635526**

APPENDEX E

FRAGILITY CURVE

1. **Fragility for curve for NSC-R (MRF)**

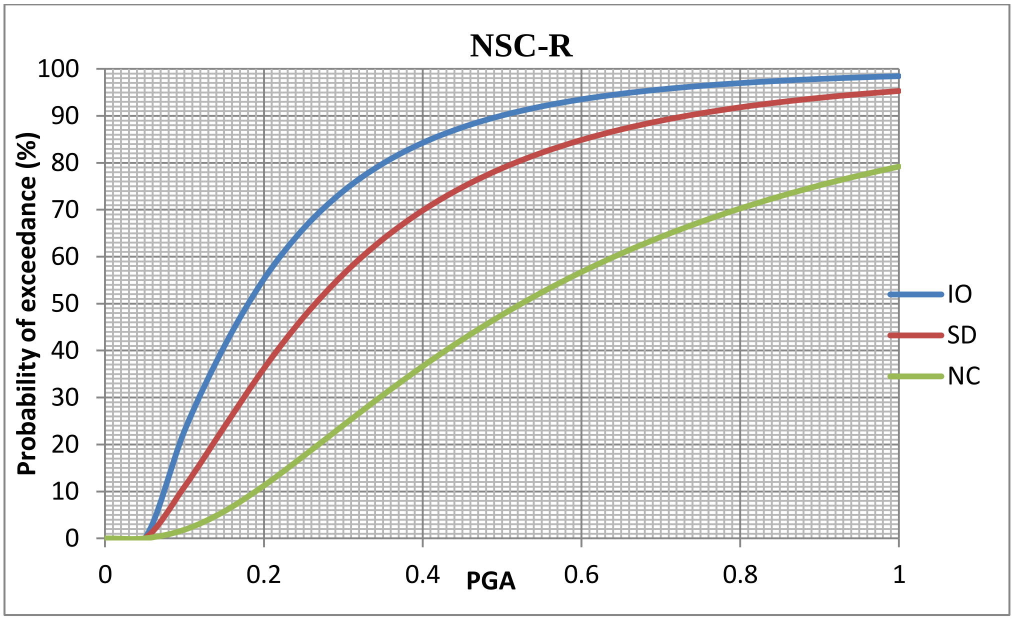

Figure E.1: Fragility curve of NSC-R MRF

2. **Fragility curve for HSC-R MRF**

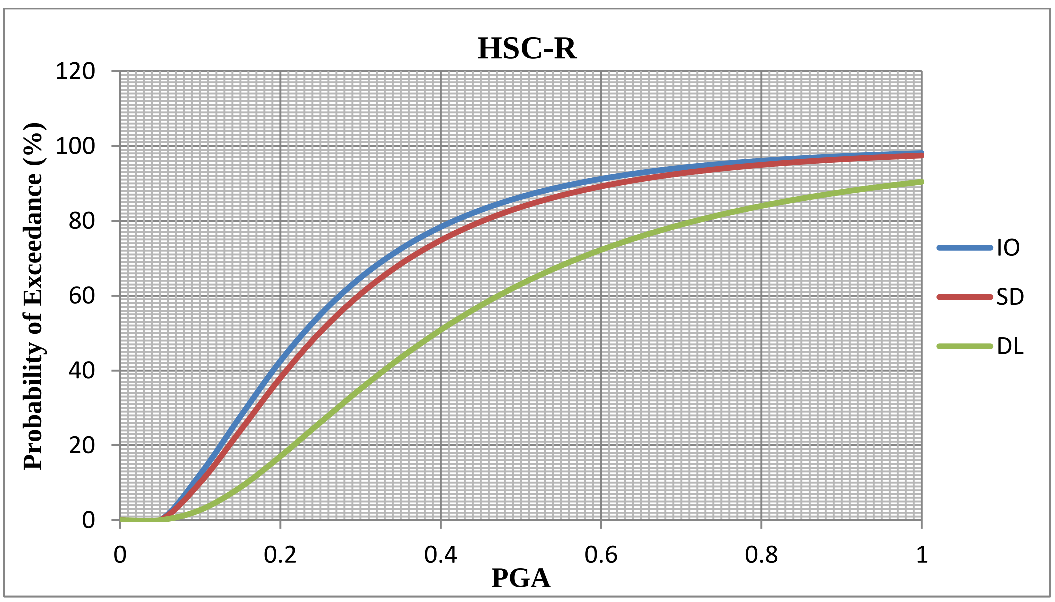

Figure F.2: Fragility curve of HSC-R MRF

3. **Fragility Curve for NSC-SI MRF**

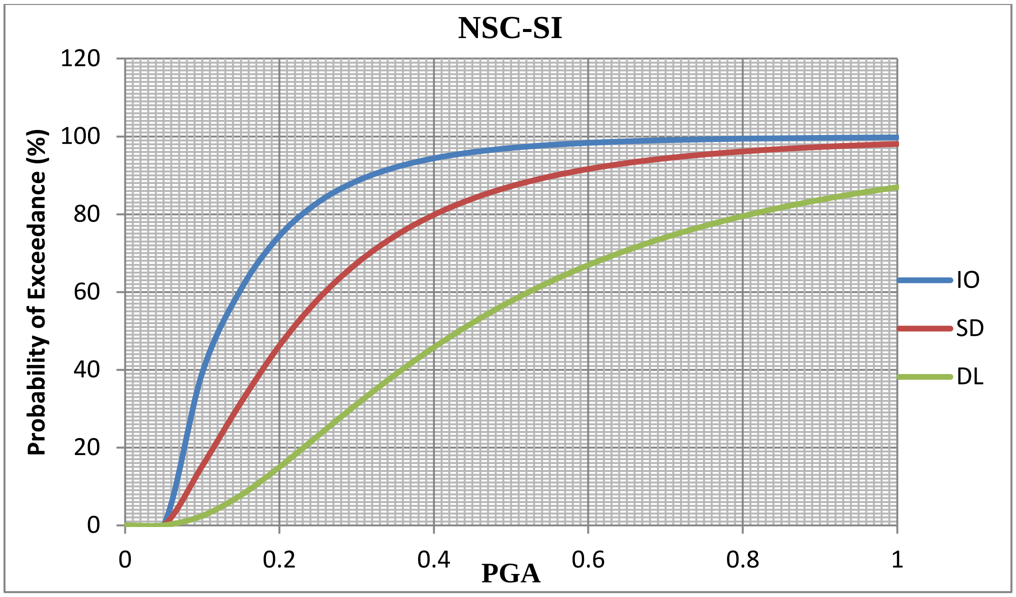

Figure E.3: Fragility curve of NSC-SI MRF

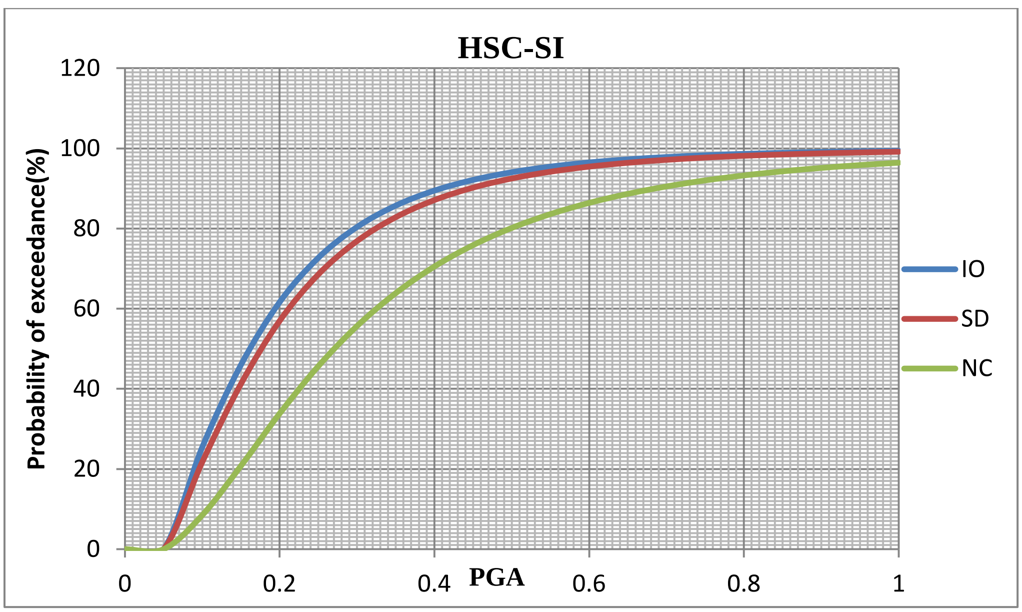

Figure E.4: Fragility curve of HSC-SI MRF