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Seismic Vulnerability Assessment and Retrofitting Techniques for MRT Compliant RCC Buildings

By:

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A THESIS

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The undersigned certify that they have read, reviewed, and recommended to the Institute of Engineering for acceptance, a thesis entitled "**Seismic Vulnerability Assessment and Retrofitting Techniques for MRT Compliant RCC Buildings**" in partial fulfillment of the requirements for the degree of M.Sc. in Engineering in Master in Disaster Risk Management.

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ABSTRACT

From 1970 to 2015, there was a rapid growth in Reinforced Concrete Cement (RCC) building construction in Kathmandu valley. Ready to use dimensions and details provided in Mandatory Rule of Thumb (MRT) issued in 1994 was primarily used for up to three storey ordinary residential buildings. This MRT was adopted to achieve the minimum seismic safety requirements specified by NBC 105:1994. The minimum dimensions and detailing were updated in the draft code of MRT in 2012. Later in 2020, NBC 105 code for seismic design of buildings in Nepal was revised.

This study aims to quantify the seismic vulnerability of MRT compliant buildings in Lalitpur Municipality City (LMC) and analyze their behavior in future earthquake (MCE). Sixty-eight samples of building drawings of RCC structures were collected from LMC and was categorized into various typologies based on dimension of structural members, number of bays, number of storeys, height of the building, grade of concrete, compliance to MRT codes. Structural analysis of final ten typologies was done and retrofitting strategies were applied to the structures as per guidelines of IITK GDSMA and Seismic Retrofitting Guidelines of Buildings in Nepal (RCC) -DUDBC and the ones satisfying performance level of Life Safety was concluded acceptable.

All typologies except for typology I required intervention for retrofitting. The size of beams, columns, slabs were deemed sufficient while the longitudinal detailing for columns were deficit in majority of the study. The reinforcement details for beams and slab were sufficient for all typologies. The retrofitting method used were RC jacketing, steel jacketing and addition of shear wall. It was found that addition of shear wall to the buildings for three storey effectively improved the performance level while for structure for up to two storey, use of concrete and steel jacketing were sufficient. Thus, seismic vulnerability of MRT compliant buildings were known and appropriate retrofitting techniques were suggested based on defined typologies.

Moreover, the retrofitting guideline summary obtained for each typology can be used directly by house owners and concerned authorities for retrofitting purpose. The findings of this research can be useful tool for DUDBC for developing general retrofitting guidelines for all types of MRT buildings. It can be also used under disaster risk mitigation policies of Nepal.

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I acknowledge the support and encouragement from my friends and family members. I am grateful for their unconditional help.

I have taken serious precautions to avoid errors in course of writing this report. Possibility of errors cannot be ruled out. Hence, comments and suggestions are gratefully accepted.

Nisha Sthapit

(PUL076MSDRM013)

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ABBREVIATIONS

ATC	:	Applied Technology Council
CBS	:	Central Bureau of Statistics
DBE	:	Design Based Earthquake
DRR	:	Disaster Risk Reduction
DUDBC	:	Department of Urban Development and Building Construction
EQ	:	Earthquake
EQx	:	Earthquake in X direction
EQy	:	Earthquake in Y direction
ESM	:	Equivalent Static Method
ETABS	:	Extended Three-Dimensional Analysis of Building System.
FEMA	:	Federal Emergency Management Agency
FRP	:	Fiber Reinforced Polymer
IITK-GSDMA	:	Indian Institute of Technology Kanpur- Gujarat State Disaster Management Authority
IS	:	Indian Standard
JICA	:	Japan International Cooperation Agency
LMC	:	Lalitpur Metropolitan City
MCE	:	Maximum Considered Earthquake
MRT	:	Mandatory Rule of Thumb
MSRM	:	Modal Response Spectrum Method
NBC	:	National Building Code
NDRRMA		National Disaster Risk Reduction and Management Authority
NEHRP	:	National Earthquake Hazard Risk Reduction Programme
PGA	:	Peak Ground Acceleration
RC	:	Reinforced Concrete
RCC	:	Reinforced Concrete Cement
SLS	:	Serviceability Limit State
ULS		Ultimate Limit State

CHAPTER 1: INTRODUCTION

1.1 Background

Nepal is in high risk of earthquake as it is formed by collision of Eurasian plate with the Indian plate. According to seismologists, a major earthquake with a moment magnitude (Mw) 8 or greater is likely to strike in Nepal every 70 years (Bilham 2018). The earthquake of 2015 in Nepal Earthquake killed more than 9000 people and nearly 22,000 were injured. More than 750000 structures was damaged.

Kathmandu is one of the top ten urbanizing cities in the world (JICA 2002). There has been a rapid growth in Reinforced Concrete Cement (RCC) building construction for past 40 years. Almost half of these buildings are non-engineered. According to reports, structures designed in accordance with code requirements account for only 8% of the overall RCC building stock. (Dixit, 2004; Shrestha and Dixit, 2008).

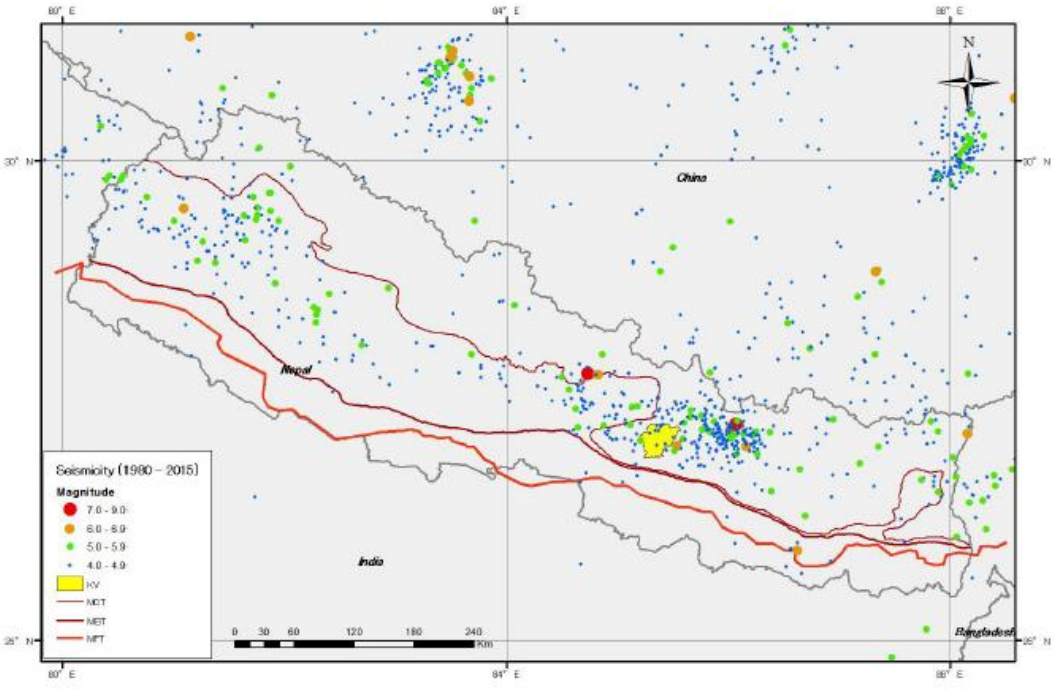


Figure 1: Distribution of past earthquakes in Nepal (Source: JICA 2018)

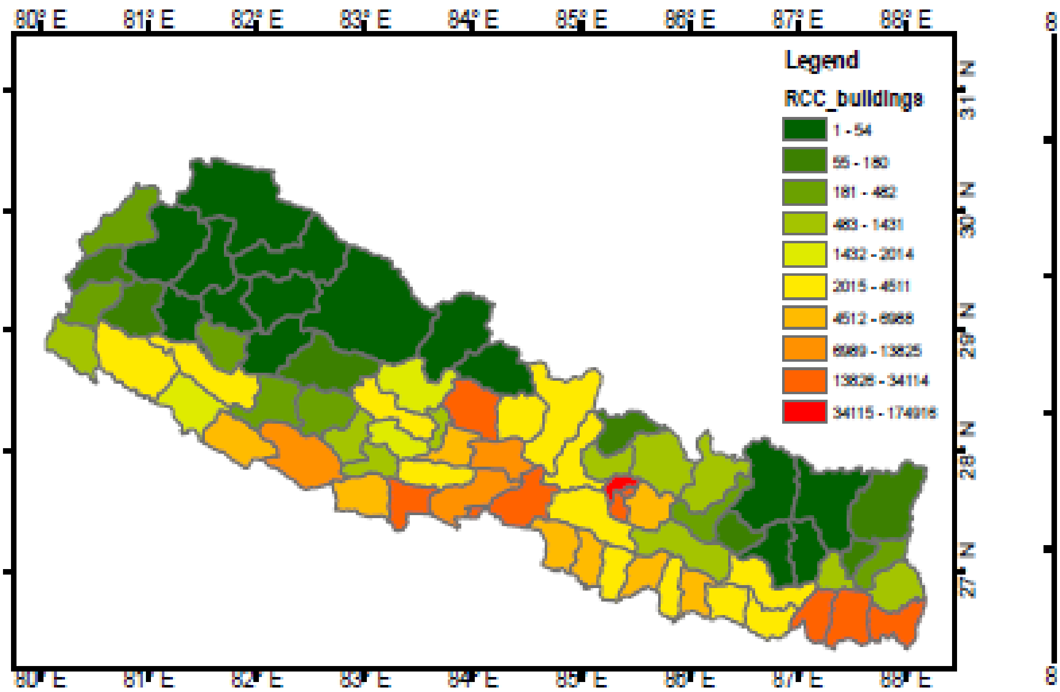


Figure 2: District wise distribution of RCC building in Nepal (Source: JICA 2016)

The seismic design code of Nepal (NBC 105) was developed in 1993 by the DUDBC (Department of Urban Development and Building Construction). It did not consider the analytical techniques and sophisticated design philosophies that can be found in the building codes of developed countries but provides basic seismic design provisions. NBC 205:1994 and NBC 201:1994, referred as Mandatory Rule of Thumb (MRT) was introduced to provide pre-defined dimensions and detailing to midrise RCC buildings. Its objective was to encourage good construction practices that is easy to implement and abide by the requirements of seismic code of buildings NBC 105:1994.

It was reported that most of the buildings built as per MRT were damaged and/or destroyed in 2015 Gorkha earthquake, regardless of its moderate shaking intensity such as cities of Kathmandu Valley. A study by Brzev et al. (2017) suggested that inadequate design and construction of RCC buildings were the main reason for damage of MRT compliant buildings.

Lalitpur Sub-Metropolitan City (LSMC) implemented the provisions for building constructions as per MRT guidelines in 2002. It was the first Municipality to incorporate the application of NBC code. In its initial years, the municipality provided flexibility to the minimum criterias and design drawings make the designers, house owners and mid-level technicians familiar with NBC. After 2009 A.D, it became mandatory to submit detailed drawings for the MRT compliant buildings as well.

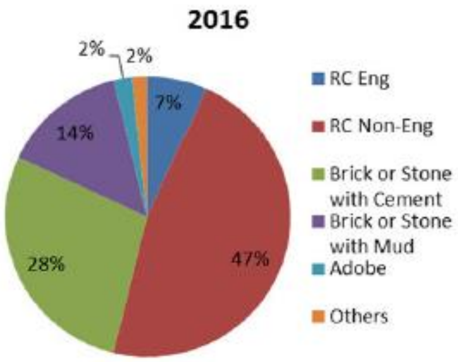


Figure 3: Building structure component ratio in Kathmandu Valley (Source: JICA 2018)

The use of MRT guidelines in building construction meant that seismic analysis and design were not required for construction of RCC buildings for up to three storey and such buildings should have area less than 100 m^2 (1000 sq. ft.).

After the Gorkha earthquake, NBC 105 was revised in 2020. As per study by Shrestha et.al, it was found that the base shear of the building was 60 percent more in NBC 105:2020 compared to NBC 105:1994. The period of vibration is higher due to use of stiffness modifiers in cracked sections in NBC 105:2020. It is evident that as per the new code, the existing buildings do not meet the criteria for good seismic performance but the data has not been quantified and verified. Thus, for future scenario earthquakes, the government must evaluate the seismic vulnerability of such buildings and suitable retrofitting technique must be adopted.

1.2 Rationale of the Study

1.2.1 Need and Importance of Research

The column size suggested by MRT (NBC 205:1994) was excessively small with insufficient detailing of reinforcement due to which it was not able to effectively resist seismic forces in Nepal earthquake of 2015. (Brzev et al., 2017).

MRT issued in 1994 was based on NBC 105:1994 in which the seismic parameter taken was different in terms of material properties, seismic zoning factor, soil type, load combination among others than that of NBC 105:2020.

Vulnerability assessment of buildings should be done after the change in codal provisions (Seismic Retrofitting Guidelines of Buildings in Nepal, 2016). Thus, the MRT buildings must be evaluated for its performance in future earthquakes and if necessary these structures should be retrofitted.

1.2.2 Problem Statement

MRT buildings with typical column sizes of 9"X9" did not enable the creation of a favorable ductile mechanism. The longitudinal rebars in the majority of the collapsed structures in 2015 Nepal earthquake were typically four, however the majority of the building columns were discovered to be built with six rebar in the vertical direction. It was found that the stirrups were 6 mm diameter bars that typically spaced at regular intervals of 0.15 m or more (Gautam et.al 2016). Masonry walls heavily influenced the response of such buildings and it performed as wall structures (Karmacharya 2018). A previous study by NSET on Nepal's seismic vulnerability found that 60% or more of these buildings are highly vulnerable to any major incipient earthquake (NSET 1999).

Most of the buildings in residential areas of Lalitpur Metropolitan City (LMC) are constructed as per MRT. Typically, columns are of size 230mmX230mm, 230mmX300mm and 300mmX300mm.

It was only in 2010, DUDBC updated the minimum size of column to 300mmX300mm in NBC 205:2012. Following the 2015 earthquake, a post-earthquake reconnaissance survey showed no ductile design and build practices of MRT RC buildings in disaster affected areas, which is a serious problem in Kathmandu Valley (Brzev et al. 2017b).

1.3 Objective of Research

The **main objective** of this thesis is to evaluate the seismic vulnerability of RCC buildings constructed as per NBC 201:1994, NBC 205:1994 and NBC 205:2012 on the basis of NBC 105:2020 and to propose appropriate retrofitting strategies based on building typology.

1.4 Research Scope

- Defining building typology based on structural parameters.
- Developing general retrofitting techniques and guidelines based on building typology.

1.5 Research Questions

- What parameters deems insufficient for buildings built as per MRT?
- Will retrofitting be suitable for all types of buildings?

1.6 Research Limitations

This research studies only the RCC buildings whose drawings and construction has been approved by the Lalitpur Municipality. Other buildings non-compliant with MRT codes are not be studied. The buildings that have 230mmX230mm, 230mmX300mm and 300mmX300mm column size are studied and other dimension of columns are not included. Only three types of retrofitting techniques i.e. RC jacketing, steel jacketing and addition of shear wall are studied in this research.

1.7 Research Purpose

More than 33934 RCC houses in LMC do not meet the specification of new code. Performance of existing MRT buildings has not been studied extensively. Similarly, need for retrofitting is discussed but specific retrofitting strategy for specific type/typology of building is yet to be determined. Thus, this research will help fulfill the gap in the area of seismic vulnerability assessment and retrofitting for representative MRT compliant buildings in Nepal.

CHAPTER 2: LITERATURE REVIEW

2.1 Reinforced Cement Concrete (RCC)

The RCC building's frame is load bearing, but the masonry walls play a crucial role in the building's structural stability. Infill masonry can have both positive and negative effect in the performance of the building. A study done by HRRP in 2018 stated that the presence of masonry infill walls weakens the frame connection, necessitating a much heavy weight RCC frame to accommodate. RCC buildings not only perform better when properly engineered, but they also necessitate a good knowledge of design than most of the masonry buildings. Kathmandu has a high proportion of non-engineered RCC buildings.

2.2 Performance of RCC building with masonry infill

The formation of hinges in RC buildings' beams and columns in response to seismic ground motion suggests that the structure is susceptible to shear or flexural failure. The failure of RC buildings with infill walls can result in diagonal shear failure of the walls and their adjoining columns. The likelihood of shear failure increases with the strength of the masonry wall (Martín Tempestti and Stavridis 2017).

2.3 Current construction practice

As per study conducted by Karmacharya et al (2018), the thickness of the infill walls is 230 mm or 115 mm and the column size is predominantly 230 mm×230 mm. The prevalent practice in most urban areas of Nepal for the construction of residential and commercial complexes generally falls under this category. These buildings are neither designed nor supervised by engineers.

2.4 Mandatory Rule of Thumb (MRT)

The primary goal of MRT is to provide predefined dimensions and reinforcement details for structural components and as well as non-structural attributes. The guideline facilitates the provision of construction for up to three-storey ordinary RCC residential buildings commonly constructed by building owner in Nepal. Three MRT for RCC buildings are issued till date which are NBC 201:1994 (MRT for RCC buildings with masonry infill wall), NBC 205:1994 and NBC 205:2012 (MRT for RCC buildings with masonry infill wall)

The column size in NBC 201:1994 and NBC 205:1994 was 230mmX230mm while it was 300mmX300mm in NBC 205:2012.

Infill walls are regarded seismically resistant only if the opening is less than 10% of the wall's total area. The openings should be located outside of the restricted zone and in the middle two-thirds of the panel.

2.5 Comparison between NBC 105 (1994 & 2020)

The following are the major changes in NBC 105:2020 compared to NBC 105:1994:

1) Drift check in both for Ultimate Limit State(ULS) and Serviceability Limit State(SLS) is done.
2) Spectral shape factor curve instead of response spectrum curve for Equivalent Static Method (ESM) and Modal Response Spectrum Method (MSRM) are used as per soil type.
3) Seismic zone was revised based on probabilistic seismic hazard analysis. Zone factors has been changed and it represents realistic value of Peak Ground Acceleration (PGA) in considered region.
4) Importance classes of building types and its factors were revised.
5) Empirical formula for fundamental translation period (time period) has been updated.
6) Horizontal base shear coefficient to be determined for ULS & SLS separately. These coefficients are in terms of ductility factor and over strength factor, replacing the performance factor K in 1994.
7) Horizontal design spectrum for modal response spectrum method is different for ULS & SLS.
8) Non-linear method analysis have been introduced opposed to earlier version.
9) Load combination has been changed and it is different for parallel and non-parallel systems.
10) Load factors has been changed.
11) Four spectral shape factor for types of soil are defined for ESM & MRSM.
12) Seismic zoning map represents PGA in NBC 105:2020.

2.6 Comparison between NBC 205:1994 and Draft NBC 205:2012

The following are the major changes in NBC 205:2012 compared to NBC 205:1994:

1) Minimum grade of concrete should be 20 N/mm^2 at 28 days (Nominal mix 1:1:5:3).
2) Minimum length of bay should be 2.1m.
3) Recommended slab thickness in 2012 was 125mm while it was 100mm in 1994.
4) The overall depth of beam was increased to 355mm in 2012 as to 325mm in 1994.
5) The reinforcement detailing for all sections were updated in 2012.
6) Dimension of column was 9"X9" in 1994 while it was updated to minimum of 12"X12" in 2012.

2.7 Design Based Earthquake and Maximum Considered Earthquake

Maximum Considered Earthquake (MCE) is defined as the most extreme seismic ground motion expected to occur at a site defined as the ground shaking extent with a probability of exceeding 2% in 50 years. The level with probability of exceedance 10% in 50 years is designed based earthquake (DBE) (Warnitchai and Munir 2011). For normal design, DBE is used and performance level is Life Safety (NBC 2020). The ground motion for DBE is taken as 2/3rd of MCE.

2.8 Performance Level of building

The performance level describes the limiting condition of damage that may be considered acceptable for a given building in given seismicity (Baris et al., 2017). The limiting condition is determined by the damage in the structure, the risk to the building's users' life safety induced by the damage, and the building's post-earthquake functionality. The following performance level ranges have been designated:

Immediate Occupancy (IO)

The state of damage after an earthquake in which only minor structural damage has occurred and the building's basic gravity and lateral force resisting systems sustain nearly all of their pre-seismic state and strengths. The risk of life-threatening injury from failure of structure is low, and the structure should be secure for unlimited access and occupancy.

Life Safety (LS)

After an earthquake, a structure may have sustained substantial damage, but there may still be some protection from either a complete or partial structural collapse. No significant structural elements have collapsed or become loose, endangering life safety inside or outside the structure. Despite the possibility of earthquake-related injuries, there is very little chance that structural damage will result in a severe injury. Before the building can be used again, it will likely need to undergo major structural repairs.

Collapse Prevention (CP)

This level represents the maximum structural damage condition where the structural system of the building is about to partially or completely collapse. The structure has sustained severe damage, which may have resulted in a significant loss of stiffness and strength in the lateral force resisting system. Despite the structure's general stability, there is a high risk of injury from falling objects either inside and outside, and a major aftershock might cause the building to collapse. Before re-occupancy, it should be anticipated that considerable structural repairs will be required.

As per ATC-40, the performance level is to be evaluated by the drift value obtained after pushover analysis.

For drift below 0.01, the performance level is Immediate Occupancy, for 0.01-0.02 it is in Damage Control, 0.02 indicates life safety and 0.33vi/pi (shear force/gravity load) indicates Collapse Prevention/ Structural stability.

2.9 Pushover Analysis

A static procedure that estimates building’s performance and structural deformations in an earthquake using a nonlinear technique can be explained as pushover analysis (Khan 2013). The dynamic forces on a building are transferred to other parts of the structure during earthquakes as individual structural elements fail or lose its elastic property. By increasing loads up until the weakest link in the structure is identified and then changing the model to account for the structural changes brought on by the weak link, a pushover analysis replicates this occurrence. A second phase involves redistributing the loads. Until the second weak link is discovered, the superstructure is "pushed" once more. This procedure is repeated until a yielding trend is identified for the entire structure when subjected to earthquake loads.

Pushover analysis method has been applied in various retrofit design and guidelines. The performance-based design of new structures that depend on ductility or redundancy to withstand seismic loads can also benefit from it. The procedures for pushover analysis of buildings have been presented in the FEMA-273, 356, 440 and ATC-40. The procedures in these documents is based upon methodology of Freeman et al.(1975) and Freeman (1978). The FEMA-440 specification modifies the ATC-40 capacity spectrum method and refers to it as the equivalent linearization method (ELM).

2.10 Retrofitting of the structure

Retrofitting refers to strengthening of components to the current requirement so that the desired protection of lives can be guaranteed in future possible earthquakes. Retrofitting can be of component strengthening and/or whole modification of structure. There are many retrofitting techniques for reinforced concrete building such as reinforced RC jacketing, steel encasement or jacketing, addition of reinforced concrete shear walls, addition of reinforced walls, addition of steel bracing, base isolation, use of Fiber Reinforced Polymer (FRP), use of dampers, etc.

The process of retrofitting should not have negative consequences on the current state of structure. The selected technique should be such that the materials are easy to acquire and skilled work force are readily available. The cost of retrofitting should be less than a third of construction of new identical existing building.

2.11 Retrofitting Techniques

Retrofitting can be done on local and global level. It depends upon the vulnerability of the structure. The understanding of mode of failure, structural behavior and weak and strong design aspects exercise considerable influence on selection of retrofitting methods. Retrofitting method is usually aimed at increasing lateral resistance of the structure.

Structural level (or Global) retrofit methods:

Conventional methods includes adding new shear walls, steel bracing, infill walls into the existing frames. Non-conventional method includes seismic base isolation and use of dampers. Member level or Local retrofit method includes jacketing by concrete, steel, carbon fiber, glass fiber etc. The main purpose of jacketing is to increase longitudinal reinforcement and to increase shear strength. The jacketing can be of columns, beams, foundation, shear wall.

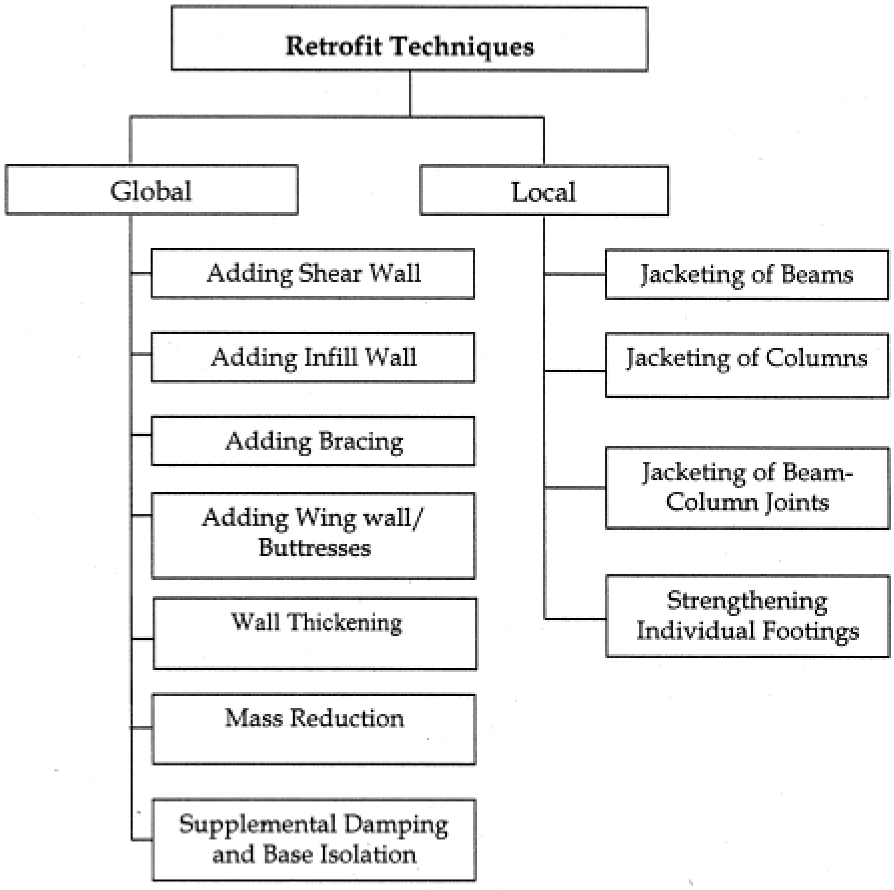


Figure 4 Retrofitting Techniques

1. RC jacketing of Columns

RC Jacketing is a technique for reinforcing and improving structural components. When the structural design is changed, it is used to enhance bearing load capacity. When a structural member fails, it is utilized to restore the integrity of the design (DUDBC 2016). It can be used on beams, columns and wall. The jacketing consists of added concrete with reinforcement in longitudinal and direction with stirrups covering the original column. The column's structural analysis determines the jacketing thickness as well as the quantity and diameter of the rebar used in the jacketing procedure. It enhances the column's ductility and flexibility. The column's ductility and shear strength are increased by the closely spaced reinforcement stirrups given in the jacket. It increases a building's ability to withstand lateral loads and prevents stiffness concentration, as is the case with shear walls.

2. RC Jacketing of Beams

Jacketing of beam is done to improve the stiffness and flexural strength. Whole length of beam should be jacketed.

3. Steel Jacketing of Columns

Steel jacketing is defined as the process of covering a part with plates of steel and grouting the space with non-shrinking material. It is a very efficient way to fix flaws like insufficient shear strength and poor lap splices at crucial points. Steel angles and strips are the widely utilized. Steel jacketing improve the ductility, and strength of columns. The jacket contributes to the increased lap splicing with increased ductility and stiffness, improving the shear behavior of structure.

4. Addition of RC Shear Wall

Shear walls are structural components used to withstand horizontal loads that are intended to be applied in-plane, usually due to seismic stresses. A frame can be strengthened by adding a stiff wall inside of it or by attaching one to it. This keeps the frame's shape and prevents rotation at the joints. Shear walls are crucial in high-rise structures that are vulnerable to seismic stresses. They also offer enough stiffness and strength to limit lateral displacements. The shear wall's size and design position have a major impact on how the structure behaves. Although the shear wall improves lateral strength, it also adds weight to the building and concentrates stiffness, which can cause columns to fail.

5. Addition of Steel Bracing

Steel bracing increases strength and stiffness of the structure. It increases the resistance to lateral loads by development of axial stress in its diagonal members. Diagonal members are fastened to steel plates, which are anchored with epoxy resins at the corners of every bay, while at ground floor, particular foundation arrangements are made for each bracing.

To prevent the building from experiencing undesirable torsion and to minimize in-plane abnormalities, the steel bracing should be installed in symmetrical placements. Although the building's lateral capacity is greatly increased by the use of steel braces, its rigidity is only slightly increased.

6. Carbon Fiber Reinforced Polymer

Fiber composite materials can be employed to reinforce concrete buildings externally for retrofitting purposes. Fiber composite have comparatively low density than concrete and are simple to attach, and can well be customized on the spot. Because of this, fiber composite has become increasingly popular as an outside reinforcement all over the globe. Synthetic fiber-based assembled fabric strips are placed to the concrete portion after already being treated with a resin binder. Because of its lightweight, installation can be performed easily with few labor costs and service interruptions. Additionally, these materials have corrosion resistance in chloride conditions, which helps lower repair costs. FRP is mostly utilized in two fields. FRP bar can be used in place of rebar or pre-stressing wires in concrete constructions is the first application. Retrofitting is the other use.

7. Shotcrete

Shotcrete is a layer of concrete sprayed in a surface. The thickness of shotcrete is less than RC jacketing. It is easier to apply in difficult areas and it can be achieved in a short amount of time and cost. The method can be very useful in seismic strengthening applications.

CHAPTER 3: RESEARCH METHODOLOGY

3.1 Research Approach

The research is quantitative in nature as it involves technical analysis. Furthermore, the research is based on simulation that is carried out using numerical analysis. Thus, the research falls under the post-positivist paradigm. Quantitative structural assessment is done based on data (approved building drawings built as per NBC 201/205-MRT) collected from municipal office.

3.2 Research Framework

A research framework is a conceptual illustration of the study's purpose. It covers the necessary steps required in order to accomplish the research's objective. A sequence of steps taken to accomplish the study objectives are utilized to construct the research framework. Figure 5 shows flowchart of the research framework.

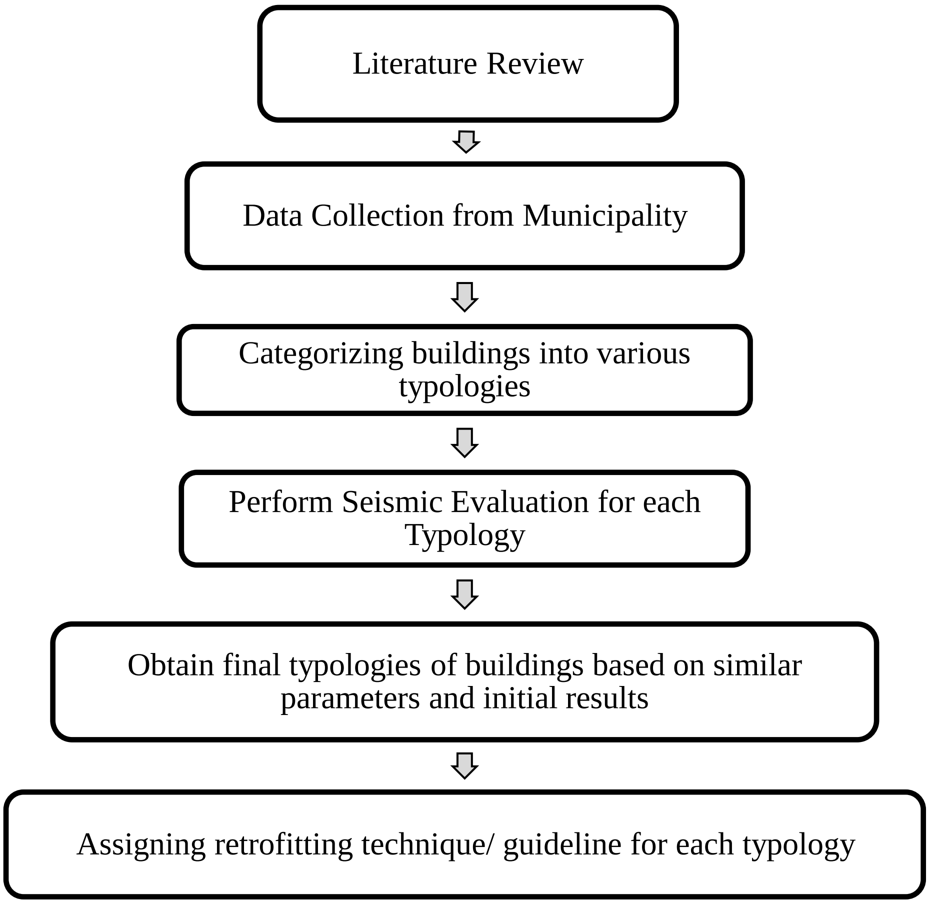


Figure 5: Research Framework

Data collection (approved building drawings) was done from municipality office. The buildings were categorized into various typologies based on sizes of frame, span, number of storeys, building and floor height, number of bays, dimension of structural components, grade of concrete.

The analysis and design was carried out using finite element software ETABS. The structural deficiencies for each typology were assessed and the ones requiring similar upgrading of structural components were generalized into final typology. The seismic performance level of buildings was evaluated by performing pushover analysis by FEMA 440/ATC 40's capacity spectrum method. The strength and deformation requirements of the structural system were used to estimate its performance, and these requirements were then compared to the capabilities available at the desired performance level.

Seismic performance of each typology was compared and buildings with similar parameters and performance patterns was generalized to obtain final typologies of the buildings. Finally, appropriate retrofitting techniques as per IITK-GSDMA, Seismic Retrofitting Guidelines of Buildings in Nepal, 2016 are suggested for each typology for Life Safety performance level complying with NBC 105:2020.

3.3 Study Area

The selected area for research is Lalitpur Metropolitan City of Lalitpur district. The city has an area of 15.43 square kilometers and is divided into 29 municipal wards. Lalitpur is one of the worst hit district in 2015 Gorkha earthquake.

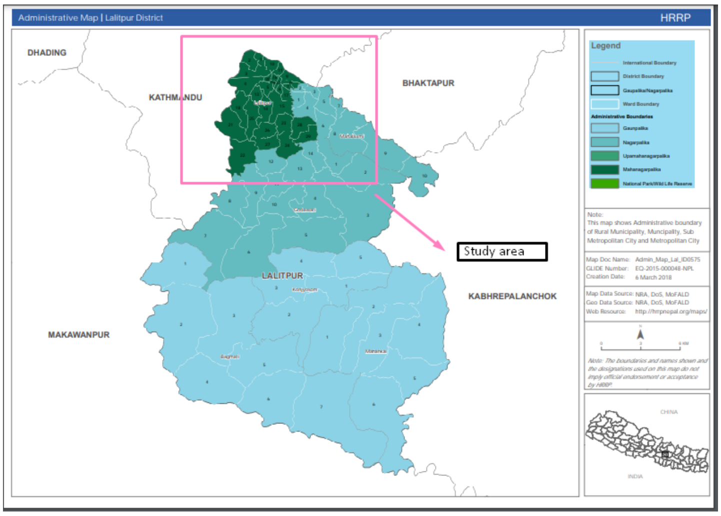


Figure 6: Map of LMC

3.4 Data Collection and data types

Primary data collection was used for data collection. All the required building drawings were provided by the municipal office from the server and archive files.

3.5 Expected Output

This thesis will benefit both house owner and Government of Nepal, which are listed below:

I. Seismic vulnerability assessment of each typology based on parameters of MRT will help house owners to understand current seismic performance of their buildings.

II. Suggested retrofitting techniques will be useful tool for LMC as well as Government of Nepal to develop general guidelines for retrofitting existing buildings built as per MRT.

CHAPTER 4: RESULTS AND DISCUSSIONS

4.1 Study Population and sample size

Simple random probability sampling was used. The population here is the total number of RCC houses(foundation) in Lalitpur district.

Sample size is obtained using Cochran's formula i.e.

$$\text{Sample size} = \frac{\dfrac{z^2 \times p\,(1-p)}{e^2}}{1 + \left(\dfrac{z^2 \times p\,(1-p)}{e^2 N}\right)}$$

Where,
z=z score=1.65

p=probability=0.5

E=margin of error=10%

N=Population Size=33734(CBS 2011)

Sample size taken=68

Sixty-eight samples in total were obtained from the LMC office. Each sample was studied individually to obtain the description of building and they were divided into various typology on the basis of their similarities of column size, no of storey, beam size, slab thickness, length, width, no of bays, building height, concrete grade, type of MRT code followed and reinforcements used. The specifics are in Appendix 1.

Criteria for selecting MRT Complaint buildings

The samples were individually studied first on the basis of structural layout and were compared if MRT criteria on structural layout restrictions has been followed. Buildings with similar layout, number of bays, column size, slab size, height of the building, length of bays were grouped under a similar typologies.

The restrictions for each MRT code for RCC buildings are as follows:

1. MRT 201:1994

Structural Layout Restrictions

1) Maximum Plan : 4.5mX3m = 14'9" X 9'10"
2) A (Length of Building) or B(Width of building) ≤ 6 Bays in length or 25m = 82'

   Each Bay ≤ 4.5m

3) H/A ≤ 3 (H=Height of the Building)
4) Area of slab ≤ 13.5 sqm = 145.31 sqft
5) H max = 11m = 36'1"

   Additional storey i.e. 3+ staircase allowed within 11m.

   Area of such storey ≤ 25% of typical floor.
6) No openings for structural infill wall.
7) $K_1 < 0.25A$ (where K1 is length of the wing)
8) Infill opening less than 10% located outside mid2/3rd.
9) Only parapet wall in cantilever.
10) At least teo infill (lateral load resisting) in each direction i.e X-axis & Y-axis.
11) At least 20% of wall should be in middle 2/3rd of total length in X-direction & Y-direction.
12) Concrete grade 15MPa Clause 5.1
13) Rebar fy = 415 N/mm^2 Clause 5.3

    Fy = 550 N/mm^2 for slab only.

    →7 Ø Fe550 or 8 Ø Fe415 → 6 Ø Fe250

2. MRT 205:1994

Structural Layout Restrictions

1) Maximum Plan : 4.5mX3m = 14'9" X 9'10"
2) A (Length of Building) or B(Width of building) ≤ 6 Bays in length or 25m = 82'

   Each Bay ≤ 4.5m
3) H/A ≤ 3 (H=Height of the Building)
4) Area of slab ≤ 13.5 sqm = 145.31 sqft
5) H max = 11m = 36'1"

   Additional storey i.e. 3+ staircase allowed within 11m.

   Area of such storey ≤ 25% of typical floor.
6) $K_1 < 0.25A$ (where K1 is the length of the wing)
7) Only parapet wall in cantilever.
8) Concrete grade 15MPa
9) Rebar fy = 415 N/mm^2

3. MRT 205:2012

Structural Layout Restrictions

1) Maximum Plan : 4.5mX3m = 14’9” X 9’10”
2) A (Length of Building) or B(Width of building) ≤ 6 Bays in length or 25m = 82’
   Each Bay ≤ 4.5m
3) Minimum length of bay=2.1m
4) H/A ≤ 3 (H=Height of the Building)
5) Area of slab ≤ 13.5 sqm = 145.31 sqft
6) H max = 11m = 36’1”
   Additional storey i.e. 3+ staircase allowed within 11m.
   Area of such storey ≤ 25% of typical floor.
7) $K_1 < 0.15A$ (where K1 is the length of the wing)
8) Only parapet wall in cantilever. Size of cantilever should not exceed 1 metre.
9) Concrete grade 20 MPa
10) Rebar fy = 415/550 N/mm^2

4.2 Categorizing and Seismic Evaluation of various typologies

Twenty typology of buildings were observed whose details can be found in Appendix 2. Seismic evaluation by Equivalent Static Method for each typology was carried out in ETABS for determining structural deficiencies in the building. The load combinations, factor and modifiers and all other parameters were taken from NBC 105:2020. Pushover analysis was done for evaluating the seismic performance of building for Maximum Considered Earthquake (MCE). The performance point was evaluated using FEMA 440 Equivalent linearization with values of demand spectral response acceleration Ss and S1 with that of Kashmir, Pakistan as Nepal do not have defined parameters. Ss is taken as 1.38g and S1 as 0.4 with soil type E as per NEHRP soil classification of Kathmandu valley. Ten typologies based on similar parameters, performance and deficiencies were finalized. Critical components and overall capacity of building was observed and retrofitting techniques were applied for global and local strengthening as required. Various trial and iterations are done to obtain optimum retrofitting strategy. The performance of retrofitted structures are evaluated and only such retrofitting methods are suggested from which the performance level is in Life Safety, Damage Control and Immediate Occupancy.

From the sample study, following results in 4.2.1 were obtained.

4.2.1 Chart showing distribution of building samples.

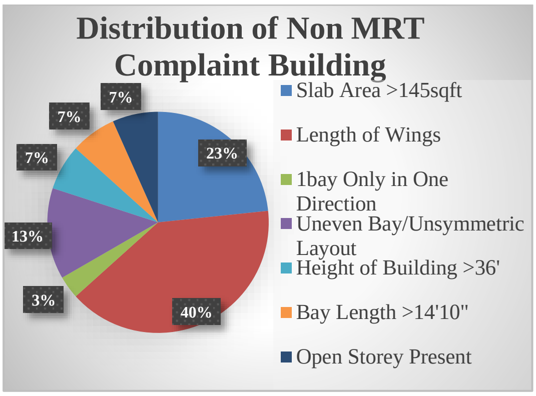


Figure 7 Distribution of Non MRT Compliant Building

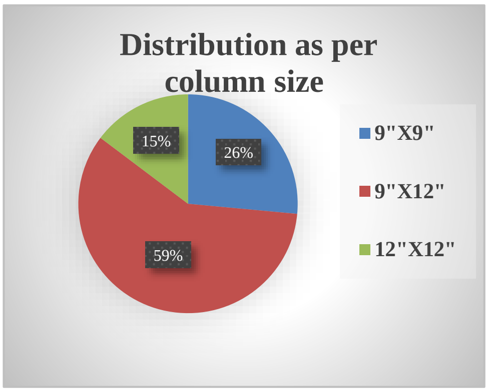


Figure 8 Distribution of MRT compliant buildings as per column size

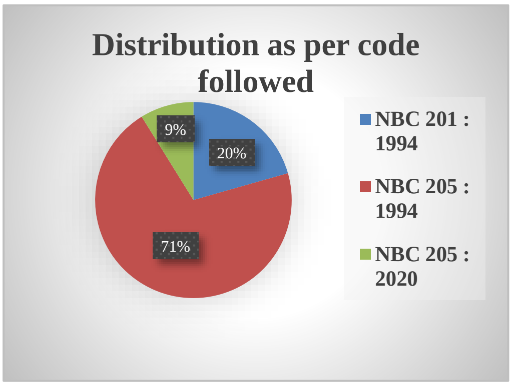


Figure 9 Distribution of MRT compliant buildings as per Code Followed

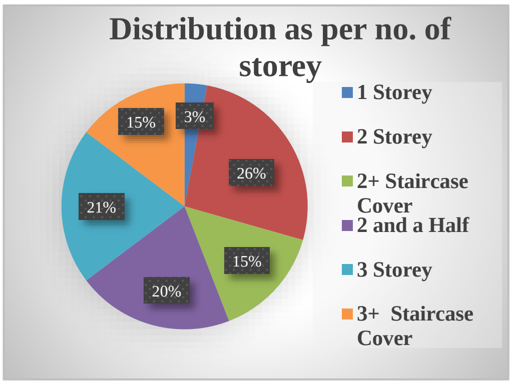


Figure 10 Distribution of MRT compliant buildings as per number of storey

It can be observed that the majority of buildings i.e. 40 %, which were non-compliant with MRT codes, had the length of wings restrictions violated. Similarly, 23% of such buildings had slab area greater than recommended 145 square feet.

More than half (59%) of MRT compliant buildings had column size of 9”X12”. Seventy one percent of MRT compliant buildings followed NBC 205:1994. Similarly, prevalence of two storey and two and half storey with/without staircase cover was observed.

4.3 **Design Input**

4.3.1 Load Calculations:

4.3.1.1 Dead Load

Dead loads are calculated based on unit weights of the specified construction materials in accordance with NBC 102:1994.

Reinforced concrete: 25 KN / m^3

Brick work with plaster: 19.2 KN/m^3

Sand/ cement screed: 20 KN/m^2

4.3.1.2 Superimposed Dead Loads

Based on architectural drawing of the building, dead loads due to partition walls, floor finish are taken as superimposed loads.

4.3.1.3 Live Loads

The Live Load for building has been adopted as given NBC 103:1994.

Room	2 kN/m^2
For passage, staircase, balconies	3 kN/m^2
For Roof	1.5 kN / m^2

4.4 **Analysis Approaches**

The structure was modeled as a three-dimensional ordinary RC moment resisting frame of main structural member beam and column to determine the required strength of the structure. The effect of infill brick wall is not considered while analyzing the structure. The analysis was performed for various combinations as per code NBC 105:2020.

The analysis of the building was done in two cases:

Case I: Considering existing building

Case II: Considering Retrofitted Building

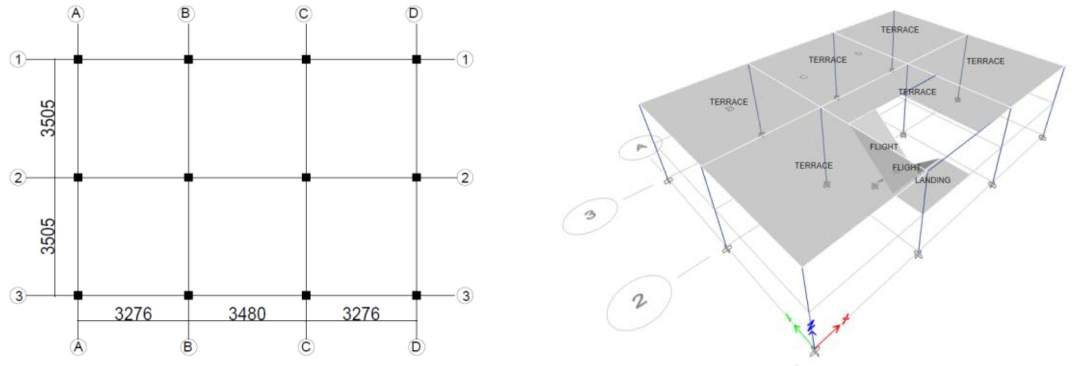
Pushover analysis was done and the performance level of building was obtained based on procedure FEMA 440 Equivalent Linearization. The expected performance of structure for residential buildings were life safety or higher as per ATC-40.

The retrofitting method were applied based on local or global strengthening requirement. If the capacity of the structure was sufficient as per the demand after the local strengthening (concrete and steel jacketing) then global strengthening are not required. However, if it is not sufficient then global strengthening (addition of shear wall) was proposed. Here, multiple iterations are done to evaluate the expected performance and iterations are done until the structure meets Life Safety performance level.

4.4.1 **Typology A**

Predefined Specifications:

Building Height: 12'5"	Plinth Area: Upto 700 sqft
No. of Storey: 1	Floor Height: 8'8"
Column Size: 9"x9"	Concrete Grade: M15
Beam Size: 9"x13"	MRT Code: 205:1994
Slab Thickness: 100mm	MRT Satisfied: Yes
Length: 29'6"	Column Reinforcements: Corners: 4-16+2-12, Remaining: 6-16
No. of Bays in Length: 3	Beam Reinforcements: 2-16+2-12(top), 2-16+1-16(bot)
Breadth: 23'6"	Slab Reinforcements: 10 mm @8"c/c
No. of Bays in Breadth: 2	Maximum length of bay=3.5m



(a) (b)

Figure 11: (a) Floor Plan of Typology A (b) 3D Model of Typology A

4.4.1.1 Case I: Considering existing building

1. Seismic analysis considering existing building as per NBC 105:2020

The building showed maximum drift of 0.001, which is less than allowable 0.00625, and maximum displacement of 2.5mm and 2.6mm in EQx and EQy direction respectively. Maximum allowable displacement is 0.00625* height of the building which is 23.75 mm in this case. Torsional irregularity was found at A3 and D3 under EQx loading. Hence, those columns must be revised.

All concrete members passed in size but the area of reinforcement for column was not sufficient. Table (1) and table (2) shows the details of existing and required reinforcement for columns and beam respectively. Where Ast is required area of reinforcement while Ar is deficit area of reinforcement in that section.

Table 1 Detailing of Column Reinforcement

Column label	Ast required	Ast provided	Deficit(Ar)	1.33 Ar
A1	989	1030	0	0
B1	2091	1206	885	1177.05
C1	2044	1206	838	1114.54
D1	1005	1030	0	0
A2	2098	1206	892	1186.36
B2	1989	1206	783	1041.39
C2	1966	1206	760	1010.8
D2	2109	1206	903	1200.99
A3	1038	1030	8	10.64
B3	2094	1206	888	1181.04
C3	2014	1206	808	1074.64
D3	977	1030	0	0

Table 2 Detailing of Beam Reinforcement

Beam Label	Ast required	Ast provided
1-1 Top	157	628
1-1 Bottom	157	603
2-2 Top	157	628
2-2 Bottom	157	603
3-3 Top	157	628
3-3 Bottom	157	603

A-A Top	157	628
A-A Bottom	157	603
B-B Top	157	628
B-B Bottom	157	603
C-C Top	157	628
C-C Bottom	157	603
D-D Top	157	628
D-D Bottom	157	603

2. Pushover analysis of existing structure for MCE

Table 3: Base shear vs Displacement for Push x direction

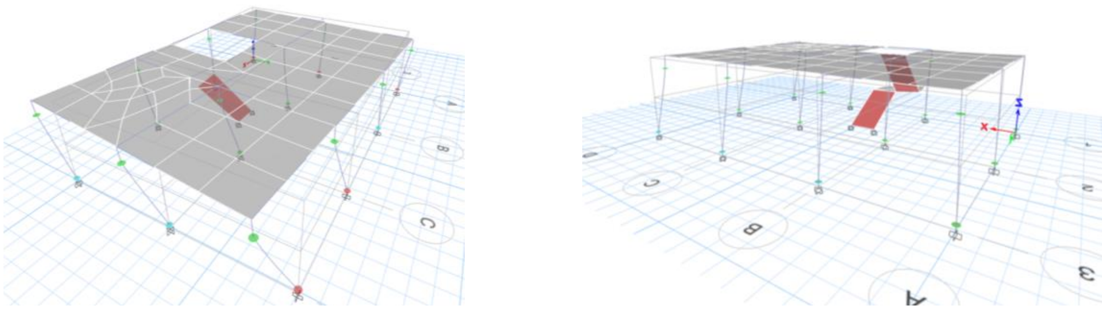
Step	Monitored Displacement	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	60	0	0	0	0	60	0	0	0	60
1	4	221	59	1	0	0	0	60	0	0	0	60
2	17	695	35	24	1	0	0	37	14	3	6	60
3	15	655	35	22	1	2	0	36	13	2	9	60

Table 4 Base shear vs Displacement for Push Y direction

Step	Monitored Displacement	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	60	0	0	0	0	60	0	0	0	60
1	5	167	59	1	0	0	0	60	0	0	0	60
2	8	251	48	12	0	0	0	60	0	0	0	60
3	40	660	36	24	0	0	0	36	17	5	2	60

The seismic weight of the building (Dead Load + 0.3Live Load) was applied as lateral force gradually until the target displacement was reached or till the collapse of structure. The performance point i.e. the non linear demand-capacity equivalence point was found at base shear 566KN and displacement of 13mm for Push X condition and 486 KN with displacement of 27mm which is less than the seismic weight of 575KN.

The performance point and formation of first plastic hinges in Collapse Prevention level occurs between step 1 and 2 from the table 3. We can locate the position of hinges formation and intervene for the retrofitting techniques.



(a) (b)

Figure 12 (a) Hinge State in Step 3(Push X) (b) Hinge State in Step 3 (Push Y)

Here, from seismic analysis, interior and face columns required additional reinforcements and after pushover analysis it was observed that the corner columns formed inelastic hinges which suggests strengthening of such columns as well. Here, we consider local enhancements i.e. RC jacketing and steel jacketing for retrofitting.

4.4.1.2 Case II: Considering Retrofitted Building

<u>Trial 1: RC jacketing in columns A3,C3, D3, A2 and steel jacketing in remaining columns deficit in reinforcements (All except A1, D1).</u>

As per IS 15988:2013, the deficit area of steel should be increased by 33%.

Provide 4 nos of 12 mm and 4 nos of 16mm bar, Ast (added) =1257 mm^2

Minimum Jacketing to be provide=100mm=4”

New concrete jacketed column size=17”*17”

Similarly, angle of L50X50X5 (IS 15988:2013) at four corners with plates of 5mm was used which has a total area of 1916 mm^2 for steel jacketing. The new seismic weight of structure was 680 KN. After the pushover analysis of retrofitted building, the base shear at performance point was found to be 699 KN and displacement was 3.9 mm. The maximum drift was 0.001. As per ATC-40, the building lies in the Immediate Occupancy performance level.

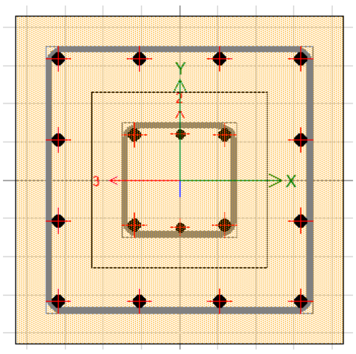


Figure 13: Concrete Jacketing

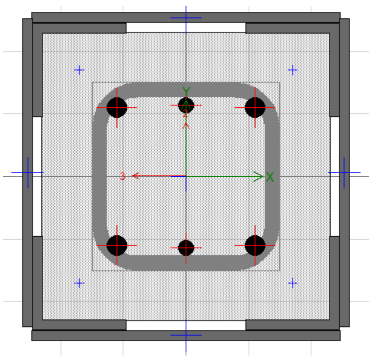


Figure 14 Steel Jacketing

Table 5 Base shear vs Displacement

Step	**Monitored Displ**	**Base Force**	**A-B**	**B-C**	**C-D**	**D-E**	**>E**	**A-IO**	**IO-LS**	**LS-CP**	**>CP**	**Total**
	mm	kN										
0	0	0	60	0	0	0	0	60	0	0	0	60

1	3	631	58	2	0	0	0	60	0	0	0	60
2	4	710	49	11	0	0	0	60	0	0	0	60
3	45	3024	16	44	0	0	0	35	21	2	2	60
4	55	3584	14	45	1	0	0	33	23	0	4	60
5	55	3524	14	45	0	1	0	33	22	1	4	60
6	65	4061	10	48	1	1	0	30	19	7	4	60
7	66	4026	10	47	2	1	0	30	18	7	5	60
8	66	4026	10	47	2	1	0	30	18	7	5	60
9	66	4025	10	47	2	1	0	30	18	7	5	60
10	66	4026	10	47	2	1	0	30	18	7	5	60
11	66	4026	10	47	2	1	0	30	18	7	5	60
12	66	4026	10	47	2	1	0	30	18	7	5	60

From the table 5, it can be observed that no plastic hinges are formed between step 1 and 2, hence RC jacketing and steel jacketing technique increases the capacity and performance of the building.

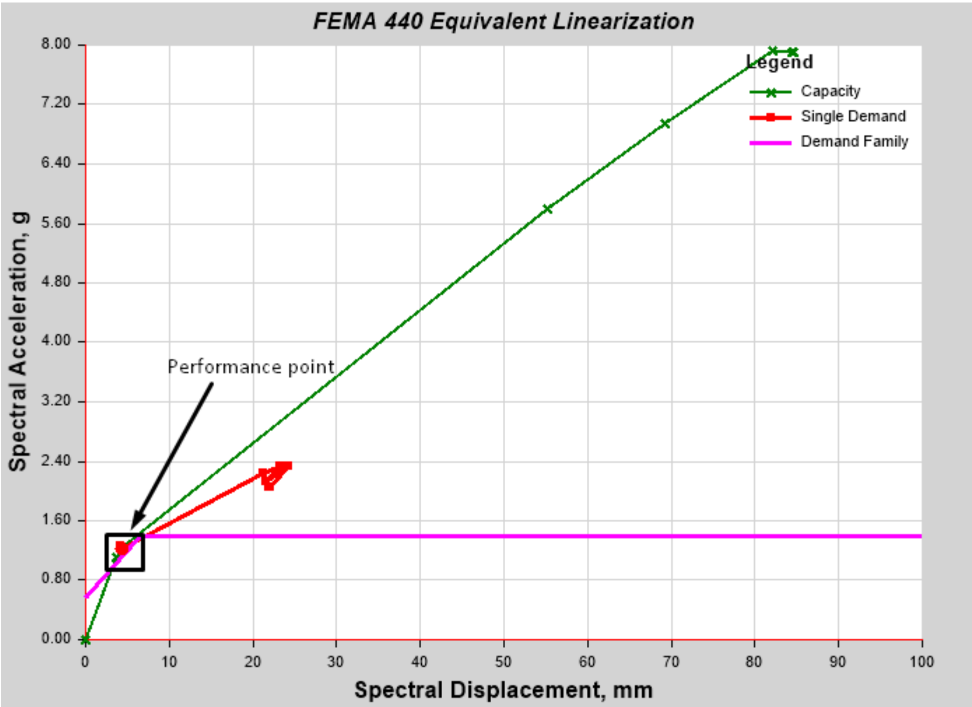
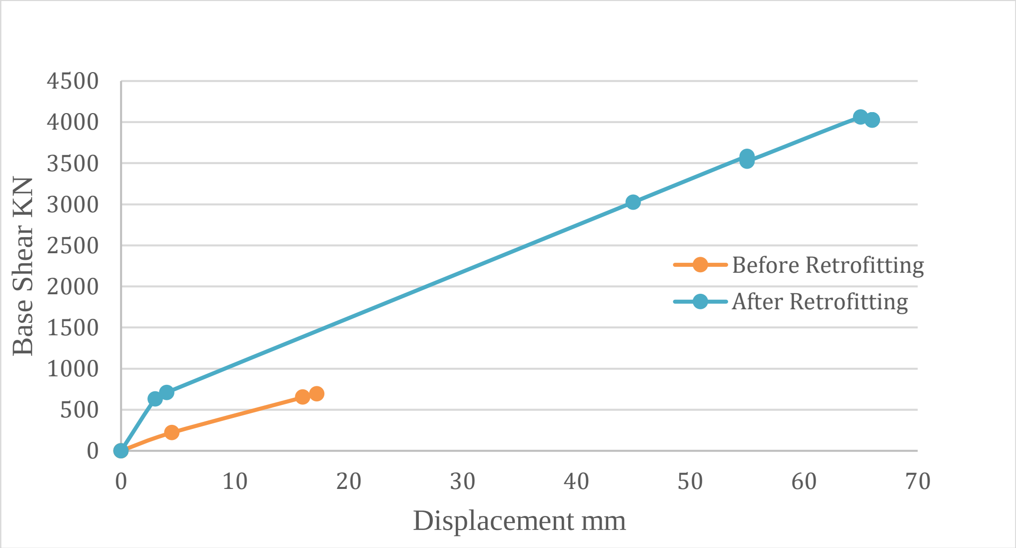


Figure 15 Performance point of retrofitted structure



Graph 1 Pushover curve before and after retrofitting for typology A

The above graph 1 shows the pushover curve of the building before and after retrofitting from which it can be interpreted that the performance (capacity) of building has improved after the application of assigned retrofitting strategy.

4.4.2 **Typology B**

Predefined Specifications:

Building Height: 18’8”	Plinth Area: 600-850 sqft
No. of Storey: 2	Floor Height: 9’4”
Column Size: 9”x9”	Concrete Grade: M15
Beam Size: 9”x13”	MRT Code: 205:1994
Slab Thickness: 100mm	MRT Satisfied: Yes
Length: 28’7” – 29’9”	Column Reinforcements: All 4-16
No. of Bays in Length: 3	Beam Reinforcements: 2-12+1-12(top), 2-12+1-12(bot)
Breadth: 23’3” – 29’6”	Slab Reinforcements: 8 dia and 10 dia @6"c/c
No. of Bays in Breadth: 2	

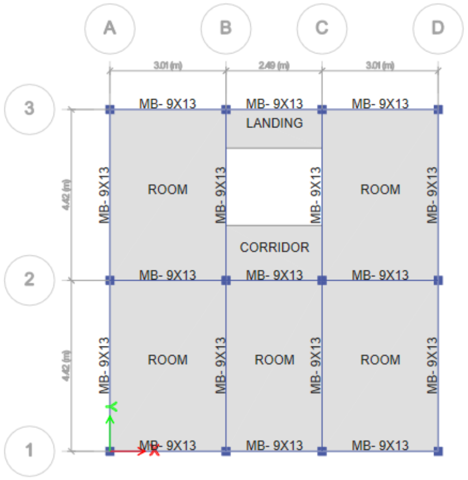


Figure 16: Plan of the Typology B

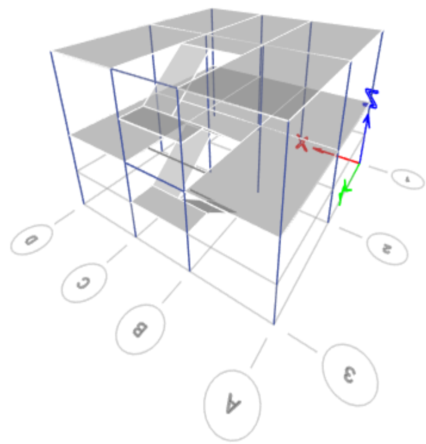


Figure 17: 3D Model of Typology B

Case I: Considering existing building

1. Seismic analysis considering existing building as per NBC 105:2020

The maximum drift of 0.001 was observed, which is less than allowable 0.00625, and maximum displacement of 2.7mm and 2.8mm in EQx and EQy direction respectively. Maximum allowable displacement is 0.00625* height of the building which is 35.5 mm in this case.

All concrete members passed in size but the area of reinforcement for column was not sufficient. Table (6) and table (7) shows the details of existing and required reinforcement for columns and beam respectively.

Table 6 Detailing for Column Reinforcements

Column Label	Ast required	Ast provided	Deficit (Ar)	1.33 Ar
A1	1266	804	462	614
B1	2217	804	1413	1879
C1	1302	804	498	662
D1	2236	804	1432	1905
A2	1508	804	704	936
B2	1569	804	765	1017
C2	2014	804	1210	1609
D2	1537	804	733	975
A3	1155	804	351	467
B3	2235	804	1431	1903
C3	2275	804	1471	1956
D3	1171	804	367	488

Here, all columns deems to have less reinforcement than required.

Table 7 Detailing for Beam Reinforcements

Beam Label	Ast required	Ast provided
1-1 Top	157	628
1-1 Bottom	157	603
2-2 Top	157	628
2-2 Bottom	157	603
3-3 Top	157	628
3-3 Bottom	157	603
A-A Top	157	628
A-A Bottom	157	603
B-B Top	157	628
B-B Bottom	157	603
C-C Top	157	628
C-C Bottom	157	603
D-D Top	157	628
D-D Bottom	157	603

2. Pushover analysis of existing structure for MCE

The seismic weight of the building was 1410 KN. From the pushover analysis, it was found that the capacity curve does not meet the demand curve. Maximum base shear at collapse point was 643KN. From the table 8, we can see that the collapse hinges starts between step 1 and 2, hence retrofitting is required to increase global as well as local capacity of the structure.

Table 8 Base shear vs displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	116	0	0	0	0	116	0	0	0	116
1	25	306	114	2	0	0	0	116	0	0	0	116
2	48	445	96	20	0	0	0	108	7	0	1	116
3	78	544	86	30	0	0	0	99	15	0	2	116
4	110	635	76	40	0	0	0	95	18	1	2	116
5	127	678	73	42	1	0	0	94	17	3	2	116
6	127	638	73	42	0	1	0	94	17	2	3	116
7	128	643	73	41	1	1	0	94	16	3	3	116
8	125	608	73	41	1	1	0	94	16	3	3	116

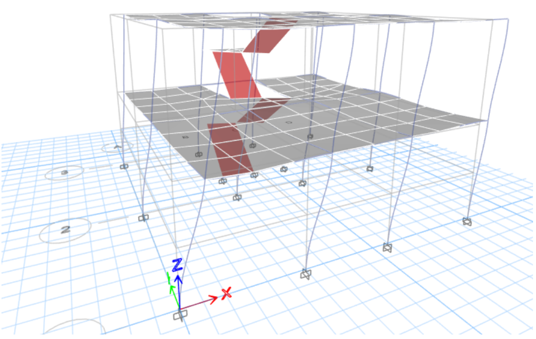


Figure 18: Hinge state in Step 1

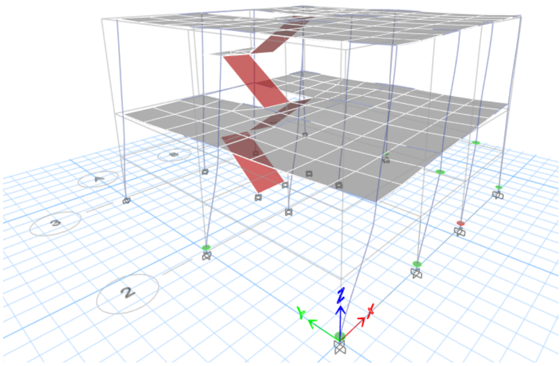


Figure 19: Hinges state in Step 2

4.4.2.1 Case II: Considering Retrofitted Building

Trial 1: RC jacketing in B2, B3 and steel jacketing in remaining columns

The RC jacketing was provided with 8 numbers of 16 mm bars with ties of 8mm with thickness of concrete jacket as 100mm. Angle of L50X50X5 (IS 15988:2013) at four corners with plates of 5mm was used for steel jacketing.

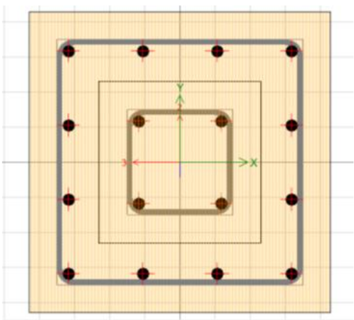


Figure 20: RC jacketing

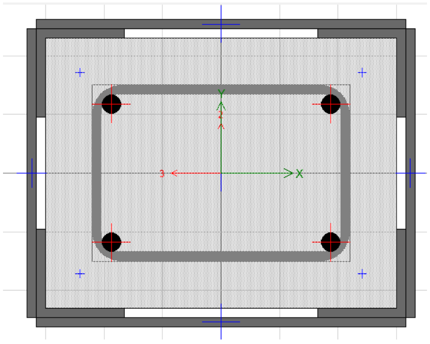


Figure 21: Steel Jacketing

Pushover analysis after retrofitting

The new seismic weight of the building was 1410 KN. The base shear, displacement and spectral acceleration at performance point was 1522 KN, 40mm, 1.31g respectively.

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	116	0	0	0	0	116	0	0	0	116
1	7	362	115	1	0	0	0	116	0	0	0	116
2	21	1107	85	31	0	0	0	116	0	0	0	116
3	55	1823	60	56	0	0	0	108	6	0	2	116
4	90	2512	53	63	0	0	0	97	17	0	2	116

5	123	3078	44	72	0	0	0	79	35	0	2	116
6	154	3595	38	78	0	0	0	69	45	0	2	116
7	165	3764	35	80	1	0	0	65	48	1	2	116
8	165	3587	34	81	0	1	0	65	47	2	2	116
9	170	3691	34	80	1	1	0	65	47	2	2	116
10	170	3644	34	80	1	1	0	65	47	2	2	116

From the table, it can be observed that the performance point lies between step 2 and step 3 with first collapse prevention hinges forming in step 3.

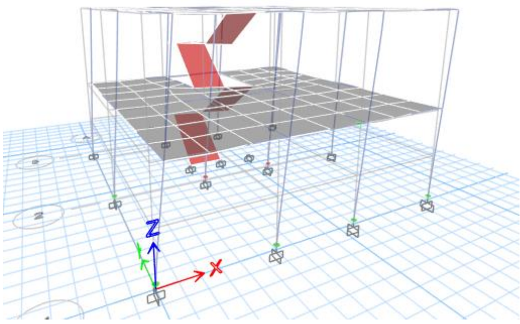


Figure 22: Hinges formation in Step3

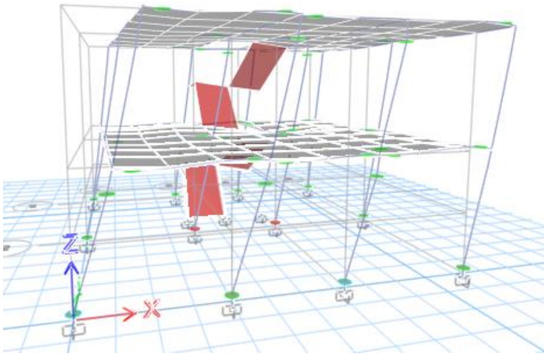


Figure 23: Hinges formation in Step 10

At final step, hinges are formed around the columns with staircase. The maximum drift was found to be 0.007 (<0.01), hence the retrofitted structure falls under Immediate Occupancy (IO).

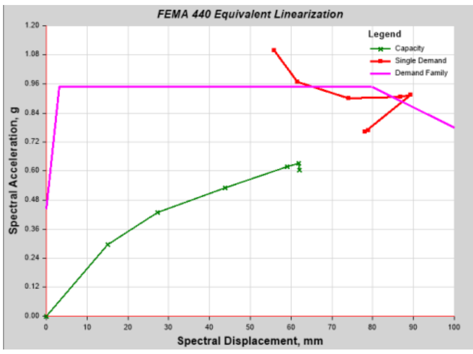


Figure 24 Demand Capacity Curve Before Retrofitting

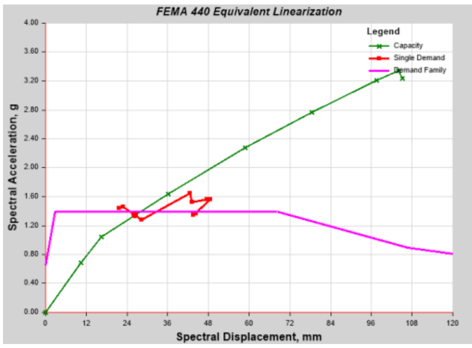
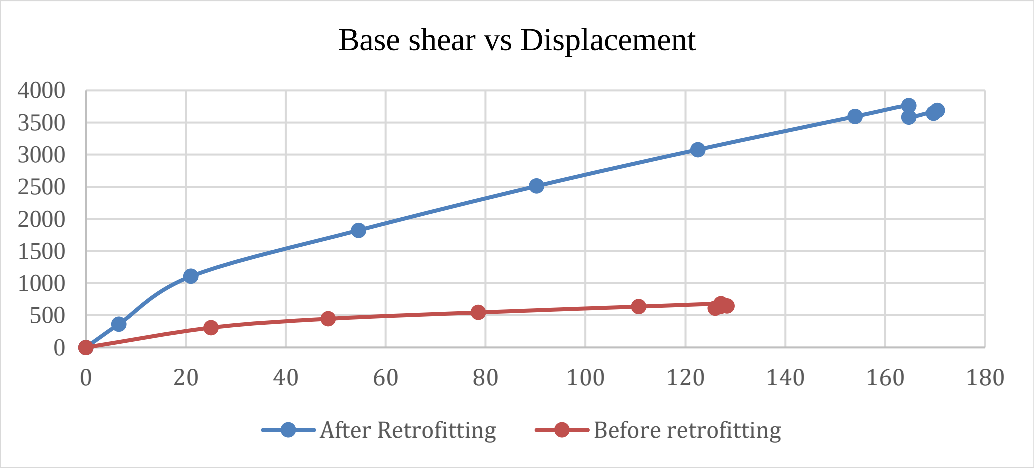


Figure 25 Demand Capacity Curve After Retrofitting

Here, the performance point was not obtained for the building in its original condition but after retrofitting the perfromance point was obtained with desirable drift i.e. performance level.



Graph 2 Pushover curve before and after retrofitting for typology B

The above graph 2 shows the pushover curve of the building before and after retrofitting from which it can be interpreted that the performance (capacity) of building has improved after the application of assigned retrofitting strategy.

4.4.3 **Typology C**

Predefined Specifications:

Building Height: 29’2”	Plinth Area: 600-800 sqft
No. of Storey: 3	Floor Height: 9’
Column Size: 9”x9”	Concrete Grade: M15
Beam Size: 9”x13”	MRT Code: 205:1994
Slab Thickness: 100mm	MRT Satisfied: Yes
Length: 23’-26”	Column Reinforcements: All 4-16
No. of Bays in Length:	Beam Reinforcements: 2-16+2-12(top), 2-16+1-16(bot)
Breadth: 23’3” – 29’6”	Slab Reinforcements: 10 dia @6"c/c
No. of Bays in Breadth: 2	

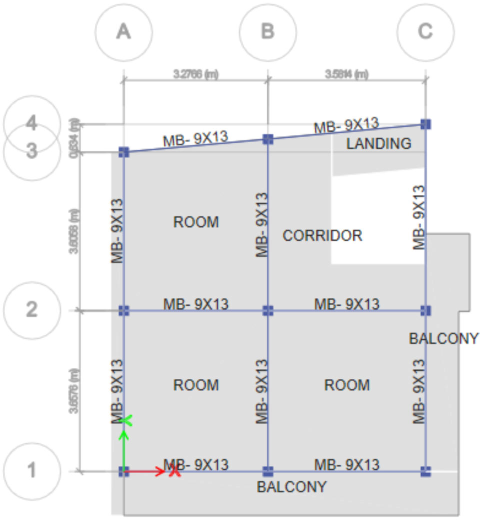


Figure 26: Plan of the Typology C

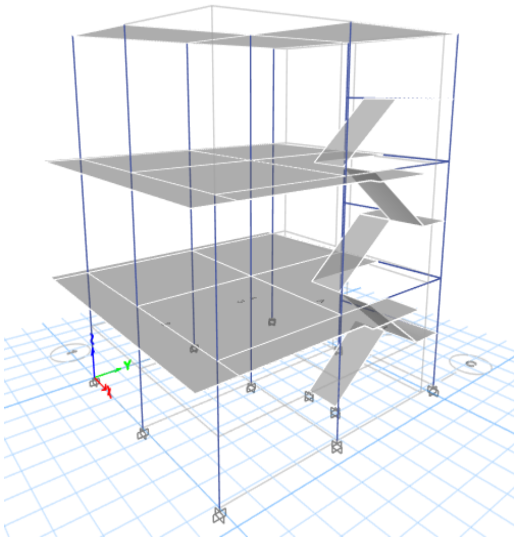


Figure 27: 3D Model of Typology C

4.4.3.1 Case I: Considering existing building

1. Seismic analysis considering existing building as per NBC 105:2020

From the computer analysis, it was found that the reinforcement in columns did not meet the design requirements. The maximum drift was 0.006 in Eqx and 0.0064 in EQy direction. The roof displacement was 44mm and 48 mm respectively for EQX and EQy directions, which was less than maximum allowable displacement of 55.56mm.

Table 9 Detailing for Column Reinforcements

Column Label	Ast required	Ast provided	Deficit	1.33 Ar
A1-storey 1	2237	1030	1207	1605.31
A1-storey 2	1723	1030	693	921.69
A1-storey 3	1130	1030	100	133
B1- storey 1	2716	1030	1686	2242.38
B1- storey 2	2275	1030	1245	1655.85
B1- storey 3	1557	1030	527	700.91
C1-storey 1	1589	1030	559	743.47
C1-storey 2	1540	1030	510	678.3
A2- storey1	2379	1030	1349	1794.17
A2- storey2	1995	1030	965	1283.45
A2- storey3	2178	1030	1148	1526.84
B2- storey1	2435	1030	1405	1868.65
B2- storey2	1933	1030	903	1200.99
B2- storey3	2076	1030	1046	1391.18
C2- storey1	3001	1030	1971	2621.43
C2- storey2	2678	1030	1648	2191.84
C2- storey3	2088	1030	1058	1407.14
A3- storey1	1820	1030	790	1050.7
A3- storey2	1871	1030	841	1118.53

A3- storey3	1319	1030	289	384.37
C4- storey1	2969	1030	1939	2578.87
C4- storey2	1230	1030	200	266
C4- storey3	1431	1030	401	533.33

Table 10 Detailing for Beam Reinforcements

Beam Label	Ast required	Ast provided
1-1 Top	405	628
1-1 Bottom	256	603
2-2 Top	459	628
2-2 Bottom	353	603
3-3 Top	398	628
3-3 Bottom	338	603
A-A Top	443	628
A-A Bottom	264	603
B-B Top	337	628
B-B Bottom	257	603
C-C Top	612	628
C-C Bottom	368	603

From the table 9, we can conclude that the reinforcement used for columns are not sufficient in longitudinal direction. Shear reinforcements were sufficient for all elements. To know the seismic performance level of existing building, we perform nonlinear static analysis i.e. pushover analysis.

2. Pushover analysis of existing structure

The seismic weight of the building was 1746 KN. The performance point was not obtained for the existing structure and the maximum base shear obtained was only 585 KN (Table 11) which means the building will collapse before reaching its capacity.

Table 11 Base Shear vs Displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	120	0	0	0	0	120	0	0	0	120
1	40	242	120	0	0	0	0	120	0	0	0	120
2	47	291	119	1	0	0	0	120	0	0	0	120
3	70	407	109	11	0	0	0	119	1	0	0	120
4	92	470	102	18	0	0	0	112	8	0	0	120
5	134	545	94	26	0	0	0	102	18	0	0	120
6	140	556	94	26	0	0	0	101	19	0	0	120
7	141	557	94	26	0	0	0	101	19	0	0	120
8	158	585	91	28	1	0	0	99	20	0	1	120
9	148	491	91	27	1	1	0	99	17	2	2	120

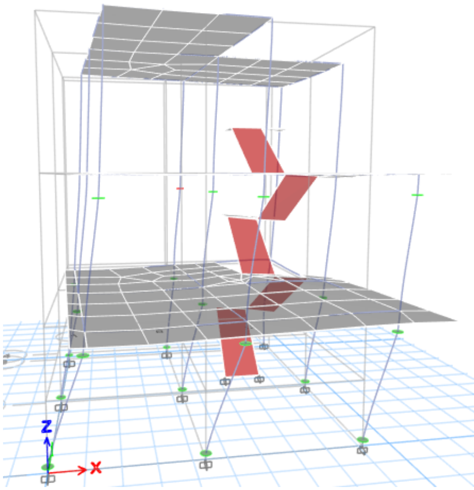


Figure 28 Hinge state in Step 8

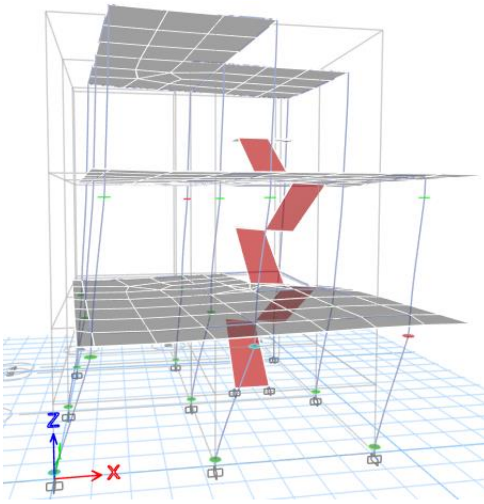


Figure 29 Hinge state in Step 9

Here, columns B2, C2 are critical.

4.4.3.2 Case II: Considering Retrofitted Building

Trial 1: RC jacketing in selected columns

Columns B2, C2 and D2 were retrofitted using RC jacketing of 4" thickness with 8 numbers of 16 mm rebar while rest were steel jacketed with L50X50X5 plates. New seismic weight of building was 1796KN. From the pushover analysis, it was found that the base shear, displacement , spectral acceleration was 1185.7 KN, 244 mm, 1.48g respectively at performance point which is less than the demand base shear. Hence, the suggested retrofitting option is not suitable.

Trial 2: RC jacketing in all columns

The RC jacketing was provided with 8 numbers of 16 mm bars and 4 numbers of 12mm rebars with ties of 8mm with thickness of concrete jacket as 100mm. Angle of L50X50X5 was used. New seismic weight was 2028KN. The base shear, displacement and spectral acceleration was found to be 2132 KN, 158mm and 1.49g respectively.

Table 12 Base Shear vs Displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	128	0	0	0	0	128	0	0	0	128
1	24	494	127	1	0	0	0	128	0	0	0	128
2	58	1099	90	38	0	0	0	128	0	0	0	128
3	118	1732	74	54	0	0	0	116	11	0	1	128

4	167	2230	70	58	0	0	0	109	18	0	1	128
5	207	2597	64	62	1	1	0	101	25	1	1	128
6	207	2579	64	61	0	3	0	101	23	3	1	128
7	217	2674	62	62	1	3	0	99	25	3	1	128
8	217	2554	61	63	0	4	0	99	24	4	1	128
9	227	2645	58	65	1	4	0	99	24	4	1	128
10	227	2646	58	65	1	4	0	99	24	4	1	128

From the table 12, we can observe that the performance point lies between step 3 and 4. One hinge at CP point was observed until step 10. The maximum drift obtained was 0.02. The building lies in Life Safety performance level. One hinge at column B2 (near staircase) was observed.

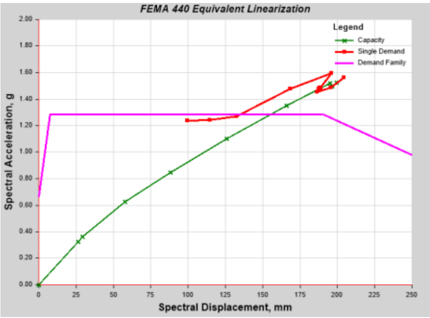


Figure 30 Demand Capacity Curve after Trial 1 Retrofitting

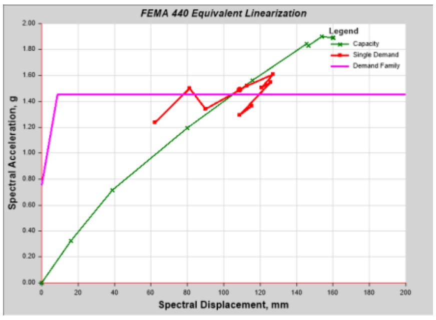
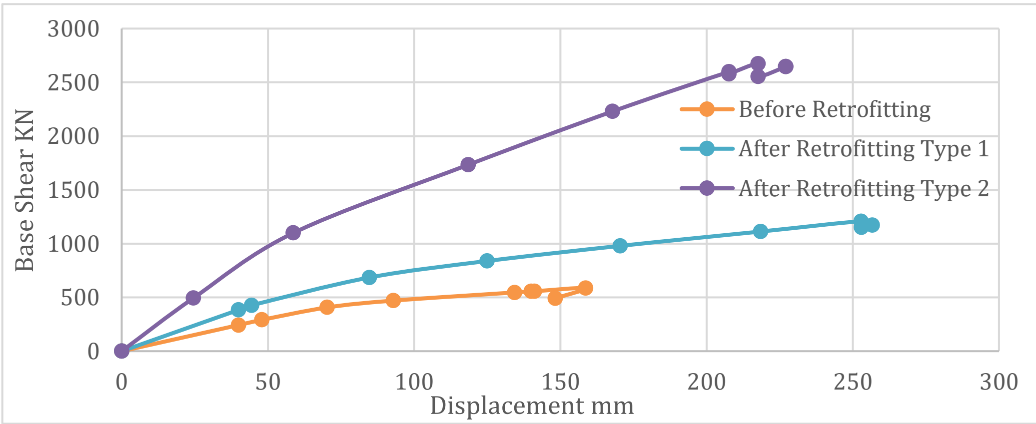


Figure 31Demand Capacity Curve after Trial 2 Retrofitting



Graph 3 Pushover curve before and after retrofitting for typology C

The above graph 3 shows the pushover curve of the building before and after retrofitting from which it can be interpreted that the performance (capacity) of building has improved after the application of assigned retrofitting strategy.RC jacketing provided the best performance.

4.4.4 **Typology D**

Predefined Specifications:

Building Height: 18’8”	Plinth Area: 553.92
No. of Storey: 2	Floor Height: 9’4”
Column Size: 9”x12”	Concrete Grade: M15
Beam Size: 9”x13”	MRT Code: 201:1994
Slab Thickness: 100mm	MRT Satisfied Yes
Length: 23’1”	Column Reinforcements: All: 4-16
	Stirrups 7 dia
No. of Bays in Length: 2	Beam Reinforcements:
Breadth: 24’	Top: 2-16(T) +2-12 Ex
No. of Bays in Breadth: 2	Bottom: 2-16+1-16 ex
	Stirrups: 7 dia @ 4"c/c and 6"c/c
	Slab Reinforcements: 8 dia @6"c/c and 7 dia

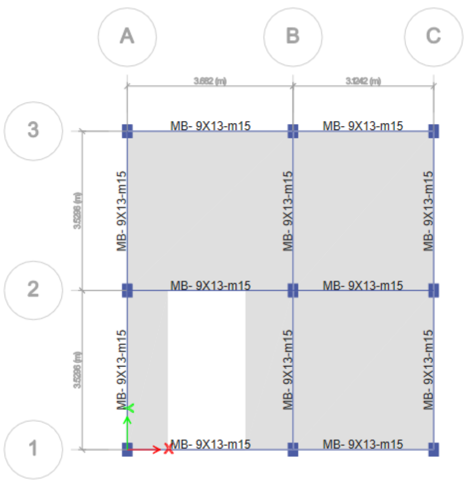


Figure 32: Plan of Typology D

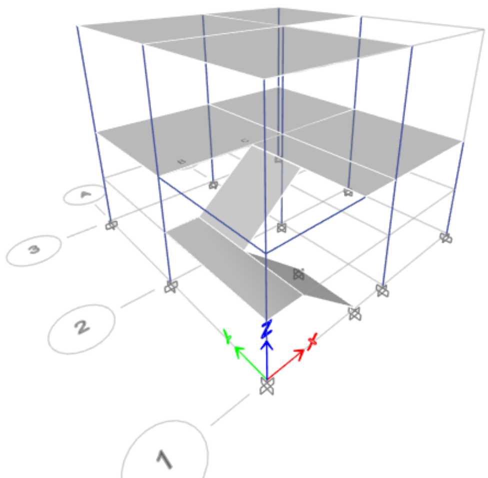


Figure 33: 3D Model of Typology D

4.4.4.1 Case I: Considering existing building

1. Seismic analysis considering existing building as per NBC 105:2020

It was found that the maximum drift and deflection was 0.006 and 28 mm respectively, which are within the permissible range of 0.00625 and 35.62mm. The sizes and reinforcement for all members except for longitudinal rebars of columns were sufficient.

The details of existing and required reinforcement for columns is shown in table 13.

Table 13 Detailing for Column Reinforcements

Column Label	Ast required	Ast provided	Deficit (Ar)	1.33 Ar
A1-storey 1	561	1030	0	0
A1-storey 2	1044	1030	14	18.62
B1- storey 1	994	1030	0	0
B1- storey 2	1097	1030	67	89.11
C1-storey 1	1028	1030	0	0
A2- storey1	1119	1030	89	118.37
A2- storey2	1228	1030	198	263.34
B2- storey1	660	1030	0	0
B2- storey2	1970	1030	940	1250.2
C2- storey1	609	1030	0	0
C2- storey2	1023	1030	0	0
A3- storey1	572	1030	0	0
A3- storey2	1054	1030	24	31.92
B3- storey1	1063	1030	33	43.89
B3- storey2	2035	1030	1005	1336.65
C3- storey1	589	1030	0	0
C3- storey2	1023	1030	0	0

From the above table 13, we can conclude that the reinforcement used for columns are not sufficient in longitudinal direction. To know the seismic performance level of existing building, we perform nonlinear static analysis i.e. pushover analysis.

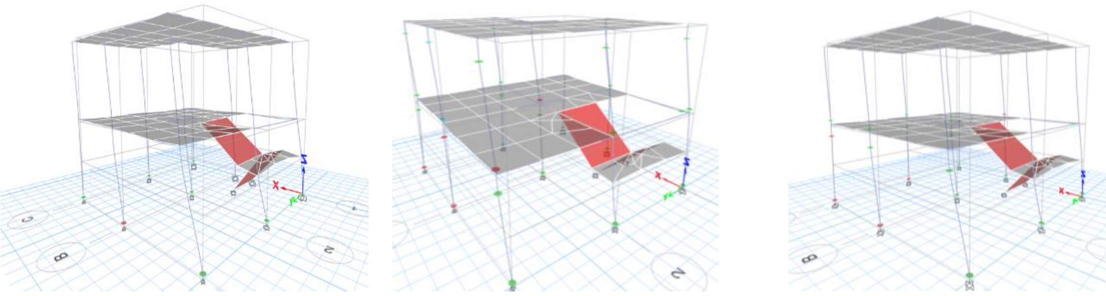
2. Pushover analysis of existing structure

Table 14 Base shear vs Displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	78	0	0	0	0	78	0	0	0	78
1	18	250	76	2	0	0	0	78	0	0	0	78
2	43	467	56	22	0	0	0	71	6	0	1	78
3	69	594	42	36	0	0	0	61	15	0	2	78
4	97	678	33	45	0	0	0	47	27	1	3	78
5	124	734	32	46	0	0	0	46	27	1	4	78
6	150	793	30	48	0	0	0	46	27	1	4	78
7	160	815	30	47	1	0	0	45	25	4	4	78
8	160	760	30	47	0	1	0	45	24	5	4	78
9	165	778	30	46	1	1	0	45	24	4	5	78
10	147	522	30	46	1	1	0	45	24	4	5	78

The seismic weight of the building was 957KN. The performance point was not obtained for the existing structure and the maximum base shear obtained was only 778 KN, which means the building will collapse before reaching its capacity.

The hinges started to form from step 2, we can learn the behavior of individual structure from formation of hinge in each step.



(a) (b) (c)

Figure 34: Hinges formation at Step 2(a) Step 4(b) and step 10 (b)

A3, B3, C3 and B2 columns were found to be critical. Retrofitting is required to increase the global as well as local capacity of the structure.

4.4.4.2 Case II: Considering Retrofitted Building

Trial 1: Steel jacketing on the critical columns

After the steel encasement with L75X75X8, the weight of building was 989KN. The performance point had base shear of 1111 KN, maximum displacement of 52mm. The maximum drift of structure was 0.012 due to which it lies in Life safety performance level.

Table 15 Base shear vs Displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	78	0	0	0	0	78	0	0	0	78
1	10	291	77	1	0	0	0	78	0	0	0	78
2	38	934	49	29	0	0	0	77	1	0	0	78
3	73	1367	42	36	0	0	0	59	18	0	1	78
4	105	1732	38	40	0	0	0	54	23	0	1	78
5	123	1927	37	39	2	0	0	53	21	3	1	78
6	110	1527	37	39	1	1	0	53	21	3	1	78

From the table 15, we can find that performance point lies between step 2 and 3 and only one hinge is in collapse prevention which means local failure are not prevalent.

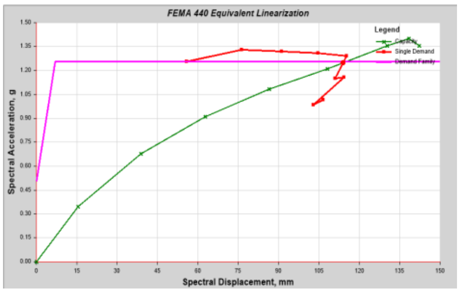


Figure 35Demand Capacity Curve
Before Retrofitting

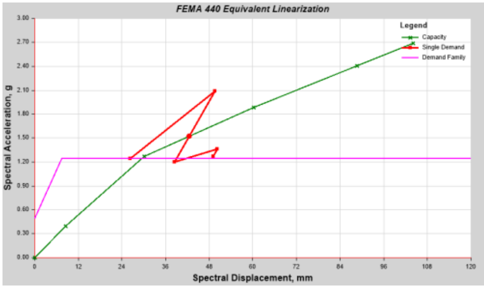
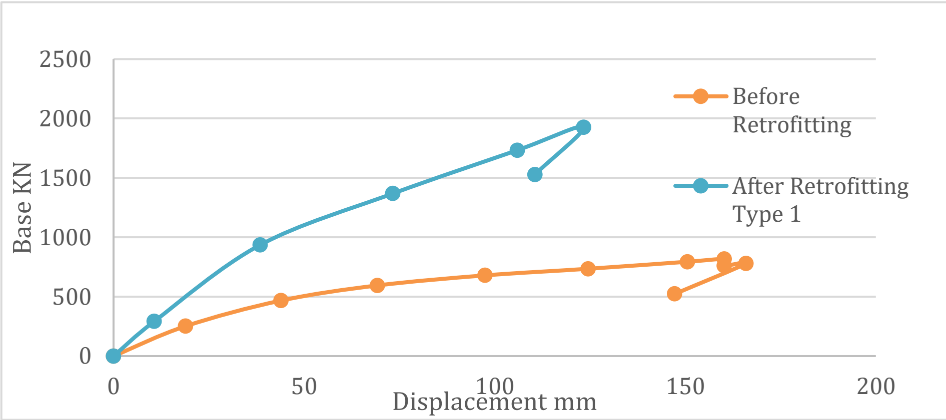


Figure 36 Demand Capacity Curve
After Retrofitting



Graph 4 Pushover curve before and after retrofitting for typology D

The above graph shows the pushover curve of the building before and after retrofitting from which it can be interpreted that the performance (capacity) of building has improved after the application of assigned retrofitting strategy.

4.4.5 **Typology E**

Predefined Specifications:

Building Height: 27’

Ground Floor Height: 9’4”

Floor Height: 9’4”

Concrete Grade: M15

Beam Size: 9”x13”

Slab Thickness: 100mm

Length: 28’9”

No. of Bays in Length:2

Breadth: 20’8”

No. of Bays in Breadth: 2

Plinth Area: 594.26

No. of Storey: 2 + SC

Column Size: 9”x12”

MRT Code: 201:1994

MRT Satisfied: Yes

Column Reinforcements:

4-16+4-12Gf and 1st Floor)

4-16+2-12(2nd Floor)

Stirrups: 8 dia @4"c/c and 6"c/c

Beam Reinforcements:

Top: 3-12(T) +2-16 Ex

Bottom: 3-16

Stirrups: 8 dia @ 4"c/c and 6"c/c

Slab Reinforcements: 8 dia @6"c/c

Figure 37: Plan of Typology E

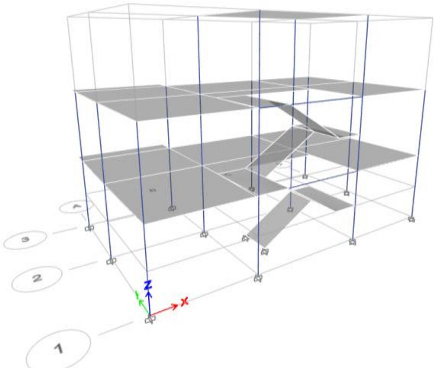


Figure 38: 3D Model of Typology E

4.4.5.1 Case I: Considering existing building

1. Seismic analysis considering existing building as per NBC 105:2020

It was found that the maximum drift and deflection was 0.0025 and 17.37 mm respectively, which are within the permissible range of 0.00625 and 53.12mm. The sizes and reinforcement for all members except for longitudinal rebars of columns were sufficient.

The details of existing and required reinforcement for columns is shown in following table 16 where Ast is required area of reinforcement while Ar is deficit area of reinforcement in that section.

Table 16 Detailing for Column Reinforcements (mm^2)

Column Label	Ast required	Ast provided	Deficit (Ar)	1.33 Ar
A1-storey 1	617	1257	0	0
A1-storey 2	1032	1257	0	0
B1- storey 1	1335	1257	0	0
B1- storey 2	1345	1030	315	418.95
B1 Storey 3	1093	1030	63	83.79
C1- storey 1	1570	1257	313	416.29
C1- storey 2	1253	1030	223	296.59
C1 Storey 3	1152	1030	122	162.26
D1-storey 1	621	1257	0	0
D1-storey 2	1030	1257	0	0

A2- storey1	622	1257	0	0
A2- storey2	1241	1030	211	280.63
B2- storey1	877	1257	0	0
B2- storey2	979	1030	0	0
B2- storey3	1057	1030	27	35.91
C2- storey1	894	1257	0	0
C2- storey2	1012	1030	0	0
C2- storey3	1085	1030	55	73.15
D2-storey 1	611	1257	0	0
D2-storey 2	1268	1257	11	14.63
A3- storey1	784	1257	0	0
A3- storey2	1045	1257	0	0
B3- storey1	1102	1257	0	0
B3- storey2	2045	1030	1015	1349.95
C3- storey1	1091	1257	0	0
C3- storey2	2041	1030	1011	1344.63
D3- storey1	760	1257	0	0
D3- storey2	1043	1257	0	0

2. Pushover analysis of existing structure

Here, most of the columns were not sufficient in reinforcement. After the pushover analysis, it was found that performance point was not reached.

Table 17 Base shear vs Displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	132	0	0	0	0	132	0	0	0	132
1	20	341	128	4	0	0	0	132	0	0	0	132
2	51	651	101	31	0	0	0	115	15	0	2	132
3	82	899	93	39	0	0	0	109	16	1	6	132
4	93	985	80	51	1	0	0	108	14	2	8	132
5	93	968	78	53	0	1	0	108	12	4	8	132
6	96	993	77	53	1	1	0	108	12	4	8	132
7	92	919	76	52	1	3	0	108	11	4	9	132

At step2, two hinges are formed, while at step 7, 9 hinges are formed. From the figures, we can observe the location of such plastic hinges.

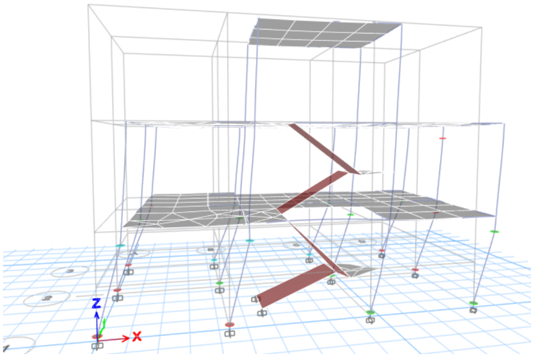


Figure 39: Location of Hinges at Step 7 (Last Step)

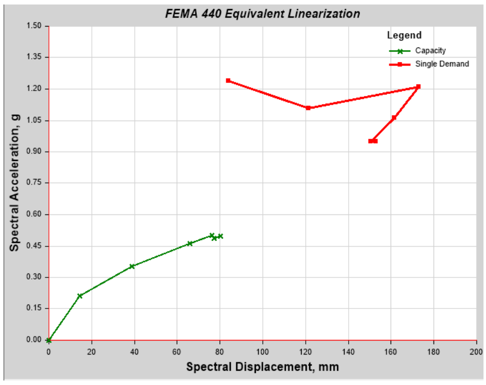


Figure 40: Demand Capacity Curve

Most of the hinges were formed in ground floor column. Interventions to increase local and global capacity of building is required.

4.4.5.2 Case II: Considering Retrofitted Building

Trial 1: Addition of shear wall

Shear wall of 230 mm thickness with 12 mm rebar at 150mm spacing was added along grid A-A and D-D. First the analysis was done by equivalent static method (NBC105:2020) to check the subsequent reinforcement demands in columns. It was observed that the requirement decreased drastically.

Pushover analysis of retrofitted building:

The new weight of building obtained was 1779 KN. The performance point had base shear of 2026.95KN, maximum displacement of 95.64mm. The maximum drift of structure was 0.012 due to which it lies in Life safety performance level.

Table 18 Base Shear vs Displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	140	0	0	0	0	140	0	0	0	140
1	19	505	138	2	0	0	0	140	0	0	0	140
2	51	1267	99	41	0	0	0	137	0	0	3	140
3	82	1835	82	58	0	0	0	117	18	0	5	140
4	113	2286	71	69	0	0	0	105	29	0	6	140
5	146	2646	63	77	0	0	0	97	37	0	6	140
6	169	2870	60	79	1	0	0	94	38	2	6	140
7	155	2482	59	80	1	0	0	94	38	1	7	140

The performance point lies between step 3 and 4. The formation of hinges on those steps are shown in following figures.

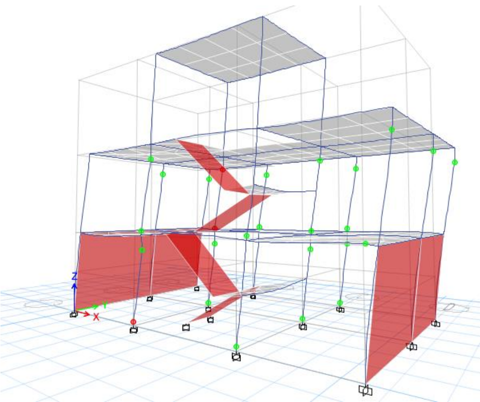


Figure 41 Hinge State in Step 3

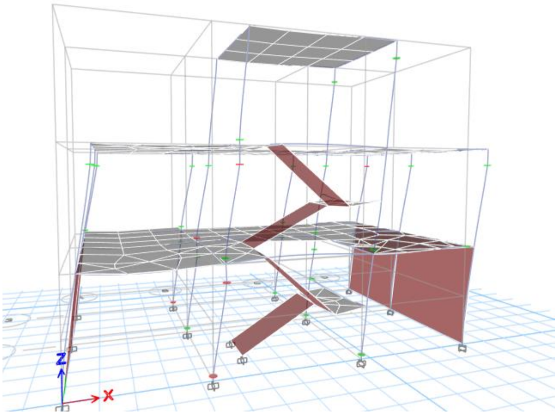


Figure 42 Hinge State in Step 7

<u>Trial 2: Concrete and steel jacketing of columns</u>

Here, columns of grid A-A and D-D are jacketed with concrete and reinforcements in 1st storey while all other columns are steel jackets for all storeys. 100mm concrete thickness with 8 numbers of 16mm and 4 numbers of 12 mm rebar was used with 8mm stirrups for concrete jacketing while L 50X50X5 was used for steel jacketing.

The table 19 shows the formation of hinges in the corresponding base shear and displacements.

Table 19 Base Shear vs Monitored Displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	136	0	0	0	0	136	0	0	0	136
1	15	595	135	1	0	0	0	136	0	0	0	136
2	48	1679	89	47	0	0	0	136	0	0	0	136
3	79	2343	80	56	0	0	0	122	13	0	1	136
4	111	2971	79	57	0	0	0	118	15	0	3	136
5	143	3601	74	62	0	0	0	111	21	0	4	136
6	145	3646	74	61	1	0	0	111	20	1	4	136
7	145	3625	74	61	0	1	0	111	20	1	4	136
8	157	3856	72	61	2	1	0	107	24	1	4	136
9	134	2898	72	61	1	2	0	107	24	1	4	136

The new weight after jacketing was 1676KN. The base shear, spectral acceleration and maximum displacement at performance point was 1998KN, 1.33g and 63.4mm respectively. The maximum drift was 0.01. The building lies in immediate occupancy performance level.

It can be seen from table 19, the performance point lies between step 2 and 3 and the corresponding base shear at formation of first CP hinge is 2343KN.

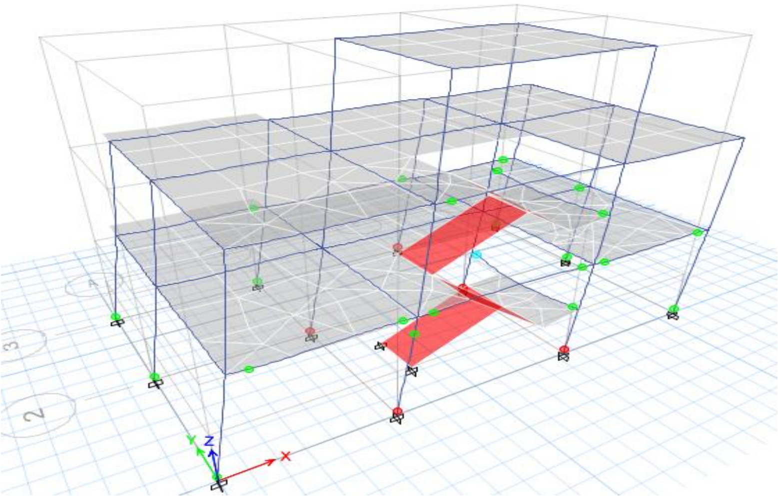


Figure 43 Hinge state in Step 8

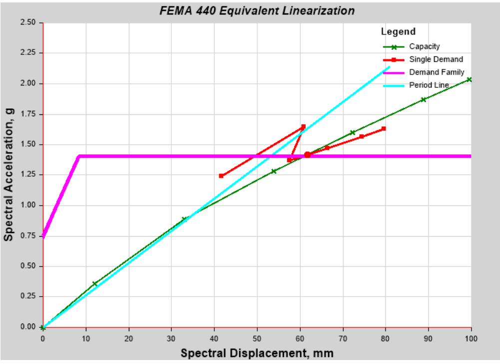


Figure 44 Demand Capacity Curve After AfterTrial 1 Retrofitting

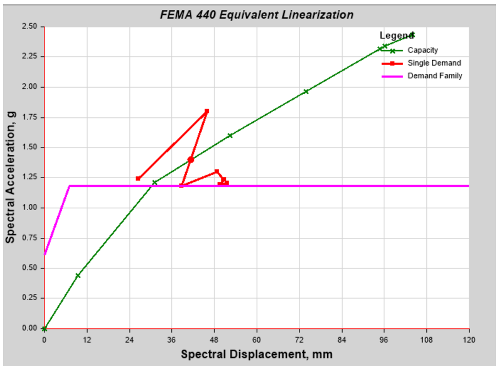
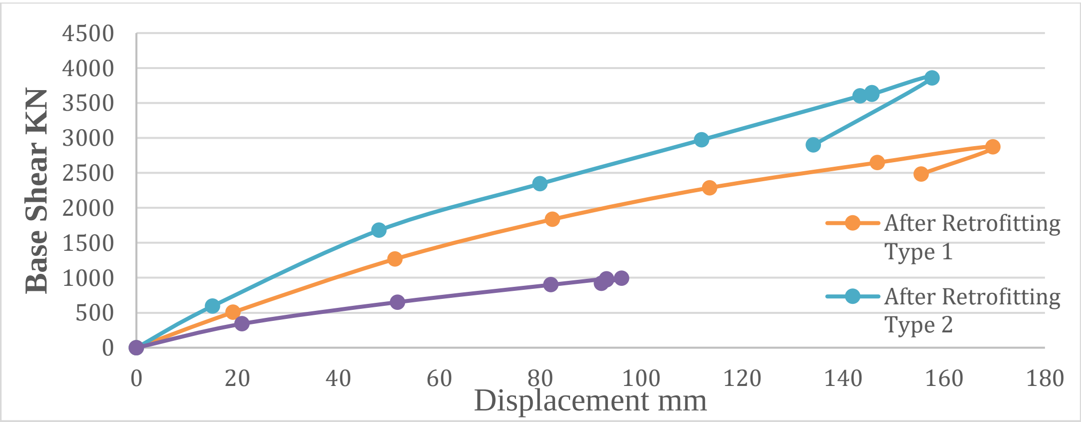


Figure 45 Demand Capacity Curve Trial 2 Retrofitting



Graph 5 Pushover curve before and after retrofitting for typology E

Both the trials has shown a desirable performance. From figure 44 and figure 45 it can be observed that the performance point in trial 2 has yielded comparatively better result as the performance point occurs in Immediate Occupancy level.

From graph 5, pushover curve of the building before and after retrofitting, it can be interpreted that the performance (capacity) of building has improved after the application of assigned retrofitting strategy.

4.4.6 **Typology F**

Predefined specifications:

Building Height: 26’3”	Plinth Area: 497.77
No. of Storey: 2 + SC	Floor Height: 8’9”
Column Size: 9”x12”	Concrete Grade: M15
Beam Size: 9”x13”	MRT Code: 201:1994
Slab Thickness: 100mm	MRT Satisfied: Yes
Length: 21’11”	Column Reinforcements: 4-16 (All)
No. of Bays in Length: 2	Beam Reinforcements:
Breadth: 21’8”	Top & Bottom: 2-16(T) +1-16 Ex
No. of Bays in Breadth: 2	Stirrups: 7 dia @ 4"c/c and 6"c/c

Slab Reinforcements: Main:8 dia @6"c/c and 7 dia(distribution)

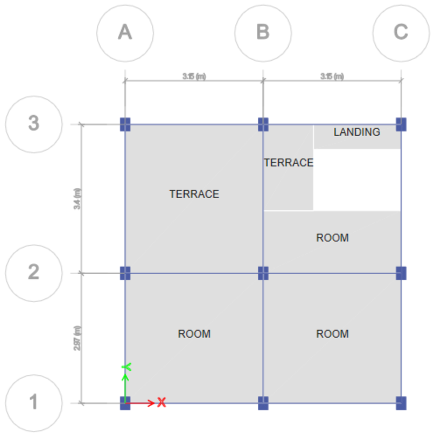


Figure 46: Plan of Typology F

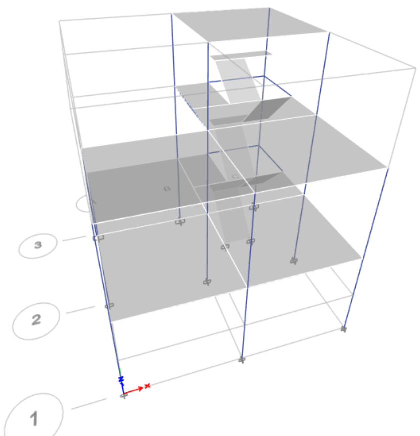


Figure 47: 3D model of Typology F

4.4.6.1 Case I: Considering existing building

1. Seismic analysis considering existing building as per NBC 105:2020

It was found that the maximum drift and deflection was 0.006 and 48 mm respectively, which are within the permissible range of 0.00625 and 50mm. The sizes and reinforcement for all members except for longitudinal rebars of columns were sufficient.

The details of existing and required reinforcement for columns is shown in table 19.

Table 20 Detailing for Column Reinforcements

Column Label	Ast required	Ast provided	Deficit (Ar)	1.33 Ar
A1-storey 1	561	804	0	0
A1-storey 2	1023	804	219	291.27
B1- storey 1	858	804	54	71.82
B1- storey 2	2038	804	1234	1641.22
C1-storey 1	561	804	0	0
C1-storey 2	1024	804	220	292.6
A2- storey1	561	804	0	0
A2- storey2	1031	804	227	301.91
B2- storey1	622	804	0	0
B2- storey2	788	804	0	0
B2- storey3	1088	804	284	377.72
C2- storey1	576	804	0	0
C2- storey2	622	804	0	0
C2- storey3	1008	804	204	271.32
A3- storey1	1025	804	221	293.93
B3- storey1	850	804	46	61.18
B3- storey2	980	804	176	234.08
B3- storey3	1007	804	203	269.99
C1- storey1	613	804	0	0
C1- storey2	561	804	0	0
C1- storey3	1009	804	205	272.65

2. Pushover analysis of existing structure

Here, most of the columns were not sufficient in reinforcement. After the pushover analysis, it was found that at performance point base shear of 707 KN was not equal or greater than the seismic weight of 1033 KN. The following table shows the base force and formation of hinges for various displacement.

Table 21 Base Shear vs Displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	94	0	0	0	0	94	0	0	0	94
1	19	257	93	1	0	0	0	94	0	0	0	94

2	50	515	66	28	0	0	0	84	10	0	0	94
3	85	703	54	40	0	0	0	72	19	0	3	94
4	118	841	46	46	2	0	0	63	22	1	8	94
5	123	853	46	45	3	0	0	63	21	2	8	94
6	124	853	46	45	3	0	0	63	21	1	9	94
7	125	853	46	44	4	0	0	63	21	1	9	94
8	126	850	46	44	4	0	0	63	20	2	9	94
9	126	848	46	43	5	0	0	63	19	3	9	94
10	129	820	45	42	7	0	0	63	17	5	9	94
11	129	817	45	40	9	0	0	63	17	5	9	94
12	126	644	45	39	8	2	0	63	16	1	14	94

4.4.6.2 Case II: Considering Retrofitted Building

Trial 1: RC jacketing in ground floor columns and rest steel jacket

100mm concrete thickness with 8 numbers of 16mm rebar was used with 8mm stirrups for concrete jacketing for all columns in ground floor while L 50X50X5 was used for steel jacketing for columns above ground floor. The new seismic weight of building was 1152 KN. After the pushover analysis of retrofitted building, it was found that the base shear was 1196 KN with displacement of 43mm and spectral acceleration of 1.5g. The obtained drift was 0.005, which indicated that the performance level of the retrofitted structure is under immediate occupancy.

Table 22 Base Shear vs Displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	94	0	0	0	0	94	0	0	0	94
1	12	464	93	1	0	0	0	94	0	0	0	94
2	43	1099	58	36	0	0	0	88	6	0	0	94
3	75	1675	50	44	0	0	0	80	13	0	1	94
4	115	2361	45	49	0	0	0	63	30	0	1	94
5	146	2857	42	51	1	0	0	52	38	3	1	94
6	178	3372	38	53	2	0	1	48	39	5	2	94
7	187	3477	37	52	4	0	1	47	38	5	4	94
8	192	3520	36	50	7	0	1	47	37	6	4	94
9	193	3466	36	50	7	0	1	47	37	5	5	94
10	194	3350	36	50	7	0	1	46	38	5	5	94
11	195	3317	36	50	6	1	1	46	38	5	5	94

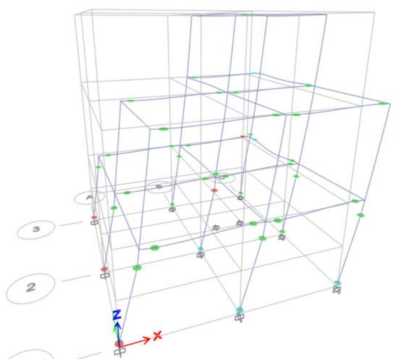


Figure 48 Hinge state at Final Step

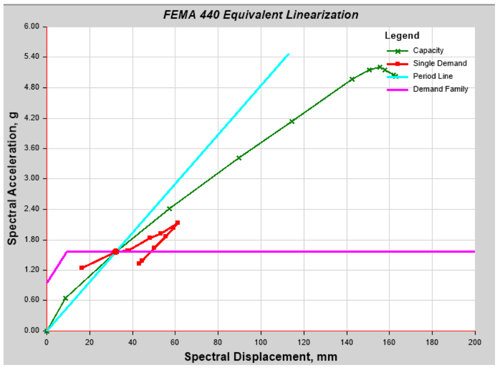
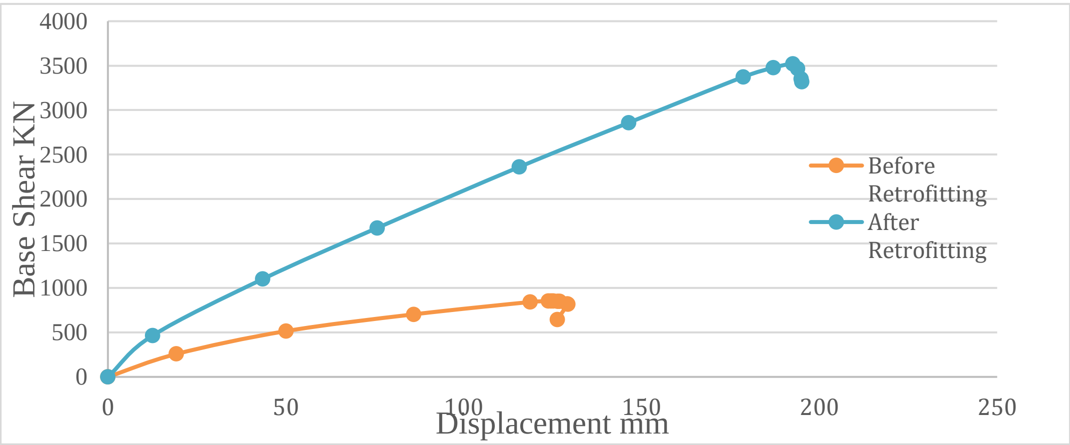


Figure 49 Demand Capacity Curve after Retrofitting

Here at performance point only one hinge at CP is formed which indicates good global structural stability.



Graph 6 Pushover curve before and after retrofitting for typology F

4.4.7 Typology G

Predefined Specifications:

Building Height: 35’5”	Plinth Area: 967.5
No. of Storey: 3	Floor Height: 9’4”
Column Size: 9”x12”	Concrete Grade: M15
Beam Size: 9”x13”	MRT Code: 201:1994
Slab Thickness: 100mm	MRT Satisfied: Yes
Length: 32’3”	Column Reinforcements:
No. of Bays in Length: 3	c3-(Corners) Gf: 4-16+4-12/ 4-16+2-12
Breadth: 30’	c2 (face) Gf: 4-16+4-12 ff&above: 4-16+2-12
No. of Bays in Breadth: 3	c1 (interior) Gf: 4-16+2-12 ff&above: 4-16+2-12

Stirups: 8 dia @3"c/c and 6"c/c

Beam Reinforcements: 2+16(top), 2-16(bottom)

Slab Reinforcements: Main: 8 dia @6"c/c & 7 dia

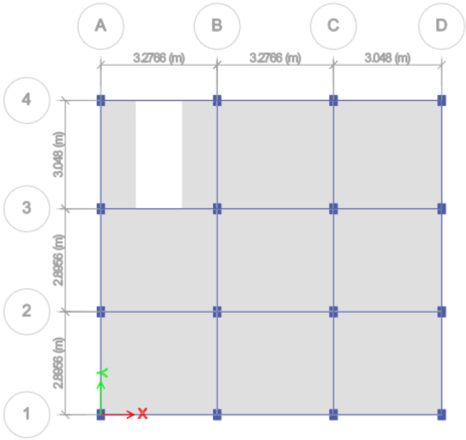


Figure 50: Plan of Building

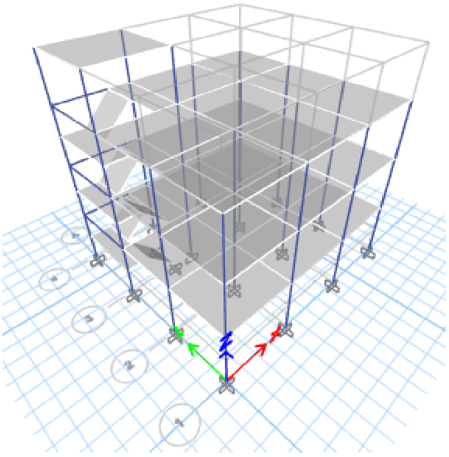


Figure 51: 3D Model of Structure

4.4.7.1 Case I: Considering existing building

1. Seismic analysis considering existing building as per NBC 105:2020

It was found that the maximum drift and deflection was 0.004 and 29 mm respectively, which are within the permissible range of 0.00625 and 66.68mm. The sizes and reinforcement for all members except for longitudinal rebars of columns were sufficient. The details of existing and required reinforcement for columns is shown in table 23 below.

Table 23 Detailing for Column Reinforcements

Column Label	Ast required	Ast provided	Deficit (Ar)	1.33 Ar
A1-storey 1	1217	1257	0	0
A1-storey 2	984	1257	0	0
A1-storey 3	1125	1257	0	0
B1- storey 1	1488	1257	231	307.23
B1- storey 2	1479	1257	222	295.26
B1- storey 3	2348	1257	1091	1451.03
C1-storey 1	1587	1257	330	438.9
C1-storey 2	1674	1257	417	554.61
C1-storey 3	2370	1257	1113	1480.29
D1- storey1	1304	1257	47	62.51
D1- storey2	1048	1257	0	0
D1- storey3	1096	1257	0	0
A2- storey1	1158	1257	0	0
A2- storey2	858	1257	0	0

A2- storey3	1356	1257	99	0
B2- storey1	1233	1257	0	0
B2- storey2	1112	1257	0	0
B2- storey3	2193	1257	936	1244.88
C2- storey1	1250	1257	0	0
C2- storey2	1111	1257	0	0
C2- storey3	2256	1257	999	0
D2- storey1	1247	1257	0	0
D2- storey2	1407	1257	150	0
D2- storey3	1366	1257	109	144.97
A3- storey1	1477	1257	220	292.6
A3- storey2	1007	1257	0	0
A3- storey3	646	1257	0	0
A3- storey3 SC	1066	1257		0
B3- storey1	2032	1257	775	1030.75
B3- storey2	1977	1257	720	957.6
B3- storey3	1345	1257	88	117.04
B3- storey4	1105	1257	0	0
C3- storey1	1153	1257	0	0
C3- storey2	1078	1257	0	0
C3- storey3	2261	1257	1004	0
D3-Storey 1	1142	1257	0	0
D3-Storey 2	1032	1257	0	0
D3-Storey 3	1340	1257	83	0
A4- storey1	2429	1257	1172	1558.76
A4- storey2	950	1257	0	0
A4- storey3	612	1257	0	0
A4- storey4	1102	1257	0	0
B4- storey1	2250	1257	0	0
B4- storey2	2088	1257	831	1105.23
B4- storey3	1630	1257	373	496.09
B4- storey4	1189	1257	0	0
C4- storey1	1294	1257	37	49.21
C4- storey2	1421	1257	164	218.12
C4- storey3	2384	1257	1127	1498.91
D1 Storey 1	1172	1257	0	0
D1 Storey 2	1024	1257	0	0

2. Pushover analysis of existing structure

Here, most of the columns were not sufficient in reinforcement. The seismic weight of the building was 2764KN.

After the pushover analysis, it was found that performance point was not reached i.e. structure collapses before reaching target base shear capacity. The following table 24 shows the base force and formation of hinges for various displacement.

Table 24 Base Shear vs Displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	256	0	0	0	0	256	0	0	0	256
1	39	582	253	3	0	0	0	256	0	0	0	256
2	84	962	178	78	0	0	0	224	29	0	3	256
3	123	1209	160	95	1	0	0	203	49	0	4	256
4	123	1185	160	95	0	1	0	203	47	2	4	256
5	125	1198	160	94	1	1	0	203	47	2	4	256

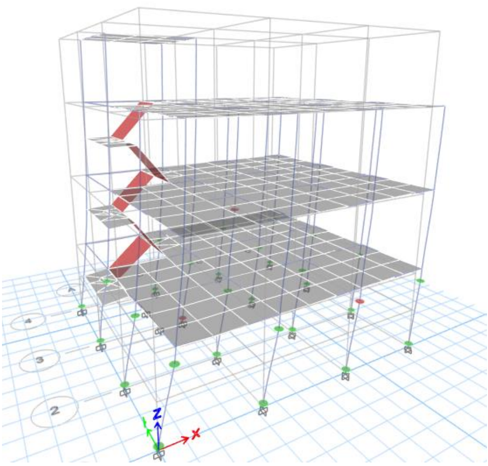


Figure 52: Hinges Formation at Step 2

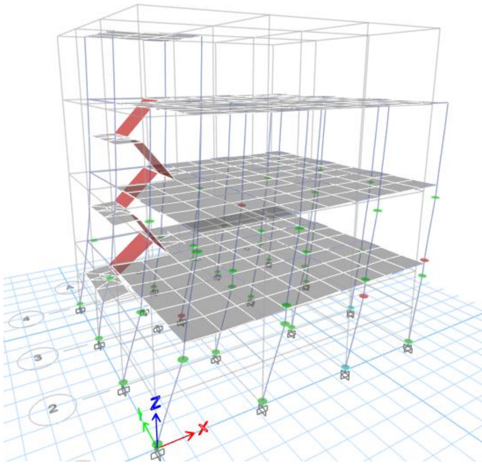


Figure 53: Hinges Formation at Step 5

4.4.7.2 Case II: Considering Retrofitted Building

Trial 1: Shearwall addition to the ground floor

To improve the global capacity of building, shear wall of thickness 230mm with 12 mm rebar at 150 mm spacing was added on all four sides of building. The performance point was not sufficient to the demand base shear of 3326KN. The addition of shearwall decreased the required reinforcement demands in the columns though. Thus, shearwall and steel jacketing was proposed for next iteration.

Trial 2: Concrete and steel jacketing to all the columns above

Steel jacketing was provided with angle L50*50*5 and mild steel plate to all columns above ground floor while all columns in ground floors were jacketed with reinforced concrete of thickness 100mm with 4 numbers of 16mm and 4 numbers of 12 mm rebar with 8 mm stirrups. The new weight of building was 3021 KN. It was found that the base shear was 3633 KN with displacement of 164mm. The obtained drift was 0.015, which indicated that the performance level of the retrofitted structure is under Damage Control.

Table 25 Base Shear vs Displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	256	0	0	0	0	256	0	0	0	256
1	22	783	255	1	0	0	0	256	0	0	0	256
2	42	1413	201	55	0	0	0	255	0	0	1	256
3	116	2802	167	89	0	0	0	223	24	0	9	256
4	169	3723	153	103	0	0	0	202	45	0	9	256
5	204	4315	143	112	1	0	0	180	63	0	13	256
6	200	4156	143	112	1	0	0	180	62	1	13	256

<u>Trial 3: Shear wall addition with steel jacketing</u>

The shearwall was added on periphery of thickness 230mm with 12 mm rebar at 150 mm spacing at ground floor while columns from first floors were steel jacketed with L50X50X5. The base shear at performance point was 3149KNwhich is more than the seismic weight of 3118KN. The corresponding deflection was 45.42mm and drift was 0.004. The performance of building lied in Immediate Occupancy level.

Table 26 Base Shear vs Displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	309	0	0	0	0	309	0	0	0	309
1	17	1312	308	1	0	0	0	309	0	0	0	309
2	64	4396	241	68	0	0	0	309	0	0	0	309
3	116	6293	220	89	0	0	0	273	36	0	0	309
4	185	8065	216	93	0	0	0	240	64	0	5	309
5	221	8989	214	94	1	0	0	235	62	0	12	309
6	221	8792	214	94	0	1	0	235	62	0	12	309
7	226	8946	214	93	1	1	0	234	63	0	12	309
8	223	8721	214	93	1	1	0	234	63	0	12	309

Here, no hinges beyond IO is formed. Hence, the addition of shear wall and steel jacketing to the existing building performs better than previous trials.

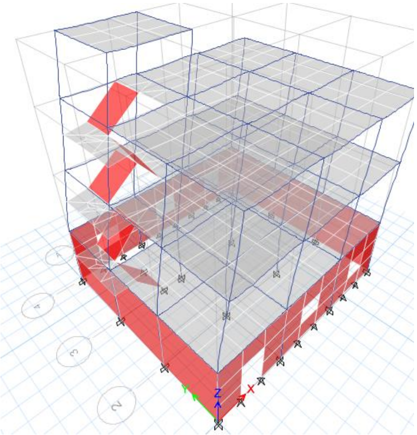


Figure 54: Step 2 (No Hinge Formation)

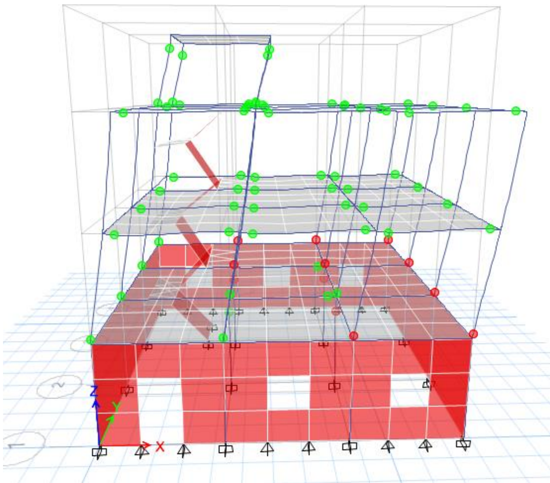
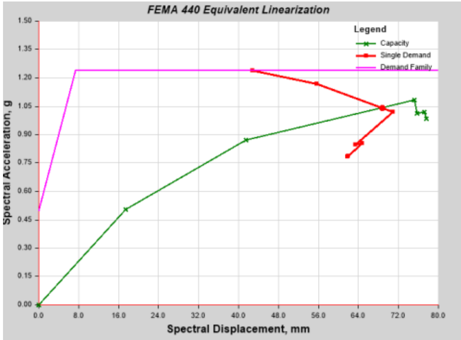
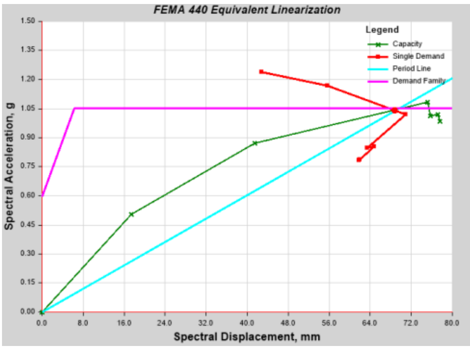


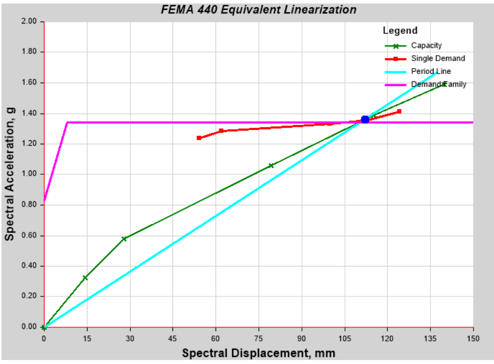
Figure 55: Hinge State in Step 8



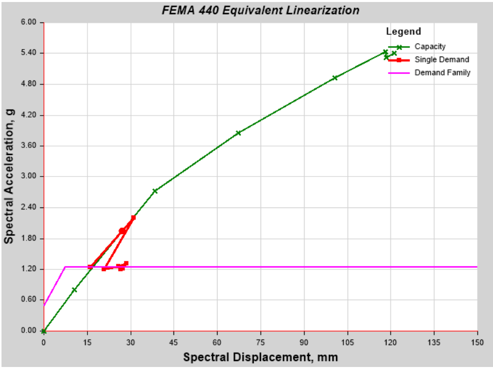
(a)



(b)

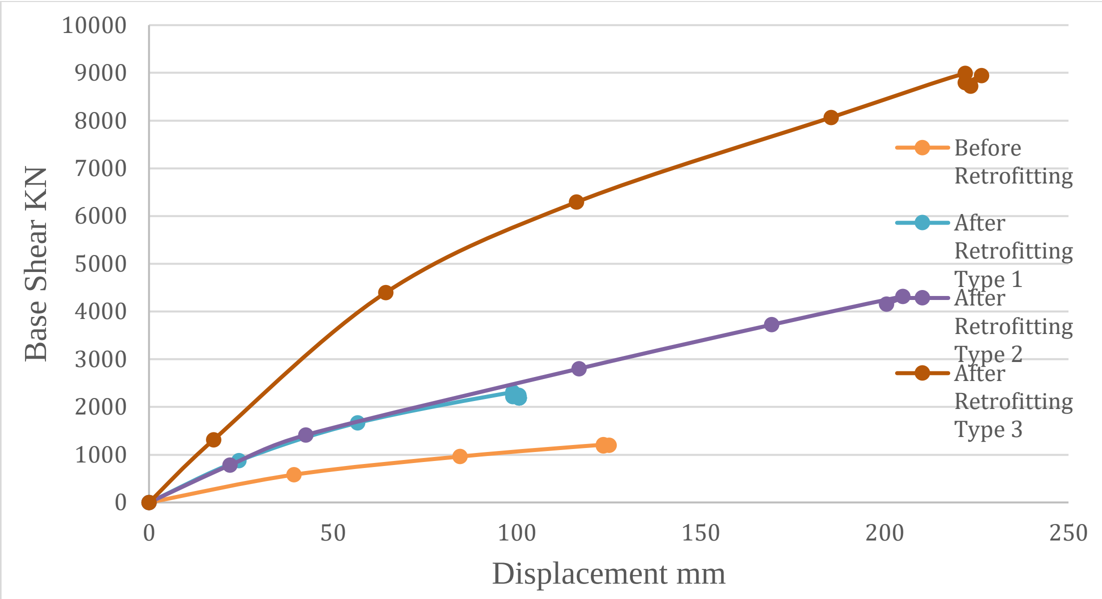


(c)



(d)

Figure 56 Demand Capacity Curve (a)Before retrofitting (b) After trial 1 retrofitting (c) After trial 2 retrofitting (d) After retrofitting trial 3



Graph 7 Pushover curve before and after retrofitting for typology G

From above graph 7 and performance point location, it can be concluded that trial 3 i.e. shear wall with steel jacketing provides better improvement on structural stability.

4.4.8 **Typology H**

Predefined Specifications:

S No: 21	Building Height: 27’10”
Plinth Area: 755.25	Ground Floor Height: 9’5”
No. of Storey: 3	Floor Height: 9’5”
Column Size: 9”x12”	Concrete Grade: M15
Beam Size: 9”x14”	MRT Code: 201:1994
Slab Thickness: 125mm	MRT Satisfied: Yes
Length: 26’6”	Column Reinforcements:
No. of Bays in Length:2	c1-(INTERIOR):Gf &ff:4-16+4-12 sf:4-12+2-12
Breadth: 28’6”	c2 (face) Gf&FF: 4-12+2-12
No. of Bays in Breadth: 3	c3 (CORNER) Gf: 4-16+2-12 ff&above: 4-16
	Stirups: 8 dia @3"c/c and 6"c/c
	Beam Reinforcements: T-2-16+1-16, B-2-16
	Slab Reinforcements: 10 mm & 8mm @6"c/c

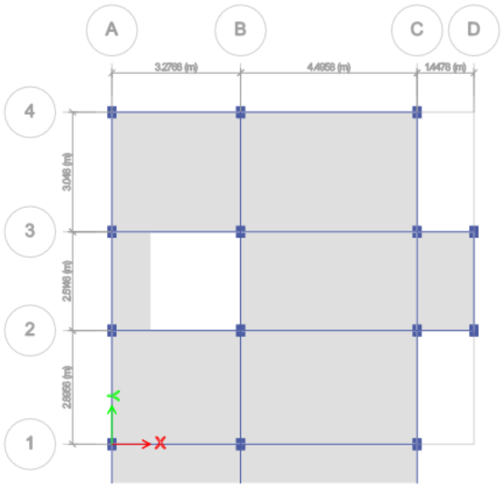


Figure 57: Plan of the Building

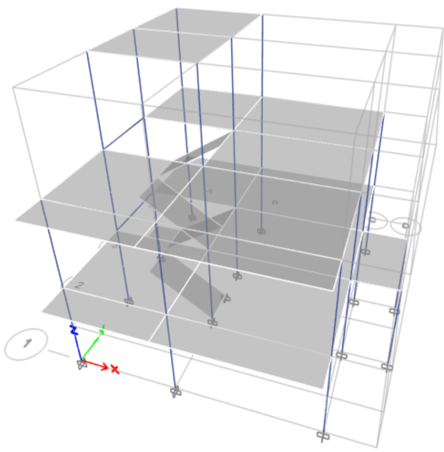


Figure 58: 3D Model of Building

4.4.8.1 Case I: Considering existing building

1. Seismic analysis considering existing building as per NBC 105:2020

The building showed maximum drift of 0.002, which is less than allowable 0.00625, and maximum displacement of 17.49mm and 9.68mm in EQx and EQy direction respectively. Maximum allowable displacement is 0.00625* height of the building which is 52mm in this case.

All concrete members passed in size but the area of reinforcement for column was not sufficient. Table 27 shows the details of existing and required reinforcement for columns.

Table 27 Detailing for Column Reinforcements

Column Label	Ast required	Ast provided	Deficit (Ar)	1.33 Ar
A1-storey 1	606	1030	0	0
A1-storey 2	1376	1030	346	460.18
B1- storey 1	779	1030	0	0
B1- storey 2	1634	1030	604	803.32
C1-storey 1	612	1030	0	0
C1-storey 2	1494	1030	464	617.12
A2- storey1	615	1030	0	0
A2- storey2	654	1030	0	0
A2- storey3	1186	1030	156	
B2- storey1	914	1257	0	0
B2- storey2	1096	1257	0	0
B2- storey3	1178	1257	0	0
C2- storey1	1010	1257	0	0
C2- storey2	1773	1257	516	686.28
D2- storey1	1241	1030	211	280.63

A3- storey1	1388	1030	358	476.14
A3- storey2	1122	1030	92	122.36
A3- storey3	1503	1030	473	629.09
B3- storey1	1126	1257	0	0
B3- storey2	1167	1257	0	0
B3- storey3	1492	1257	235	312.55
C3- storey1	957	1030	0	0
C3- storey2	1934	1030	904	1202.32
D3-Storey 1	1238	1030	208	276.64
A4- storey1	561	1030	0	0
A4- storey2	561	1030	0	0
A4- storey3	1176	1030	146	194.18
B4- storey1	1278	1030	248	329.84
B4- storey2	1346	1030	316	420.28
B4- storey3	1175	1030	145	192.85
C4- storey1	597	1030	0	0
C4- storey2	1232	1030	202	268.66

From the table 27, we can conclude that the reinforcement used for columns are not sufficient in longitudinal direction. To know the seismic performance level of existing building, we perform nonlinear static analysis i.e. pushover analysis.

2. Pushover analysis of existing structure

The seismic weight of the building was 1947KN. The performance point was not obtained for the existing structure and the maximum base shear obtained was only 1387KN (Table28) which means the building will collapse before reaching its capacity.

Table 28 Base Shear vs Displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	152	0	0	0	0	152	0	0	0	152
1	20	452	151	1	0	0	0	152	0	0	0	152
2	55	1075	115	37	0	0	0	139	11	0	2	152
3	76	1387	98	54	0	0	0	126	19	1	6	152

Here corner columns are at high risk of failure. Hence, to increase stiffness and global strength retrofitting by adding of shearwall and steel jacketing is considered.

4.4.8.2 Case II: Considering Retrofitted Building

<u>Trial 1: Addition of shearwall and steel jacketing</u>

Shear wall of thickness 230mm with 12 mm rebar at 150 mm spacing is added on the ground floor while upper storey columns are provided with steel jacketing. The performance of the retrofitted structure was obtained through pushover analysis. The new seismic weight of building was 2449 KN. After the pushover analysis of retrofitted building, it was found that the base shear was 5657 KN(Table 28) with displacement of 49mm before the formation of plastic hinge with drift of 0.006 and lies in immediate occupancy performance level.

Table 29 Base Shear vs Displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	203	0	0	0	0	203	0	0	0	203
1	12	1623	202	1	0	0	0	203	0	0	0	203
2	49	5657	165	38	0	0	0	203	0	0	0	203
3	85	8393	152	51	0	0	0	183	19	0	1	203
4	123	10973	146	57	0	0	0	171	30	0	2	203
5	123	11009	146	56	1	0	0	171	28	0	4	203
6	123	10725	146	56	0	1	0	171	28	0	4	203
7	124	10810	146	55	1	1	0	169	30	0	4	203
8	119	10189	146	55	1	1	0	169	30	0	4	203

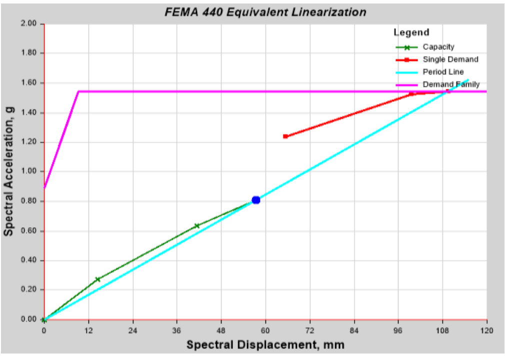


Figure 59: Demand Capacity Curve before retrofitting

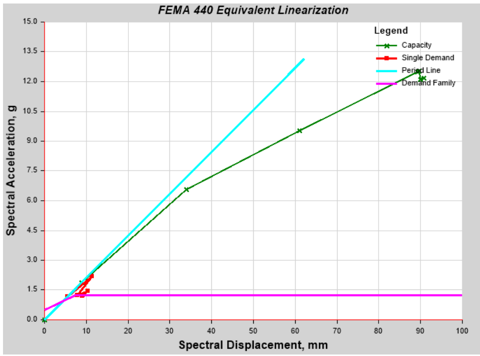
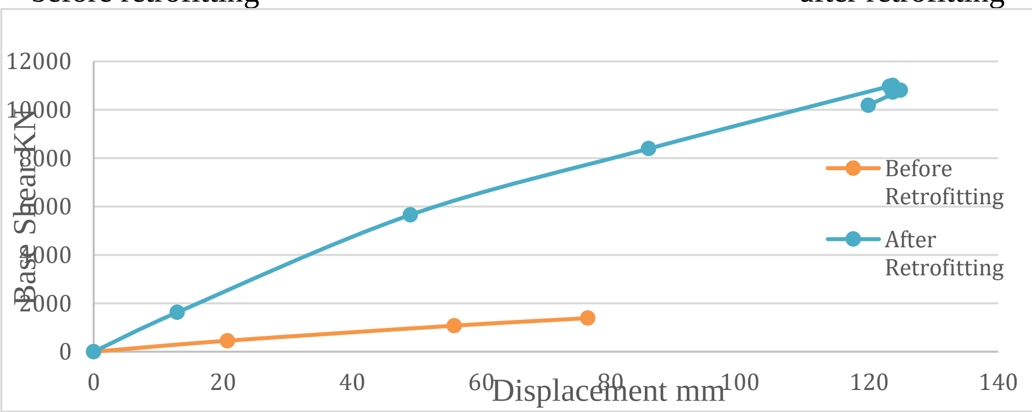


Figure 60: Demand Capacity Curve after retrofitting



Graph 8 Pushover curve before and after retrofitting for typology H

From above graph 8, it can be concluded the opted retrofitted strategy provides better improvement on structural stability.

4.4.9 **Typology I**

Predefined specifications:

Building Height: 18’8”

Ground Floor Height: 9’4”

Floor Height: 9’4”

Concrete Grade: M20

MRT Code: 205:2012

MRT Satisfied: Yes Length: 18’3”

No. of Bays in Length: 2

Breadth: 37’7”

Beam Reinforcements: 2-16+3-12

3-12+1-16 bottom (extra)

Plinth Area: 655

No. of Storey: 1+SC

Column Size: 12”x12”

Beam Size: 9”x14”

Slab Thickness: 125mm

Column Reinforcements:

4-16+4-12

No. of Bays in Breadth: 3

Slab Reinforcements: 8 dia @6"c/c

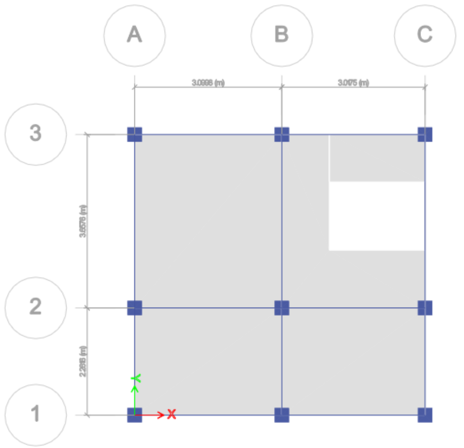


Figure 61: 3D Model of the Building

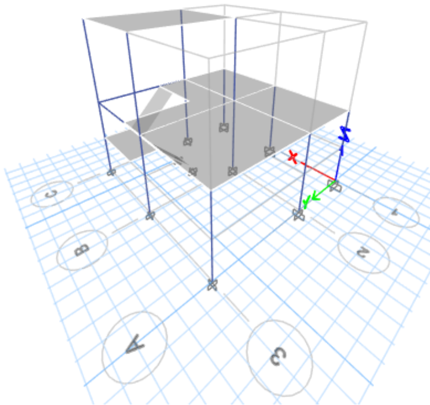


Figure 62: Plan of the Building

4.4.9.1 Case I: Considering existing building

1. Seismic analysis considering existing building as per NBC 105:2020

The building showed maximum drift of 0.001, which is less than allowable 0.00625, and maximum displacement of 3mm and 4mm in EQx and EQy direction respectively. Maximum allowable displacement is 0.00625* height of the building which is 35mm in this case.

All the concrete member were sufficient to resist the lateral load and reinforcement provided for all members were sufficient. Table 30 shows the detail of required and provided reinforcement in columns.

Table 30 Detailing for Column Reinforcements

Column label	Ast required	Ast provided
A1-storey 1	816	1257
B1- storey 1	978	1257
C1-storey 1	814	1257
A2- storey1	978	1257
B2- storey1	744	1257
B2- storey2	828	1257
C2- storey1	772	1257
C2- storey2	817	1257
A3- storey1	807	1257
B3- storey1	744	1257
B3- storey2	821	1257
C3- storey1	744	1257
C3- storey2	816	1257

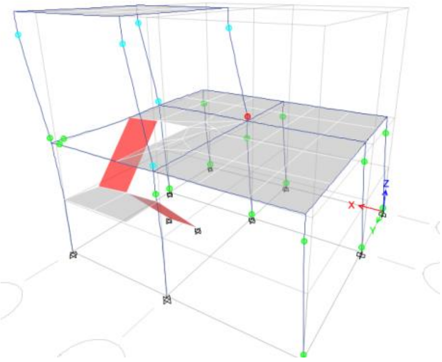
2. Pushover analysis of existing structure

From the pushover analysis it was found that base shear, displacement of structure was 779 KN, and 39 mm respectively which is higher than the demand weight of 649 KN. The drift was 0.007, which indicates it lies in performance level of immediate occupancy.

Table 31 Base shear vs Displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	60	0	0	0	0	60	0	0	0	60
1	9	280	59	1	0	0	0	60	0	0	0	60
2	34	744	33	27	0	0	0	57	3	0	0	60
3	61	959	18	42	0	0	0	45	15	0	0	60
4	90	1129	17	43	0	0	0	41	19	0	0	60
5	121	1312	15	45	0	0	0	37	23	0	0	60
6	142	1437	12	47	1	0	0	35	24	1	0	60
7	142	1437	12	47	1	0	0	35	24	1	0	60
8	142	1438	12	47	1	0	0	35	24	1	0	60
9	142	1253	12	47	0	1	0	35	24	0	1	60

10	144	1277	12	47	0	1	0	35	24	0	1	60
11	167	1418	12	46	1	1	0	35	18	6	1	60
12	150	957.	12	46	1	1	0	35	18	6	1	60



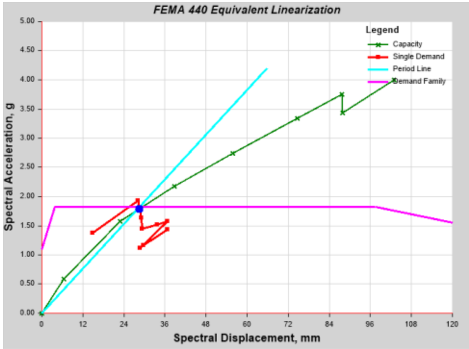


Figure 63 Demand capacity curve

Figure 64 Hinge state at final step

From the table 30, we can conclude that no collapse hinges are formed at performance point which indicates no local or global failure. Hence retrofitting is not required.

4.4.10 **Typology J**

Predefined specifications:

Building Height: 36’

Plinth Area: 429.02

Ground Floor Height: 9’4”

No. of Storey: 3

Floor Height: 9’4”

Column Size: 12”x12”

Concrete Grade: M20

Beam Size: 9”x14”

MRT Code: 205:2012

Slab Thickness: 125mm

MRT Satisfied: Yes

Length: 21’

Column Reinforcements: 4-16+4-12 & 8mm stirrups

No. of Bays in Length:2

Beam Reinforcements: 2-16+1-16(e) top, 2-16+1-12 bottom

Breadth: 20’5”

Slab Reinforcements: 8 dia @6"c/c

No. of Bays in Breadth: 2

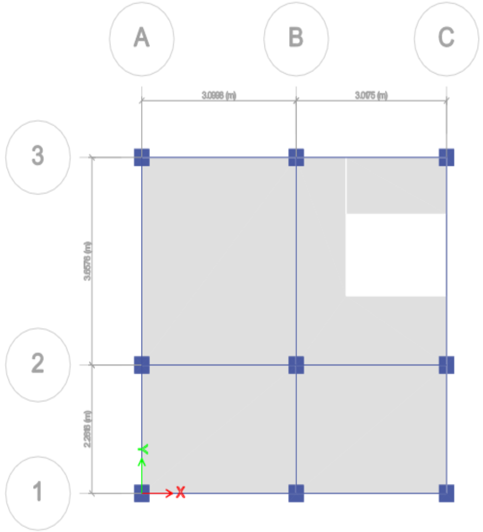


Figure 65: Plan of the Building

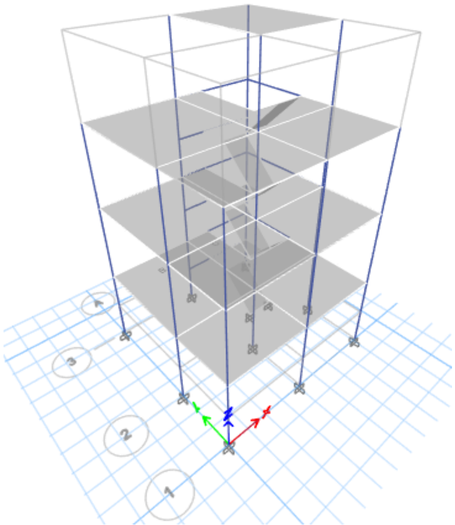


Figure 66: 3D Model of the Building

4.4.10.1 Case I: Considering existing building

1. Seismic analysis considering existing building as per NBC 105:2020

The building showed maximum drift of 0.001, which is less than allowable 0.00625, and maximum displacement of 12.7mm and 13.04mm in EQx and EQy direction respectively. Maximum allowable displacement is 0.00625* height of the building which is 68.5mm in this case. All the concrete member were sufficient to resist the lateral load and reinforcement provided for all members were sufficient except for A2 column. The following table 32 shows the detail of required and provided reinforcement in columns.

Table 32 Detailing for Column Reinforcements

Column Label	Ast required	Ast provided	Deficit (Ar)	1.33 Ar
A1-storey 1	782	1257	0	0
A1-storey 2	744	1257	0	0
A1-storey 3	824	1257	0	0
B1- storey 1	744	1257	0	0
B1- storey 2	770	1257	0	0
B1- storey 3	784	1257	0	0
B1- storey 4	816	1257	0	0
C1-storey 1	744	1257	0	0
C1-storey 2	770	1257	0	0
C1-storey 3	784	1257	0	0
C1-storey 4	816	1257	0	0
A2- storey1	744	1257	0	0
A2- storey2	765	1257	0	0

A2- storey3	1684	1257	427	567.91
B2- storey1	744	1257	0	0
B2- storey2	744	1257	0	0
B2- storey3	750	1257	0	0
B2- storey4	1052	1257	0	0
C2- storey1	783	1257	0	0
C2- storey2	789	1257	0	0
C2- storey3	774	1257	0	0
C2- storey4	1064	1257	0	0
A3- storey1	744	1257	0	0
A3- storey2	819	1257	0	0
A3- storey3	819	1257	0	0
B3- storey1	744	1257	0	0
B3- storey2	744	1257	0	0
B3- storey3	744	1257	0	0
B3- storey4	812	1257	0	0
C3- storey1	744	1257	0	0
C3- storey2	744	1257	0	0
C3- storey3	744	1257	0	0
C3-Storey 4	815	1257	0	0

2. Pushover analysis of existing structure

The seismic weight of the building was 1613KN. After the pushover analysis, it was found that at performance point base shear was only 1453KN. Thus, retrofit is suggested.

Table 33 Base shear vs displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	148	0	0	0	0	148	0	0	0	148
1	21	390	147	1	0	0	0	148	0	0	0	148
2	51	814	112	36	0	0	0	144	4	0	0	148
3	82	1111	89	59	0	0	0	135	13	0	0	148
4	118	1403	73	75	0	0	0	120	25	3	0	148
5	150	1640	63	85	0	0	0	111	32	0	5	148
6	165	1749	60	87	1	0	0	110	33	0	5	148
7	165	1749	60	87	1	0	0	110	33	0	5	148
8	165	1749	60	87	1	0	0	110	33	0	5	148
9	165	1749	60	87	1	0	0	110	33	0	5	148

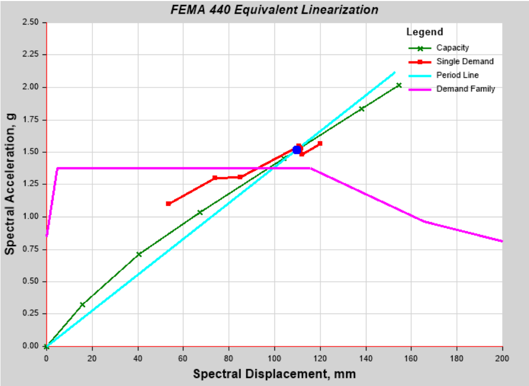
Trial 1: Addition of shear wall with RC jacketing

From the results of previous retrofitting techniques, here we add shear wall in ground floor with column jacketing for all columns. 230mm shearwall with 12 mm rebar at 150 mm spacing in ground floor and 100mm concrete thickness with 4 numbers of 16 and 2 numbers of 12mm rebar was used with 8 mm stirrups.

After the pushover analysis, it was found that the performance point base shear was 2289 KN, which is greater than demand base shear of 1905 KN with maximum displacement of 100 mm. The maximum drift is 0.01; the building lies in damage control level.

Table 34 Base shear vs displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0	0	148	0	0	0	0	148	0	0	0	148
1	18	542	147	1	0	0	0	148	0	0	0	148
2	68	1729	85	63	0	0	0	146	1	0	1	148
3	120	2641	65	83	0	0	0	125	22	0	1	148
4	128	2778	64	83	1	0	0	121	23	1	3	148
5	128	2778	64	83	1	0	0	121	23	1	3	148
6	128	2778	64	83	1	0	0	121	23	1	3	148



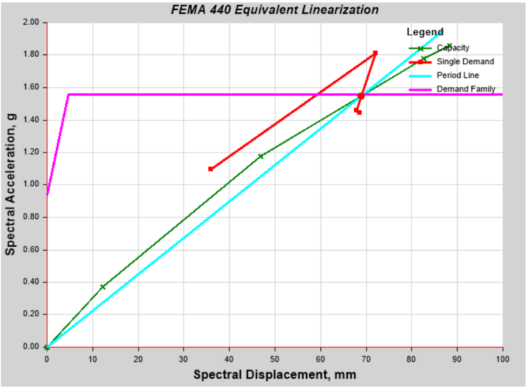
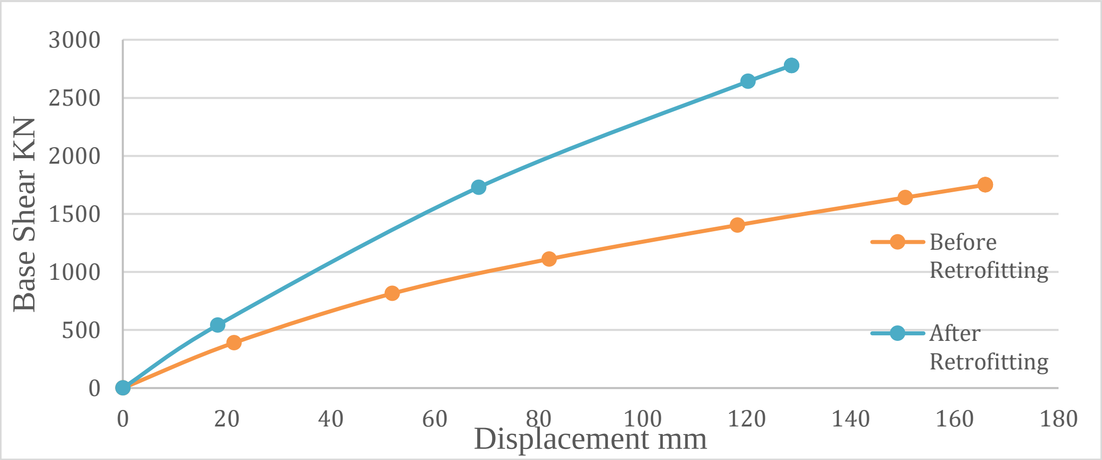


Figure 67 Demand capacity curve before after retrofitting

Figure 68 Demand capacity curve



Graph 9 Pushover curve before and after retrofitting for typology J

From above graph, it can be concluded the opted retrofitted strategy provides better improvement on structural stability.

CHAPTER 5: CONCLUSION AND RECOMMENDATION

5.1 Conclusion

This study was carried out to assess the seismic vulnerability of MRT compliant RCC buildings and to propose retrofitting techniques as per the typologies defined. Based on the results, it was found that all the typologies except Typology I with column size 12”X12” of one storey were vulnerable to future earthquakes. The major deficiency in all the studied structure was insufficient reinforcements in the columns. It was because of the fact that the earthquake force considerations taken in NBC 105:2020 was significantly higher than in NBC 105:1994 of which all the MRTs were based on. As per the pattern of formation of plastic hinges, most of the columns supporting slab adjoining staircase developed first hinges at collapse prevention point that means first local failures were more prevalent around staircase area.

Retrofitting strategies were applied based on typologies defined. The application and criteria for local retrofitting methods were based on IITK GSDMA guidelines and seismic retrofitting guidelines of buildings in Nepal. In this research, only three types of retrofitting techniques are studied. The details of retrofitting strategy summary can be found in Appendix 3.

Building typologies with 9”x9” column size were the most vulnerable structures to the earthquake. The reinforcement provision of only four number of 16 diameter did not provide sufficient capacity for the earthquake base shear demand. The sizes of beams, columns and slabs were sufficient to resist the vertical load but it was not the same for lateral loads. Concrete jacketing and steel jacketing applied for all building typology with 9”X9” for upto two storey was sufficient. For 3 storey building , RC jacketing for all columns was used as retrofitting technique.

For buildings with column size 9”X12” and upto 2 storey, application of steel and RC jacketing showed satisfactory improvement to performance level of the building i.e. from damage control to life safety. The addition of shearwall with jacketing in the ground floor showed the best improvement with these structures. Majority of the buildings belonged into this typology that indicates the construction prevalence of 9”X12” column buildings in the study area. Similarly, column reinforcement of 4 numbers of 16 mm diameter and 2 numbers of 12 mm was mostly used. For buildings with more than 3 storeys and 9”X12” column, a combination of addition of shearwall and steel or RC jacketing was best option for retrofitting in terms of improvement in structural stiffness and strength.

The building typology I with 12”X12” columns for upto 2 storey with less than 655 sqft plinth area did not require any intervention for retrofitting. It is because the minimum size, detailing and grade of concrete was increased in NBC 205:2012 than previous MRT guidelines. Its performance in its existing condition was in damage control level. For buildings above 2 storey, shear wall addition with RC jacketing improved performance level of building to damage control level since local strengthening only was not sufficient.

Hence, it is concluded that ready to use dimension and material properties defined by MRT guidelines are not sufficient for seismic loading as per new NBC code 105:2020 and are vulnerable to any major earthquakes. Hence, interventions for strengthening i.e. retrofitting is needed. Suggested retrofitting strategies from this research proved to have desired performance level. The techniques can be applied directly to the buildings falling under the specifications of defined typologies.

5.2 Recommendations

As Nepal is prone to major earthquakes and the state of existing buildings are not up to desired performance level which indicates failure or damage to the building in future earthquake. The solution of retrofitting these structures are mitigation measure to the disaster. The retrofitting guidelines suggested by this research can be implemented directly for suitable MRT compliant buildings and can be incorporated in National Strategic Action Plan (NSAP) of Nepal issued by Government of Nepal. Further, DRR policies can include vulnerability assessment and retrofitting of MRT compliant in local level as well for resilient development.

The following table 35 illustrates the findings of this research, which can be used by concerned authorities for their study.

Table 35 Findings and its Application

S.N	Findings	Stakeholder	Application
1	Buildings with columns 9”X9” are most vulnerable in comparison to 9”X12” and 12”X12” to future earthquakes.	LMC, Government of Nepal, NDRRMA	To conduct vulnerability assessment of existing housing stocks.
2	Major structural deficiency was insufficient reinforcement in columns.	LMC, DUDBC, NDRRMA	Provision of retrofitting existing structures based on defined building typology and retrofitting techniques developed by this research.
3	The applied retrofitting strategies for various typologies demonstrated acceptable level of performance in future earthquakes.	DUDBC NDRRMA	Developing retrofitting guidelines for all MRT compliant buildings based on the findings of this research. Incorporation of retrofitting as a mitigation measure and including it in National Strategic Action Plan (NSAP).

There are many MRT compliant RCC buildings in major cities of Nepal. This research studies only a group of representative MRT buildings in Lalitpur Metropolitan City. In depth study of larger populations can be done for even better depiction of building vulnerability and its need for retrofitting. Furthermore, current condition of existing buildings like concrete grade, construction practice, deterioration of building, presence of vulnerable factors can be taken for consideration for actual representation of existing buildings. Other mitigation measures can be studied comparing the time and cost.

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APPENDICES

APPENDIX-1 BUILDING DATA

S. N.	PLINTH AREA -SQFT	NO OF STOREYS	COLUMN SIZE	BEAM SIZE	SLAB THICKNESS	LENGTH	BAY	BREADTH	BAY	BUILDING HT	GF HT	FLOOR HEIGHT	GRADE	MRT CODE	LENGTH GRID	BREADTH GRID
1	**783.75**	2+sc	9"X9"	9"*13"	100mm	41'3"	4	19	2	26'9"	9'	9'	M15	205:1994	7'3"+11'9"+10'9"+10'9"	5'+11'+3'
2	**712.25**	3	9"X9"	9"*14"	100mm	48'6"	3	23'6"	2	31'6"	9'	9'	M15	205:1994	12'3"+13'9"+11'3"	11'+10'9"+1'
3	**625**	3+sc	9"X12"	9"*14"	100mm	23'1"	2	40'8"	3	39'4"	9'10"	9'10"	M20	205:1994	8'10"+11'5"	14'9"+11'11"+12'6"
4	**815.93**	3	9"x12"	9"*13"	100mm	33'5"	3	27'11	2	41'7"	9'	9'	M15	201:1994	11'10"+12'9"+4'9"	11'6"+11'6"
5	**970**	3	9"X12"	9"X14"	100mm	40'	3	24'3"	2	31'6"	9'	9'	M15	201:1994	15'+9'+15'	11'+13.75
6	**1092**	2+1/2	12"*12"	9"*14"	125mm	28'	2	39	3	32'6"	9'4"	9'4"	M20	205:2012	14'2"+12'10"	15'7"+11'4"+11'1"
7	**801.56**	1	9"X9"	9"*13"	100mm	33'8"	3	23'9"	2	14'	8'8"	8'8"	M15	205:1994	10'9"+11'5"+10'9"	11'6"+11'6"
8	**498.29**	3	9"X9"	9"*14"	100mm	24'10"	2	24'8"	3	33'5"	9'4"	9'4"	M15	205:1994	10'1"+14'	8'9"+7'9"+7'4"
9	**760.24**	3	9"X12"	9"X14"	125mm	25'5"	3	29'	3	32'	9'	9'	M15	201:1994	12'1"+12'9"	3'3"+11'6"+8'3"+5'
10	**456.47**	2+1/2	9"x12"	9"*14"	100mm	27'2"	2	16'8"	2	32'6"	9'4"	9'4"	M20	205:1994	10'2"+14'9"	9'3"+6'
11	**497.77**	2	9"x12"	9"*13"	100mm	21'4"	2	23'4"	2	19'1"	8'9"	8'9"	M15	201:1994	10'4"+10'4"	11'2"+11'2"

12	**599.45**	3	9"x12 "	9"X14"	100mm	23'9"	2	23'2"	2	32'6"	9'4"	9'	M15	201:1994	12'6"+10'6"	11'6"+12'2 "
13	**599.8**	3	12"*1 2"	9"*14"	125mm	28'	3	23'6"	2	32'6"	9'4"	9'	M15	205:1994	11'2"+6'9" +9'1"	11'8"+10'1 0"
14	**638.79**	2+sc	9"x12 "	9"*14"	125mm	31"	2	22'11 "	2	27'8"	9'	FF-9'	M20	201:1994	15'2"+11'	11'2"+10'9 "
15	**726**	2+sc	9"x12 "	9"*13"	100mm	30"	3	27"	2	31'9"	9'4"	9'4"	M15	201:1994	11'10"+12' 9"+4'9"	11'6"+11'6 "
16	**814.52**	2+sc	9"x12 "	9"*13"	100mm	35'3"	3	25	2	33'	9'	9'	M15	201:1994	10'9"+13'+ 10'9"	11'+9'3"'+ 3'
17	**915.28**	2+1/2	9"x12 "	9"*14"	125mm	35'2"	3	30'10 "	3	33'	9'4"	9'4"	M20	201:1994	8'1"+12"11 "+13'5"	3'+9'9"+1 3'1"+4'
18	**967.5**	3	9"x12 "	9"*13"	100mm	32'3"	3	30	3	32'6"	9'4"	9'4"	M15	201:1994	10'9"+10'+ 10'9"	9'6"+9'6"= 10'
19	**858**	2+sc	12"*1 2"	9"*13"	100mm	25	2	33	3	29'3"	9'5"	9'5"	M15	205:1994	11'+10'+11'	11'+13'+3'
20	**978.64**	3+sc	9"x12 "	9"*13"	100mm	29'6"	2	37'11 "	4	36'	9'4"	All 9'4"	M15	205:1994	10'6"+14'6"	3'3"+10'+ 12'2"+11'9
21	**755.25**	3	9"x12 "	9"X14"	125mm	26'6"	2	28'6"	3	33'	9'5"	9'5"	M15	201:1994	10'9"+14'9'	9'6"+8'3"+ 10
22	**828.58**	2+sc	9"x12 "	9"*13"	125mm or 5"	29'3"	2	32'8"	3	32'6"	9'4"	9'4"	M15	201:1994	13'6;;+14'9' '	9'8"+11'10 "+10'5"
23	**620.28**	3	9"x12 "	9"*13"	100mm	23'4"	2	29'10 "	3	32'6"	9'4"	9'4"	M15	201:1994	11'1"+11'6"	3'6"+12'3" +13'1"
24	**760.24**	3	9"X12 "	9"X14"	125mm	36'3"	3	20'11 "	3	32'	9'	9'	M15	201:1994	12'1"+12'9" +10'9"	3'3"+11'6" +8'3"+5'
25	**827.23**	2+sc	9"x12 "	9"*14"	125mm	36'5"	3	23'2"	2	32'6"	9'4"	9'4"	M15	201:1994	12'6"+9'+1 2'6"	11'2"+11'2 "
26	**998.64**	3	12"*1 2"	9"*13"	125mm or 5"	27'4"	3	35'5"	5	32'9"	9'	9'	M15	205:1994	13'5"+13'	10'11"+13' 4"+10'2"

27	**846.37**	3+sc	9"X9"	9"*13"	125mm or 5"	28'2"	3	46'4"	5	32'9"	9'	9'	M15	205:1994	11'11"+13'3"(+4'3" cantilever)	12'6"+14'5"
28	**815.93**	3+sc	9"x12"	9"*13"	100mm	33'5"	3	27'11"	2	41'7"	9'	9'	M15	201:1994		
29	**655**	3	12"*12"	9"*14"	125mm or 5"	18'3"	2	37'7"	3	32'6"	9'4"	9'4"	M20	205:2012	8'8"+8'8"	12'1"+14'8"+9'10"
30	**553.92**	2	9"x12"	9"*13"	100mm	23'1"	2	24	2	23'2"	9'4"	9'4"	M15	201:1994	12'1"+10'3"	11'7"+11'7"
31	**594.26**	2+sc	9"x12"	9"*13"	100mm	28'9"	2	20'8"	2	32'6"	9'4"	9'4"	M15	201:1994	14'+14'	9'10"+9'10"
32	**710.8**	2+sc	9"x12"	9"X14"	100mm	29'	4	27'2"	2	32'6"	9'4"	9'4"	M20	201:1994	5'+9'9"+7'6"+5'9"	13'9"+12'8"
33	**234.47**	3+sc	12"*12"	9"*14"	125mm or 5"	18'4"	2	13'1"	1	33'8"	8'5"	8'5"	M20	205:2012	8'11"+8'5"	11'11"(12'1")
34	**654.625**	3	9"X12"	9"X14"	100mm	29'	3	25'6"	2	32'6"	9'4"	9'4"	M15	201:1994	10'6"+10'6"+7'3"	14'+10'6"
35	**839.22**	3	12"*12"	9"*14"	125mm or 5"	34'7"	3	24'3"	2	32'1"	9'9"	9'9"	M20	205:2012	12'1"+12'+9'6"	12'6"+10'9"
36	**691.31**	2+sc	9"x12"	9"*13"	100mm	26'	3	29'	3	32'6"	9'4"	9'4"	M15	201:1994	10'3"+4'8"+10'5"	7'9"+10"+10'3"
37	**704.24**	3	9"X9"	9"X14"	100mm	37'6"	2	27'3"	4	32'6"	9'4"	9'4"	M15	205:1994	14'9"+11'	2'5"+7'10"+11'+6'8"(8'10")
38	**780.5**	2+sc	9"x12"	9"*14"	125mm	33'1"	5	27'8"	3	30'2"	9'4"	9'4"	M15	201:1994		unsymmetric grids
39	**698.75**	2+1/2	9"X9"	9"*13"	100mm	21'6"	2	32'6"	3	32'	9'	9'	M15	205:1994	12'9"+8'	12'6"+7'6"+9'
40	**625.31**	2+1/2	9"X9"	9"*13"	100mm	21'9"	2	28'9"	3	32'	9'	9'	M15	205:1994	12'+9'	12'+7'+9'
41	**428.95**	3	9"X12"	9"*13"	100mm	30'	3	18'5"	2	29'3"	8'4"	8'4"	M15	201:1994		

42	**566.44**	3	9"X12"	9"*13"	125mm	19'6"	2	29'5"	2	33'	9'	9'	M20	205:1994 but rebar 205 :2012	8'8"+8'8"	8'8"+4'3"+13'3"
43	**850**	2	9"X9"	9"*13"	100mm	28'7"	3	29'9"	2	23'2"	9'4"	9'4"	M15	205:1994	9'10"+8'2"+9'10"	14'6"+14'6"
44	**760.24**	3	9"X12"	9"X14"	125mm	36'3"	3	29'	3	32'	9'	9'	M20	201:1994	12'1"+12'9"+10'9"	3'3"+11'6"+8'3"+5'
45	**513.5625**	2	9"X9"	9"*13"	100mm	20'9"	2	24'9"	2	19'1"	8'9"	8'9"	M15	205:1994	10'+10'	12'+12'
46	**860**	2	12"*12"	9"*14"	125mm	37'5"	3	23'	2	23'2"	9'4"	9'4"	M20	205:2012	12'2"+11'4"+12'2"	11'9"+10'3"
47	**595.43**	3	9"X12"	9"X14"	125mm	24'2"	3	29'4"	3	32'6"	9'4"	9'4"	M15	201:1994	12'4"+10'10"	5'6"+12'1"+8'11"+2'
48	**492.66**	2	9"x12"	9"*13"	100mm	21'4"	2	23'4"	2	19'1"	8'9"	8'9"	M15	201:1994	10'6"+10'2"	11'+11'
49	**801**	3	12"*12"	9"*14"	125mm or 5"	31'7"	3	24'2"	2	32'3"	9'9"	9'9"	M20	205:2012	11'1"+10'6"'+9'8"	12'+11'2"
50	**611.97**	3	9"X9"	9"*13"	100MM	23'3"	3	25'4"	2	23'3"	9'	9'	M15	205:1994	10'9"+11'9"	12'9"+12'7"
51	**691.68**	2	9"X9"	9"*13"	100MM	23'3"	3	29'9"	2	23'2"	9'4"	9'4"	M15	205:1994	10'9"+11'9"	12'9"+11'9"
52	**606.13**	2	9"X12"	9"*13"	100mm	19'6"	2	29'5"	3	33'	9'	9'	M20	201:1994	8'8"+8'8"	8'8"+4'3"+13'3"
53	**410.6**	3	9"x12"	9"*13"	100mm	29'11"	3	17'10"	2	33"	9'4"	9'4"	M15	201:1994		
54	**625.33**	2+sc	9"x12"	9"*14"	125mm	31"	2	27'4"	2	22'11"	9'	9'	M20	201:1994	15'2"+11'	11'2"+10'9"
55	**529.37**	3	9"X9"	9"*13"	100mm	26'3"	2	20'2"	2	23'3"	9'	9'	M15	205:1994	12'6"+13'	9'5"+10'4"
56	**691.68**	1	9"X9"	9"*13"	100mm	23'3"	3	29'9"	2	13'10"	9'4"	9'4"	M15	205:1994	10'9"+11'9"	12'9"+11'9"

57	**429.02**	3	12"*12"	9"*14"	125mm	21'0	2	20'5"	2	32'6"	9'4"	9'4"	M20	205:2012	10'2"+9'11"	7'5"+12'
58	**624**	3	9"X9"	9"*14"	100mm	36'3"	3	24'	2	31'6"	9'	9'	M15	205:1994	11'3"+13'+11'3"	11'6"+11'6"+1'
59	**497.77**	2+SC	9"x12"	9"*13"	100mm	21'11"	2	21'8"	2	31'	8'9"	8'9"	M15	201:1994	9'9"+11'2"	10'4"+10'4"
60	**552.09**	2	9"x12"	9"*14"	125mm	19'6"	2	33'1"	3	21'8"	9'4"	9'4"	M20	205:2012	9'5"+9'4"	10'8"+11'1'+10'4"
61	**735.75**	2+1/2	9"X9"	9"*13"	100mm	27'3"	2	27'	3	33'3"	9	8'6"	M15	205:1994		
62	**538.37**	2+1/2	9"x12"	9"*14"	125mm	22'11"	2	24'5"	3	32'9"	9'4"	9'4"	M20	205:1994		
63	**425.95**	2+1/2	9"x12"	9"*14"	100mm	27'2"	2	15'1"	2	32'6"	9'4"	9'4"	M20	205:1994	10'2"+14'9"	9'3"+6'
64	**817.95**	3+sc	9"x12"	9"*14"	100mm	34'8"	3	24'9"	3	32'	9'	9'	M20	205:1994	12'+10'+11'8"	11'11"+12'1"€
65	**809**	2	12"*12"	9"*14"	125mm	24'6"	2	33'	3	21'8"	9'4"	9'4"	M20	205:2012	11'6"+12'	8'+12'+12'
66	**693.25**	1	9"X9"	9"*13"	100mm	29'6"	3	23'6"	2	14'	8'8"	8'8"	M15	205:1994	9'+10'6"'+9'	11'5+11'4"
67	**783.75**	2+sc	9"X9"	9"*13"	100mm	41'3"	4	19	2	26'9"	9'	9'	M15	205:1994	7'3"+11'9"+10'9"+10'9"	5'+11'+3'
68	**585**	1	12"*12"	9"*14"	125mm	23'	2	16'9"	1	13'10"	9'4"	9'4"	M15	205:1994	11'+11'	13'6"+(2'6")

APPENDIX 2- BUILDING TYPOLOGY

TYPE 1

	Building Height: 14’
Plinth Area: 693.25	Ground Floor Height: 8’8”
No. of Storey: 1	Floor Height: 8’8”
Column Size: 9”x9”	Concrete Grade: M15
Beam Size: 9”x13”	MRT Code: 205:1994
Slab Thickness: 100mm	MRT Satisfied: Yes but reinforcement slightly higher
Length: 29’6”	Column Reinforcements: Corners: 4-16+2-12, Remaining: 6-16
No. of Bays in Length: 3	Beam Reinforcements: 2-16+2-12(top), 2-16+1-16(bot)
Breadth: 23’6”	Slab Reinforcements: 10 mm @8"c/c
No. of Bays in Breadth: 2	

TYPE 2

	Building Height: 23’2”
Plinth Area: 689.875 - 850	Ground Floor Height: 9’4”
No. of Storey: 2	Floor Height: 9’4”
Column Size: 9”x9”	Concrete Grade: M15
Beam Size: 9”x13”	MRT Code: 205:1994
Slab Thickness: 100mm	MRT Satisfied: Yes
Length: 28’7” – 29’9”	Column Reinforcements: All 4-16
No. of Bays in Length: 3	Beam Reinforcements: 2-12+1-12(top), 2-12+1-12(bot)
Breadth: 23’3” – 29’6”	Slab Reinforcements: 8 dia and 10 dia @6"c/c
No. of Bays in Breadth: 2	

TYPE 3

Building Height: 21’8” – 29’6”

Plinth Area: 552.09 – 606.13

No. of Storey: 2

Column Size: 9”x12”

Beam Size: 9”x13” – 9’14”

Slab Thickness: 100mm

Length: 19’6”

No. of Bays in Length: 2

Breadth: 29’5”- 33’1”

No. of Bays in Breadth: 3

Ground Floor Height: 9’4”

Floor Height: 9’4”

Concrete Grade: M20

MRT Code: 205:2012

MRT Satisfied: Yes

Column Reinforcements:

c1-(Corners) Gf: 4-16+4-12 ff&above: 4-16+2-12

c2 (face) Gf: 4-16+4-12 ff&above: 4-16+2-12

c3 (interior) Gf: 4-16+2-12 ff&above: 4-16+2-12

Stirrups: 8 dia @3"c/c and 6"c/c (3 legged)

Beam Reinforcements:

Top: 2-16(T) +2-12 Ex

Bottom: 2-16+1-16 Ex

Stirrups: 8 dia @ 4"c/c and 6"c/c

Slab Reinforcements: 8 dia @6"c/c and 8 dia(distribution)

TYPE 4

Building Height: 23’2”

Plinth Area: 553.92

No. of Storey: 2

Column Size: 9”x12”

Beam Size: 9”x13”

Slab Thickness: 100mm

Length: 23’1”

No. of Bays in Length: 2

Breadth: 24’

No. of Bays in Breadth: 2

Ground Floor Height: 9’4”

Floor Height: 9’4”

Concrete Grade: M15

MRT Code: 201:1994

MRT Satisfied: Yes

Column Reinforcements: All: 4-16+2-12 Stirrups 7 dia

Beam Reinforcements:

Top: 2-16(T) +2-12 Ex

Bottom: 2-16+1-16 ex

Stirrups: 7 dia @ 4"c/c and 6"c/c

Slab Reinforcements: 8 dia @6"c/c and 7 dia (distribution)

TYPE 5

Building Height: 31’

Plinth Area: 497.77

Ground Floor Height: 8’9”

No. of Storey: 2 + SC

Floor Height: 8’9”

Column Size: 9”x12”

Concrete Grade: M15

Beam Size: 9”x13”

MRT Code: 201:1994

Slab Thickness: 100mm

MRT Satisfied: Yes

Length: 21’11”

Column Reinforcements: 4-16 (All)

No. of Bays in Length: 2

Beam Reinforcements:

Breadth: 21’8”

Top & Bottom: 2-16(T) +1-16 Ex

No. of Bays in Breadth: 2

Stirrups: 7 dia @ 4"c/c and 6"c/c

Slab Reinforcements: Main:8 dia @6"c/c and 7 dia(distribution)

TYPE 6

Plinth Area: 497.77

No. of Storey: 2 + SC

Floor Height: 8’9”

Column Size: 9”x12”

Concrete Grade: M15

Beam Size: 9”x13”

MRT Code: 201:1994

Slab Thickness: 100mm

MRT Satisfied Yes

Length: 21’11”

Column Reinforcements: 4-16 (All)

No. of Bays in Length: 2

Beam Reinforcements:

Breadth: 21’8”

Top & Bottom: 2-16(T) +1-16 Ex

No. of Bays in Breadth: 2

Stirrups: 7 dia @ 4"c/c and 6"c/c

Slab Reinforcements: Main:8 dia @6"c/c and 7 dia(distribution)

TYPE 7

Building Height: 32’6”

Plinth Area: 967.5

Ground Floor Height: 9’4”

No. of Storey: 3

Floor Height: 9’4”

Column Size: 9”x12”	Concrete Grade: M15
Beam Size: 9”x13”	MRT Code: 201:1994
Slab Thickness: 100mm	MRT Satisfied: Yes
Length: 32’3”	Column Reinforcements:
No. of Bays in Length: 3	c3-(Corners) Gf: 4-16+4-12 ff&above: 4-16+2-12
Breadth: 30’	c2 (face) Gf: 4-16+4-12 ff&above: 4-16+2-12
No. of Bays in Breadth: 3	c1 (interior) Gf: 4-16+2-12 ff&above: 4-16+2-12
	Stirups: 8 dia @3"c/c and 6"c/c
	Beam Reinforcements: 2+16(top), 2-16(bottom)
	Slab Reinforcements: Main: 8 dia @6"c/c & 7 dia (distribution)

TYPE 8

	Building Height: 33’
Plinth Area: 755.25	Ground Floor Height: 9’5”
No. of Storey: 3	Floor Height: 9’5”
Column Size: 9”x12”	Concrete Grade: M15
Beam Size: 9”x14”	MRT Code: 201:1994
Slab Thickness: 125mm	MRT Satisfied: Yes
Length: 26’6”	Column Reinforcements:
No. of Bays in Length:2	c1-(INTERIOR): Gf &ff: 4-16+4-12 sf: 4-12+2-12
Breadth: 28’6”	c2 (face) Gf&FF: 4-12+2-12
No. of Bays in Breadth: 3	c3 (CORNER) Gf: 4-16+2-12 ff&above: 4-16
	Stirups: 8 dia @3"c/c and 6"c/c
	Beam Reinforcements: T-2-16+1-16, B-2-16
	Slab Reinforcements: 10 mm & 8mm @6"c/c

TYPE 9

	Building Height: 32’6”
Plinth Area: 655	Ground Floor Height: 9’4”

No. of Storey: 3 | Floor Height: 9’4”
Column Size: 12”x12” | Concrete Grade: M20
Beam Size: 9”x14” | MRT Code: 205:2012
Slab Thickness: 125mm | MRT Satisfied: Yes
Length: 18’3” | Column Reinforcements:
No. of Bays in Length: 2 | c1 (interior) 8-12
Breadth: 37’7” | c2 face 4-16+4-12
No. of Bays in Breadth: 3 | corner 6-16+2-12

Beam Reinforcements: 2-16+3-12 top, 3-12+1-16 bottom (extra)

Slab Reinforcements: 8 dia @6"c/c

TYPE 10

Building Height: 32’6”

Plinth Area: 429.02

Ground Floor Height: 9’4”

No. of Storey: 3

Floor Height: 9’4”

Column Size: 12”x12”

Concrete Grade: M20

Beam Size: 9”x14”

MRT Code: 205:2012

Slab Thickness: 125mm

MRT Satisfied: Yes

Length: 21’

Column Reinforcements: 4-16+4-12 & 8mm stirrups

No. of Bays in Length:2

Beam Reinforcements: 2-16+1-16(e) top, 2-16+1-12 bottom

Breadth: 20’5”

Slab Reinforcements: 8 dia @6"c/c

No. of Bays in Breadth: 2

TYPE 11

Building Height: 32’6”

Plinth Area: 599.8 – 655

Ground Floor Height: 9’4”

No. of Storey: 3

Floor Height: 9’4”

Column Size: 12”x12”

Concrete Grade: M15

Beam Size: 9”x13”

Slab Thickness: 125mm

Length: 18’3” – 28’

No. of Bays in Length: 3

Breadth: 23’6” – 37’7”

No. of Bays in Breadth: 2

MRT Code: 201:1994

MRT Satisfied: Reinforcement slightly high

Column Reinforcements:

c1 (interior) 8-12

c2face 4-16+4-12

corner 6-16+2-12

Beam Reinforcements: 2-16+2-12 extra top, 2-16+1-16 bottom

Slab Reinforcements: 8 dia @6"c/c

TYPE 12

Building Height: 19’1”

Plinth Area: 492.66 – 497.77

No. of Storey: 2

Column Size: 9”x12”

Beam Size: 9”x13”

Slab Thickness: 100mm

Length: 21’4” – 23’3”

No. of Bays in Length: 2

Breadth: 21’4” – 23’4”

No. of Bays in Breadth: 2

Ground Floor Height: 8’9”

Floor Height: 8’9”

Concrete Grade: M15

MRT Code: 205:1994

MRT Satisfied: Yes

Column Reinforcements:

c1-(Corners) Gf: 4-16+4-12 ff&above: 4-16+2-12

c2 (face) Gf: 4-16+4-12 ff&above: 4-16+2-12

c3 (interior) Gf: 8-16 ff&above: 8-16

Stirups: 8 dia @3"c/c and 6"c/c

Beam Reinforcements:

Top & Bottom: 2-16(T) +1-16 Ex

Stirrups: 7 dia @ 4"c/c and 6"c/c

Slab Reinforcements: Main: 8 dia @6"c/c and 7 dia(distribution)

TYPE 13

Building Height: 33’

Plinth Area: 814.52

No. of Storey: 2 + SC

Column Size: 9”x12”

Beam Size: 9”x13”

Ground Floor Height: 9’

Floor Height: 9’

Concrete Grade: M15

MRT Code: 205:1994

Slab Thickness: 100mm

Length: 35’3”

No. of Bays in Length: 3
12

Breadth: 25’

No. of Bays in Breadth: 2

MRT Satisfied: Yes

Column Reinforcements:

c1-(Corners) Gf: 4-16+4-12 ff&above: 4-16+4-

c2 (face) Gf: 4-16+4-12 ff&above: 4-16+2-12

c3 (interior) Gf: 4-16+4-12 ff&above: 4-16+2-12

Stirups: 7 dia @3"c/c and 6"c/c

Beam Reinforcements:

Top: 2-16(T) +1-12 Ex

Bottom: 2-16+1-12e

Stirrups: 7 dia @ 4"c/c and 6"c/c

Slab Reinforcements: Main: 10 dia @6"c/c & 8 dia(distribution)

TYPE 14

Plinth Area: 425.95 – 456.47

No. of Storey: 2 + 1/2

Column Size: 9”x12”

Beam Size: 9”x14”

Slab Thickness: 100mm

Length: 27’2”

No. of Bays in Length:2
and bottom)

Breadth: 15’1” – 16’8”

No. of Bays in Breadth: 2

Building Height: 32’6”

Ground Floor Height: 9’4”

Floor Height: 9’4”

Concrete Grade: M20

MRT Code: 205:1994

MRT Satisfied: Yes

Column Reinforcements: All Gf:4-16+2-12

Beam Reinforcements: Beam: 2-16+2-16e (top

Slab Reinforcements: Main-10dia and dist 8 dia

TYPE 15

Plinth Area: 566.44

No. of Storey: 3

Column Size: 9”x12”

Beam Size: 9”x13”

Slab Thickness: 125mm

Length: 19’6”

No. of Bays in Length: 2

Building Height: 33’

Ground Floor Height: 9’

Floor Height: 9’

Concrete Grade: M20

MRT Code: 201:1994

MRT Satisfied: Yes

Column Reinforcements:

Corners: 4-16+4-12

Breadth: 29’5”

No. of Bays in Breadth: 3

Rest: 4-16+2-12(All Floors)

Beam Reinforcements: max: 2-16+1-16€ Top and bottom

Slab Reinforcements: 10 mm main and 8 mm dist @6"c/c

TYPE 16

Plinth Area: 978.64

No. of Storey: 3 + SC

Column Size: 9”x12”

Beam Size: 9”x13”

Slab Thickness: 100mm

Length: 29’6”

No. of Bays in Length: 2

Breadth: 37’11”

No. of Bays in Breadth: 4

Building Height: 36’ (except plinth and parapet)

Ground Floor Height: 9’4”

Floor Height: All 9’4” except top – 8’

Concrete Grade: M15

MRT Code: 205:1994

MRT Satisfied: All except slab area=168 sqft (>150sqft)

(size of columns as per 205:1994)

Column Reinforcements:

c1-(INTERIOR): Gf: 4-16+4-12 ff&above: 4-12+4-12

c2 (face) Gf&FF: 4-16+2-12 Sf&above: 4-12+2-12

c3 (CORNER) Gf: 4-16+2-12 ff&above: 4-16

Stirups: 8 dia @3"c/c and 6"c/c

Beam Reinforcements: As per205:1994, 2-16+3-12(T)2-16+1-16

Slab Reinforcements: As per 205:1994 i.e main 10 dia & dis 8dia

TYPE 17

Plinth Area: 817.95

No. of Storey: 3 + SC

Column Size: 9”x12”

Beam Size: 9”x14”

Slab Thickness: 100mm

Length: 34’8”

Building Height: 32’

Ground Floor Height: 9’

Floor Height: 9’

Concrete Grade: M20

MRT Code: 205:1994

MRT Satisfied: Yes except grid alignment

Column Reinforcements: 4 – 16 (All)

No. of Bays in Length:3

Beam Reinforcements: 2-16+1-16 extra (top), 2-16+1-16(bot)

Breadth: 24’9”

Slab Reinforcements: 7 dia & 8 dia @6"c/c

No. of Bays in Breadth: 3

TYPE 18

Building Height: 21’8”

Plinth Area: 809

Ground Floor Height: 9’4”

No. of Storey: 2

Floor Height: 9’4”

Column Size: 12”x12”

Concrete Grade: M20

Beam Size: 9”x14”

MRT Code: 205:2012

Slab Thickness: 125mm

MRT Satisfied: Yes

Length: 24’6”

Column Reinforcements: Corners: 4-16+4-12 and 8mm stirrups

No. of Bays in Length:2

Beam Reinforcements: 2-16+1-16(e) top, 2-16+1-12 bottom

Breadth: 33’

Slab Reinforcements: 8 dia @6"c/c

No. of Bays in Breadth: 3

TYPE 19

Building Height: 29’3”

Plinth Area: 858

Ground Floor Height: 9’5”

No. of Storey: 2 + SC

Floor Height: 9’5”

Column Size: 12”x12”

Concrete Grade: M20

Beam Size: 9”x13”

MRT Code: 205:1994

Slab Thickness: 100mm

MRT Satisfied: Yes

Length: 25’

Column Reinforcements: 4-16+4-12 & 8mm stirrups

No. of Bays in Length:2

Beam Reinforcements: 2-16+1-16(e) top, 2-16+1-12 botto

Breadth: 33’

Slab Reinforcements: 8 dia @6"c/c

No. of Bays in Breadth: 3

TYPE 20

	Building Height: 32’6”
Plinth Area: 594.26	Ground Floor Height: 9’4”
No. of Storey: 2 + SC	Floor Height: 9’4”
Column Size: 9”x12”	Concrete Grade: M15
Beam Size: 9”x13”	MRT Code: 201:1994
Slab Thickness: 100mm	MRT Satisfied: Yes
Length: 28’9”	Column Reinforcements:
No. of Bays in Length:2	8-12(Gf and 1st Floor)
Breadth: 20’8”	6-12(2nd Floor)
No. of Bays in Breadth: 2	Stirrups: 8 dia @4"c/c and 6"c/c
	Beam Reinforcements:
	Top: 3-12(T) +2-16 Ex
	Bottom: 3-16
	Stirrups: 8 dia @ 4"c/c and 6"c/c
	Slab Reinforcements: 8 dia @6"c/c and 8 dia (distribution)

APPENDIX-3 RETROFITTING STRATEGY/GUIDELINE SUMMARY

Based on the results of seismic and pushover analysis, the buildings confirming to the specifications of the various typologies can be assigned the following retrofitting techniques. For typologies with more than one retrofitting option, any one method can be used as per preference and guidelines of materials used for each method i.e. number and diameter of bars, steel plates, size of jacketing, thickness of shear wall should be followed.

<u>Typology A</u>

Number of storey=1

Column Size: 9”X9”

Column Rebar: 4-16+2-12

Retrofitting technique used: RC jacketing and Steel Jacketing

RC jacketing:

100mm Concrete thickness with 4 numbers of 12 and 4 numbers of 16mm rebar used with 8 mm stirrups.

Steel jacketing: Section L50X50X5 used

Performance Level: Immediate Occupancy

<u>Typology B</u>

Number of storey=1

Column Size: 9”X9”

Column Rebar: 4-16

Retrofitting technique used: RC jacketing and Steel Jacketing

RC jacketing:

100mm Concrete thickness with 8 numbers of 16 mm rebar used with 8 mm stirrups.

Steel jacketing: Section L50X50X5 used

Performance Level: Immediate Occupancy

<u>Typology C</u>

Number of storey=2 (+SC)

Column Size: 9”X9”

Column Rebar: 4-16+2-12

Retrofitting technique used: RC jacketing

RC jacketing:

100mm Concrete thickness with 8 numbers of 16 and 4 numbers of 12mm rebar used with 8 mm stirrups.

Performance Level: Life Safety

Typology D

Number of storey=2

Column Size: 9”X12”

Column Rebar: 4-16+2-12

Retrofitting technique used: Steel Jacketing

Steel jacketing: Section L75X75X8 used

Performance Level: Immediate Occupancy

Typology E

Number of storey=2 +Staircase Cover

Column Size: 9”X12”

Column Rebar: 4-16+4-12(Gf and 1st Floor)
4-16+2-12(Upper Floor)

Retrofitting technique used:

1. Addition of Shear Wall:

230mm Shearwall with 12 mm rebar at 150 mm spacing along parallel grids.

Performance Level: Life Safety

2. Concrete and Steel Jacketing

100mm Concrete thickness with 8 numbers of 16 and 4 numbers of 12mm rebar used with 8 mm stirrups for two parallel grids.

Section L50X50X5 used for steel jacketing for all columns.

Performance Level: Immediate Occupancy

Typology F

Number of storey=2 +Staircase Cover

Column Size: 9”X12”

Column Rebar: 4-16+2-12

Retrofitting technique used: RC jacketing and Steel Jacketing

RC jacketing:

100mm Concrete thickness with 8 numbers of 16 mm rebar used with 8 mm stirrups for all columns in ground floor

Steel jacketing: Section L50X50X5 used for all columns above ground floor.

Performance Level: Immediate Occupancy

Typology G

Number of storey=3

Column Size: 9”X12”

Column Rebar: 4-16+4-12

Retrofitting technique used:

Option 1.Concrete and steel jacketing:

100mm concrete thickness with 4 numbers of 16 mm rebar and 4 numbers of 12 mm rebar used with 8 mm stirrups for all columns in ground floor and steel jacketing with section L50X50X5 for all columns above ground floor.

Performance Level: Damage Control

Option 2: Shear wall addition and steel jacketing:

230mm shearwall with 12 mm rebar at 150 mm spacing in ground floor and steel jacketing using L50X50X5 for all columns above ground floor.

Performance Level: Immediate Occupancy

Typology H

Number of storey=3

Column Size: 9”X12”

Column Rebar: 4-16+4-12

Retrofitting technique used:

Shear wall addition and steel jacketing:

230mm shearwall with 12 mm rebar at 150 mm spacing in ground floor and steel jacketing using L50X50X5 for all columns above ground floor.

Performance Level: Immediate Occupancy

Typology I

Number of storey=1+Staircase Cover

Column Size: 12”X12”

Column Rebar: 4-16+4-12 and 6-16+2-12

Retrofitting technique used:

Not Required

Performance Level: Damage Control

<u>Typology J</u>

Number of storey=3

Column Size: 12”X12”

Column Rebar: 4-16+4-12

Retrofitting technique used:

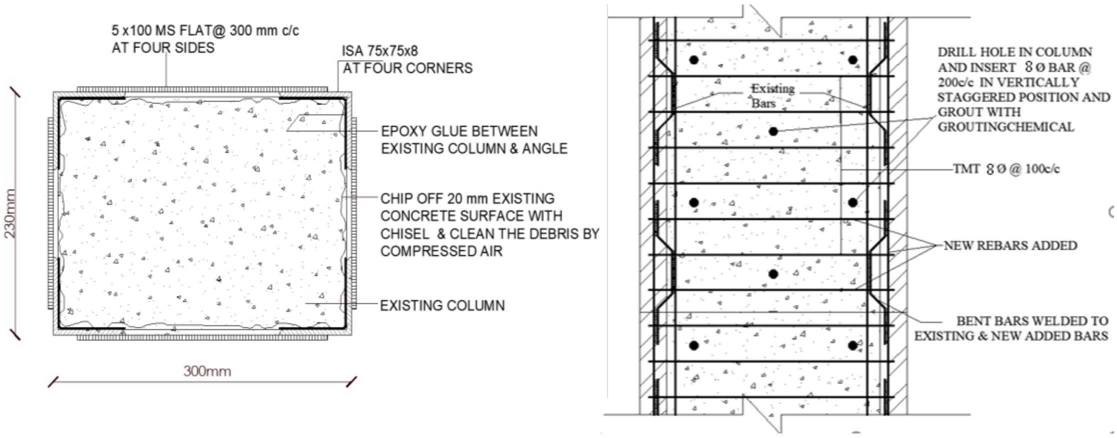
Shear wall addition and RC jacketing:

230mm shearwall with 12 mm rebar at 150 mm spacing in ground floor and 100mm concrete thickness with 4 numbers of 16 and 2 numbers of 12mm rebar used with 8 mm stirrups.

Performance Level: Damage Control, For RC jacketing, rebar with a minimum diameter of 12 mm should be used and concrete with a minimum thickness of 100 mm. The ties must have a minimum diameter of 8 mm and a maximum spacing of 200 mm.

All four corners of the steel jacketing shall be reinforced with steel angles with a minimum dimension of L 50X50X5. Steel plate with a minimum thickness of 5mm and no more than 300mm between each piece shall be provided.

The reinforcements for the shear wall addition should be at least 150 mm thick (ideally 230 mm), and they should be connected to the existing columns with extra welded connections.



(a) (b)

Figure 69: Retrofitting detail of column (a- typical plan, b-typical section)

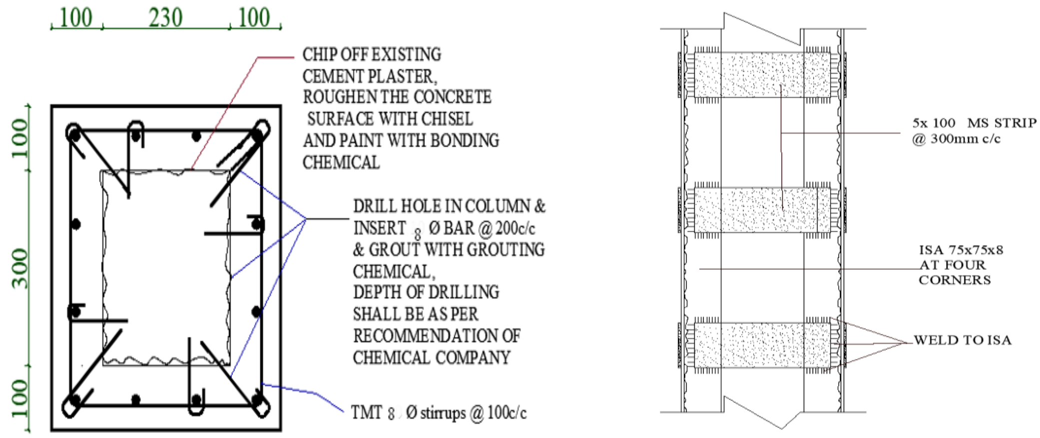


Figure 70: Detail of Steel Jacketing Plan (a), Typical Section of Steel Jacketing (b)

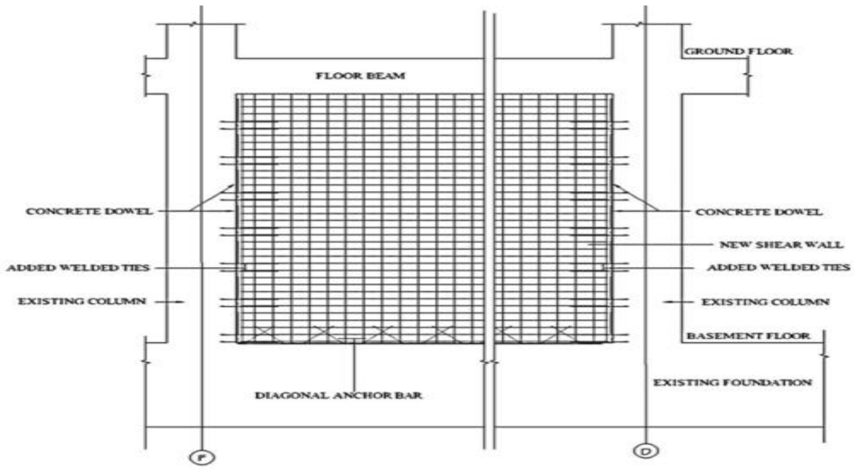


Figure 71: Typical details of Shear Wall Addition