# SEISMIC RETROFITTING OF MULTI-STOREY REINFORCED CONCRETE BUILDING USING FRP JACKETING

**A DISSERTATION**

***Submitted in the partial fulfilment of the requirements for the award of the degree of***

**MASTER OF TECHNOLOGY**

**in**

**EARTHQUAKE ENGINEERING**

**(With specialization in Structural Dynamics)**

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# CANDIDATE'S DECLARATION

I hereby declare that the work presented in this dissertation entitled **"Seismic Retrofitting of Multi-Storey Reinforced Concrete Building Using FRP Jacketing",** submitted in the partial fulfilment of the requirement for the award of the degree of **Master of Technology** in **Department of Earthquake Engineering** from **Indian Institute of Technology Roorkee**, under the guidance of **Dr. Pankaj Agarwal** Professor and Head in Department of Earthquake Engineering, Indian Institute of Technology Roorkee. It is an authentic record of my work carried out during June 2018 to June 2019.

I certify that the work presented in this dissertation has not been submitted for award of any other degree or diploma.

Place: Roorkee — Pradeep Mishra

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# CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of my knowledge and belief.

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At last, I would like to express my thankfulness to my parents and family members for their love, inspiration and constant encouragement during the course of this dissertation.

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# ABSTRACT

Past earthquakes including the Bhuj (2001) in which many concrete structures have been severely damaged or collapsed, have indicated the need for evaluating the seismic adequacy of existing buildings. In particular, the seismic rehabilitation of older concrete structures in high seismicity areas is a matter of growing concern, since structures vulnerable to damage must be identified and an acceptable level of safety must be determined. The major part of the seismic threat to human life and property comes from old buildings. Concrete buildings constructed in the past without a proper seismic design poses a serious public safety problem. In a densely populated country like India, it can cause a magnified effect on the earthquake hazard, it is really of prime importance to strengthen the seismically vulnerable structures.

In this study, an existing building which is primarily a moment resisting frame of seven storeys, has been analysed for gravity and seismic loading, and is retrofitted for seismic demand. The capacity of the existing and retrofitted building has been evaluated and compared through non-linear static analysis of the building. The local retrofitting techniques FRP jacketing has been used to strengthen the existing building and improving the overall performance behavior of the building. The enhancement in the existing member capacities has been evaluated through their strength and ductility parameters, i.e., through axial load-moment-interaction diagrams, moment-curvature diagrams and moment-rotation diagrams. The global performance of the structure has been evaluated through nonlinear static analysis of the existing and retrofitted building. The study shows that, the member ductility and strength parameters can be greatly enhanced from FRP wrapping, but their effect in the global performance of the structure is less effective; particularly in terms of enhancing the base shear carrying capacity.

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# CHAPTER 1 INTRODUCTION

## 1.1 General

Due to poor construction practices and ignorance for earthquake resistant design of buildings in our country, most of the existing buildings are vulnerable to future earthquakes. They fail due to unpredicted seismic motion causing extensive damage to numerous buildings of varying degree *i.e.* either full or partial or slight. In light of these facts, it is extremely important to seismically evaluate the existing building with the present day knowledge to avoid major destruction in the future earthquakes. The Buildings found to be seismically deficient should be retrofitted/strengthened.

Also with the increasing problem of aging infrastructures, the structural retrofit work has come to the forefront of the industry practice. The direct and indirect costs of demolition and reconstruction of structurally deficient constructions are prohibitive for most individual owners, as well as for the national or regional economy. Moreover, it is unacceptable for the society and for the environment, as it represents a large and unnecessary waste of natural resources and energy. Therefore, structural retrofitting is becoming more and more important and receives considerable emphasis throughout the world.

In 2013, Bureau of Indian Standards (BIS) has published a guideline for seismic evaluation and retrofitting of structures, IS 15988:2013. In this code, various retrofitting techniques has been discussed and protocol to use them have been issued.

## 1.2 Retrofitting

Retrofitting is the modification of existing structures to make them more resistant to seismic activity or ground motion. The necessity of seismic retrofitting of structures arises under following circumstances:

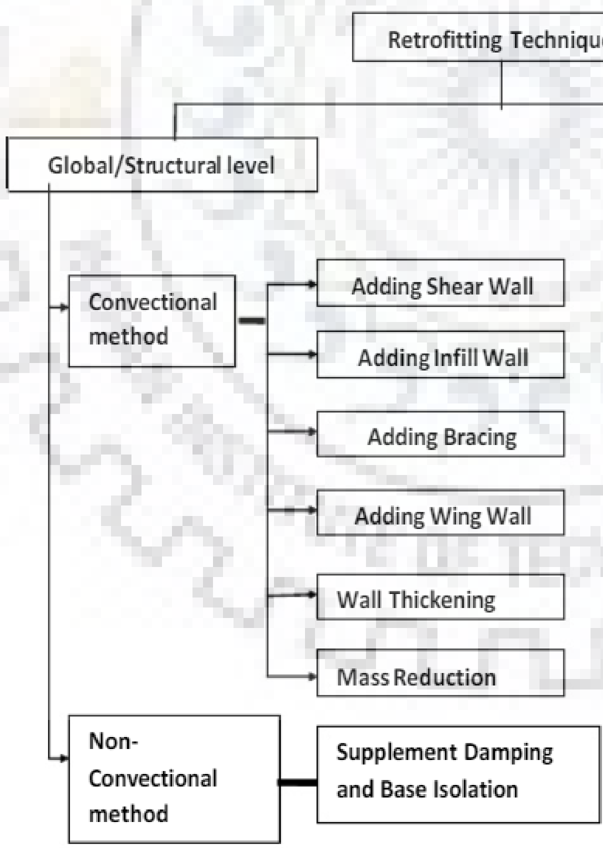
- Earthquake damaged structures.
- Earthquake-vulnerable structures that have not yet experienced severe earthquakes.
- The structure is not designed as per the codes.
- Subsequent revision of codes and design practice.
- Deterioration of strength due to aging of the building.

- Change in the use of the building.

### 1.2.1 Retrofitting Methods

Retrofitting methods that should be applied to a structure depends on the level of deficiency of the existing structure, it's required performance, availability of spaces etc. The retrofitting technique can be local in which only member level performances are enhanced or global through which the overall performance of the structure is upgraded (Agarwal and Shrikhande 2007).

The local retrofitting techniques are mainly done through confining the members which is discussed in subsequent sections in the dissertation. While in global retrofitting, an additional members are installed in the existing system, primarily aimed to transfer the lateral load to the foundation through an alternative path. Global retrofitting can also be done by reducing the seismic demand on the structure, which is known as unconventional method off retrofitting. The various techniques of retrofitting method are described as follow:



## 1.3 Fiber Reinforced Polymer (FRP)

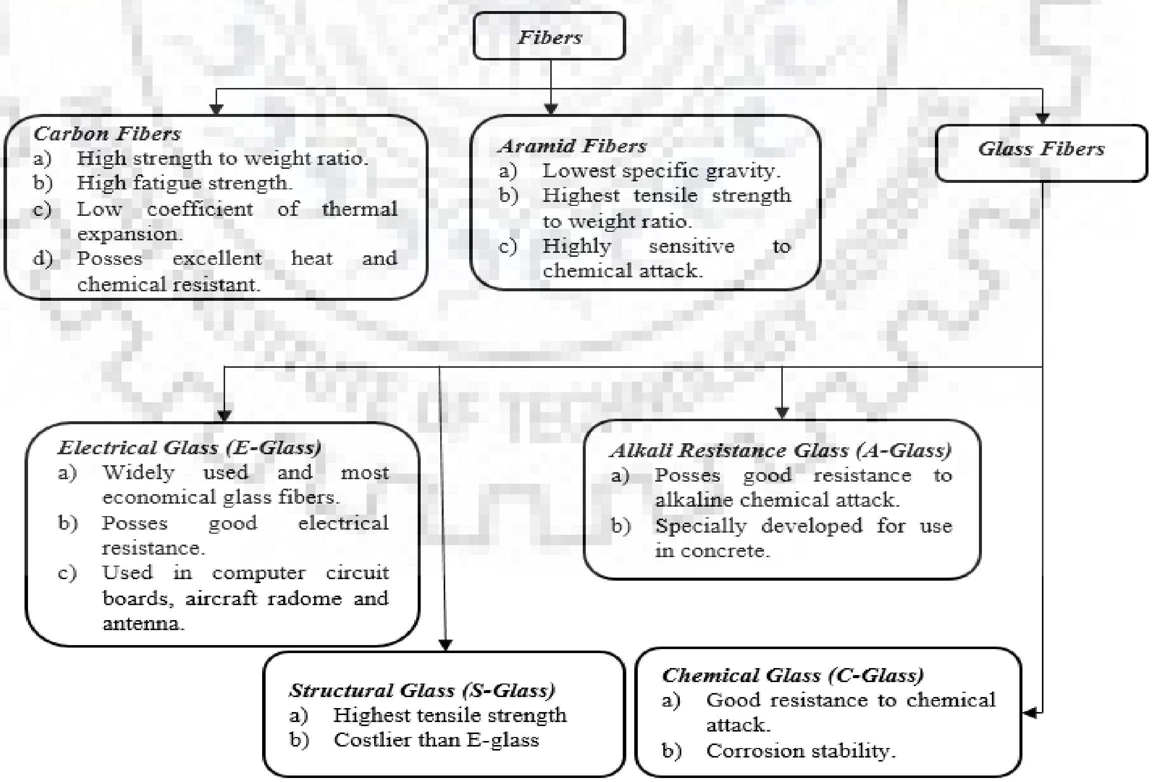
FRP is a composite material consists of *high strength fibers* embedded in a *polymer matrix.* The fibers of the composite are mainly responsible for strength and the stiffness properties, while the polymeric matrix contributes to the load transfer and provides environmental protection. The common fiber types include carbon, aramid, and glass; whereas common matrices are epoxies and esters.

The properties of FRP which enables its use in construction industry can be summarized as follow:

- High fatigue resistance.
- Excellent tensile strength in the direction of fibers.
- High strength to weight ratio.
- Extremely corrosion resistant.

## 1.4 Type of fibers used in composites

The types of fibers used in structural application are carbon fiber, glass fiber, and aramid fibers. A brief description of each type is described below:



FRP materials are lightweight, noncorrosive, and display high tensile strength. These materials are easily available in several forms, ranging from factory-made laminates to dry fiber sheets that can be enclosed to conform to the geometry of a structure before addition of polymer resin. The thin profiles of cured FRP systems are often desirable in applications where aesthetics appearance or access is a concern.

A comparison of typical properties for different FRP composites i.e., high strength carbon fiber, high modulus and ultra-high modulus carbon fiber, E-glass, S-glass, and aramid fiber is shown in Table 1.1

*Table 1.1 Comparative properties for various FRP composites (ACI 440.2R-2008)*

| Fiber type | Elastic modulus | | Ultimate strength | | Rupture strain, minimum, % |
|---|---|---|---|---|---|
| | $10^3$ ksi | GPa | ksi | MPa | |
| **Carbon** | | | | | |
| General purpose | 32 to 34 | 220 to 240 | 300 to 550 | 2050 to 3790 | 1.2 |
| High-strength | 32 to 34 | 220 to 240 | 550 to 700 | 3790 to 4820 | 1.4 |
| Ultra-high-strength | 32 to 34 | 220 to 240 | 700 to 900 | 4820 to 6200 | 1.5 |
| High-modulus | 50 to 75 | 340 to 520 | 250 to 450 | 1720 to 3100 | 0.5 |
| Ultra-high-modulus | 75 to 100 | 520 to 690 | 200 to 350 | 1380 to 2400 | 0.2 |
| **Glass** | | | | | |
| E-glass | 10 to 10.5 | 69 to 72 | 270 to 390 | 1860 to 2680 | 4.5 |
| S-glass | 12.5 to 13 | 86 to 90 | 500 to 700 | 3440 to 4140 | 5.4 |
| **Aramid** | | | | | |
| General purpose | 10 to 12 | 69 to 83 | 500 to 600 | 3440 to 4140 | 2.5 |
| High-performance | 16 to 18 | 110 to 124 | 500 to 600 | 3440 to 4140 | 1.6 |

# CHAPTER 2 LITERATURE REVIEW

This chapter of the dissertation primarily deals with the work done in the past by researchers, related to confinement of concrete due to fiber reinforced polymer (FRP), internal transverse reinforcement etc. The models that researchers has proposed for the evaluation of strength and ductility parameters of the concrete confined by various confinement methods is also discussed in this chapter.

**Mander et al. (1988)** had developed a stress-strain model for concrete confined by any general type of internal transverse reinforcements. They have also checked for the dynamic characteristics of the concrete under varying strain rate. The confining reinforcements may or may not have equal confining pressure in either direction. The model allows the determination of peak stress and corresponding strain of the concrete confined by circular, spiral, or rectangular hoops in mathematical relationship under uni-axial loading. They confirmed that the confinement of the concrete by suitable arrangement of the transverse reinforcement results in a significant increase in both the strength and ductility of the sample under the mentioned conditions.

**Benzaid and Mesbah (2013)** conducted a series of test to evaluate the performance of circular and square column strengthened with carbon fiber reinforced polymer (CFRP) sheets. Based on the effective lateral confining pressure and the effective circumferential failure strain, they had proposed numerical relationships so as to predict the strength of FRP confined concrete and the corresponding failure strain in the CFRP sheet. They concluded a new confinement model for CFRP wrapped square and circular section which takes into account the ratio of actual failure strain to the ultimate strain of the composites. They concluded that a circular section showed better performance for strength and ductility as compared to the square section with similar confining arrangement. The failure of the specimen is observed to be brittle and sudden owing to the material property of the confining materials. The CFRP confinement yielded higher results in terms of strength and ductility (Hadigheh et al. 2014) for low-strength concrete specimen as compared to higher strength concrete grade.

**Eslami and Ronagh (2012)** carried out an analytical study to investigate the efficiency the glass fiber reinforced polymers (GFRP) in improving the seismic performance of the reinforced concrete (RC) frames. They had analysed two 8-storey 2D RC frames with different stirrups confinement before and after wrapping with GFRP laminates. The original structure was wrapped with GFRP sheets wrapped around the members at their critical regions for energy dissipation. Nonlinear static analysis has been performed for the existing and retrofitted frame using SAP2000 with manually defined hinge parameters and lumped plasticity approach. The stress-strain model proposed by Lam and Teng (2003a,b) was selected for FRP-confined concrete. The retrofitting strategy used was focused on increasing the ductility of the frame rather than enhancing the strength of the same. They concluded that the FRP was inefficient in increasing the ductility of the code compliant frames due to their better energy dissipation characteristics owing to better transverse detailing whereas for poorly confined frames, the increase in ductility was significant

**ACI Committee 440** (ACI 440.2R-2008)***:*** This committee adopted the stress-strain model predicted by Lam and Teng (2003a,b) for concrete confined with FRP. This model predicts the maximum compressive strength *($f_{cc}'$)* and ultimate axial strain *($\varepsilon_{ccu}$)* for FRP confined rectangular concrete columns subjected to axial loading. A shape factor is also introduced, that corresponds to the confinement effectiveness coefficient. The formulas used for calculation of maximum confined concrete compressive strength and maximum confinement pressure is given by;

$$f_{cc}' = f_c' + \psi_f 3.3 \kappa_a f_l \qquad \text{Eqn. (2.1)}$$

$$f_l = \frac{2nE_f t_f \varepsilon_{fe}}{D} \qquad \text{Eqn. (2.2)}$$

$$\varepsilon_{fe} = \kappa_\varepsilon \varepsilon_{fu} \qquad \text{Eqn. (2.3)}$$

The peak strain value of FRP wrapped member is given by,

$$\varepsilon_{ccu} = \varepsilon_c' \left\{ 1.50 + 12\kappa_b \left( \frac{f_l}{f_c'} \right) \left( \frac{\varepsilon_{fe}}{\varepsilon_c'} \right)^{0.45} \right\} \qquad \text{Eqn. (2.4)}$$

$$\varepsilon_{ccu} \leq 0.01 \qquad \text{Eqn. (2.5)}$$

$f_{cc}'$ = Maximum confined concrete compressive strength; $f_c'$ = Unconfined cylinder compressive strength of concrete; $\psi_f$ = 0.95; $\kappa_a$ = Efficiency factor accounts for the geometry of the section; $f_l$ = Maximum confinement pressure; $n$ = Number of layer of FRP; $E_f$ = Young's modulus of FRP; $t_f$ = Thickness of FRP wrapping; $\varepsilon_{fe}$ = Effective strain level in the FRP at failure; $D$ = Diameter of circular cross-section; $\kappa_\varepsilon$ = FRP strain efficiency factor; $\varepsilon_{fu}$ = Ultimate strain of FRP, $\varepsilon_{ccu}$ = Maximum compressive strain in FRP confined concrete; $\varepsilon_c'$ = Unconfined concrete compressive strain; $\kappa_b$ = Efficiency factor.

In case of combined axial compression and bending the strength enhancement is predicted by using the same equations given above, in the members where eccentricity is less than or equal to 0.1 times effective depth of the section (ACI 440.2R-2008).

**Lam and Teng (2003a,b)** developed a design oriented stress-strain model for fiber reinforced polymer (FRP) confined concrete based on certain assumptions. Numerical relationships has been proposed for maximum confined compressive strength, ultimate axial strain, lateral confining pressure etc. for the concrete confined with FRP.

**Toutanji et al. (2010)** carried out analytical study on non-circular columns wrapped with CFRP (continued previous study done by Matthys et al. on circular columns). They evaluated the ultimate axial strength and the entire stress-strain response of FRP confined concrete based on previously developed models by many researchers They concluded that the model developed by Lam and Teng performed best among all the evaluated models in terms of shape and critical values.

**Chaallal and Shahawy (2000)** concluded that the rectangular sections confined laterally using FRP were not as effective as circular sections of same cross-sectional area. This was due to the higher stress concentration found at the corners and the non-uniformity in confinement.

# CHAPTER 3 EVALUATION OF STRENGTH AND DUCTILITY PARAMETERS

This chapter deals with the evaluation of strength and ductility parameters in terms of P-M interaction, moment-curvature, and moment-rotation. The model used for the design of FRP systems to confined reinforced concrete members is also discussed in this chapter. One of the important input data that is required to estimate the strength and ductility of any section is the peak strain of the materials. The peak strain data for the materials of existing structure can be selected from IS 456 (2000), plain and reinforced concrete–code of practice. But for members wrapped by FRP laminates, it is prime importance that the peak strain value of the confined concrete shall be chosen in such a way that the member behaviour is closely predicted, as the failure of the FRP wrapped members are sudden if they are governed by the rupture of FRP itself.

## 3.1 Estimation of peak strain of FRP wrapped members

Many researchers have already proved that the peak strain capacity of the concrete materials increases due to confinement actions of the FRP on the member. Many such mathematical models are also provided by these authors in research publications. However, for practical application of the particular technique, a reliable estimate of the quantity to be used in design calculations has not been provided.

ACI 440.2R-08; Guide for the design and construction of externally bonded FRP system for strengthening concrete structures, provides guidelines for designing retrofitting solution using FRP. This report provides recommendation for peak strain to be used in design calculations. The peak strain value of 0.004 is recommended for completely wrapped members with FRP in shear strengthening applications, which seemed to be too conservative in lieu of the reported values of peak strain in research publications. The peak strain value of 0.01 is recommended for axially loaded members to prevent excessive cracking and the resulting loss of concrete integrity (ACI 440.2R 2008).

Experimental results shows that with increase in strain, the internal integrity of the concrete in the member diminishes and causes excessive cracking. If the member is unwrapped at this stage, the residual strength of the concrete becomes very less than the peak strength of the concrete even in unconfined state. So if very high peak strain is considered for FRP wrapping, it could affect the performance of the structure, due to sudden failure mode of FRP wrapped members.

### 3.1.1 Material and constitutive model

In this study, the stress-strain model as predicted by Lam and Teng (2003a,b) for FRP-confined concrete has been adopted.

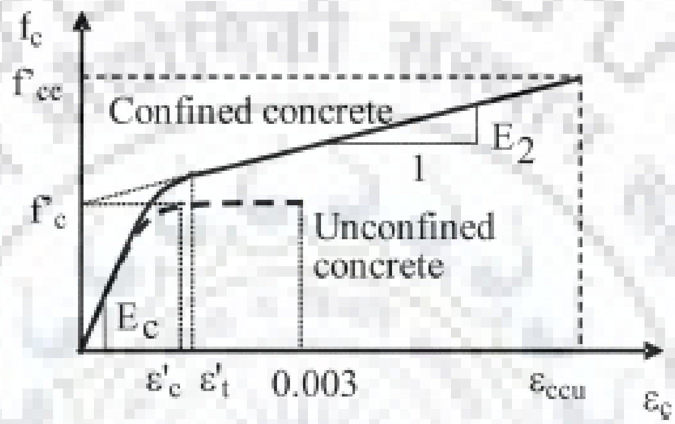



*Figure 3.1 Lam and Teng's stress-strain model for FRP-confined concrete (Lam and Teng 2003b)*

The value of maximum confined concrete compressive strength and maximum confinement pressure on the members are calculated by considering environmental factors, efficiency factors and an additional reduction factor of 0.95 (ACI 440.2R 2008).

$$f_{cc}' = f_c' + \psi_f 3.3 \kappa_a f_l \qquad \text{Eqn. (3.1)}$$

$$f_l = \frac{2nE_f t_f \varepsilon_{fe}}{D} \qquad \text{Eqn. (3.2)}$$

$$\varepsilon_{fe} = \kappa_\varepsilon \varepsilon_{fu} \qquad \text{Eqn. (3.3)}$$

$f_{cc}'$ = Maximum confined concrete compressive strength; $f_c'$ = Unconfined cylinder compressive strength of concrete; $\psi_f$ = 0.95; $\kappa_a$ = Efficiency factor accounts for the geometry of the section; $f_l$ = Maximum confinement pressure; $n$ = Number of layer of FRP; $E_f$ = Young's modulus of FRP; $t_f$ = Thickness of FRP wrapping; $\varepsilon_{fe}$ = Effective strain level in the FRP at failure; $D$ = Diameter of circular cross-section; $\kappa_{\varepsilon}$ = FRP strain efficiency factor; $\varepsilon_{fu}$ = Ultimate strain of FRP.

The peak strain value of FRP wrapped members are taken as predicted by mathematical model of Lam and Teng (2003a,b).

$$\varepsilon_{ccu} = \varepsilon_c'\left\{1.50 + 12\kappa_b\left(\frac{f_l}{f_c'}\right)\left(\frac{\varepsilon_{fe}}{\varepsilon_c'}\right)^{0.45}\right\} \quad \text{Eqn. (3.4)}$$

$$\varepsilon_{ccu} \le 0.01 \quad \text{Eqn. (3.5)}$$

$\varepsilon_{ccu}$ = Maximum compressive strain in FRP confined concrete; $\varepsilon_c'$ = Unconfined concrete compressive strain; $\kappa_b$ = Efficiency factor.

The properties of the CFRP composite material used in this analysis are as follows: fiber thickness $t_f$ = 0.165 mm per layer, ultimate tensile strength $f_{fu}$ = 3700 MPa, tensile modulus $E_f$ = 227500 MPa, ultimate tensile strain $\varepsilon_{fu}$ = 0.017 (Bank 2006).

### 3.1.2 Determination of shear capacity of an FRP strengthened member

FRP strengthening systems can be used to increase the shear capacity of concrete beams, columns, and walls. FRP strengthening systems are applied to the webs of beams and function in a fashion similar to that of internal steel shear reinforcements such as stirrups, hoops, or ties.

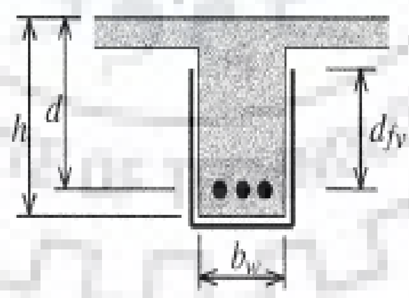



*Figure 3.2 Geometry of beam showing FRP wrapping (U-wrap)*

The equations used for determination of shear capacity of an FRP strengthened member is mentioned below (ACI 440.2R 2008).

The nominal shear capacity of FRP strengthened beam is given as

$$V_n = V_c + V_s + \psi_f V_f \qquad \text{Eqn. (3.6)}$$

where, $V_c$ = Existing shear capacity of the concrete, $V_s$ = Shear capacity of the existing steel shear reinforcement, $V_f$ = Shear capacity of the FRP strengthening system, $\psi_f$ = Additional capacity reduction factor.

$$V_f = 2nt_f f_{fe} d_f \qquad \text{Eqn. (3.7)}$$

$n$ = Number of layer of FRP; $E_f$ = Young's modulus of FRP; $t_f$ = Thickness of FRP wrapping; $\varepsilon_{fe}$ = Effective strain level in the FRP at failure

$$f_{fe} = E_f \varepsilon_{fe} \qquad \text{Eqn. (3.8)}$$

$$\varepsilon_{fe} = \kappa_v \varepsilon_{fu} \qquad \text{Eqn. (3.9)}$$

where, $\kappa_v$ = Shear bond reduction coefficient, which is given by ACI 440.2R (2008) as

$$\kappa_v = \frac{k_1 k_2 L_e}{11900 \varepsilon_{fu}} \leq 0.75 \qquad \text{Eqn. (3.10)}$$

$$L_e = \frac{23300}{(n_f t_f E_f)^{0.58}} \qquad \text{Eqn. (3.11)}$$

$$K_1 = \left( \frac{f_c'}{27} \right)^{2/3} \qquad \text{Eqn. (3.12)}$$

$$K_2 = \frac{d_{fv} - L_e}{d_{fv}} \qquad \text{Eqn. (3.13)}$$

where, $Le$ = Active bond length, $f_c'$ = Unconfined cylinder compressive strength of concrete, $K_1$ and $K_2$ = Bond reduction coefficient, $d_{fv}$ = Effective depth of FRP strengthening system.

### 3.2 Determination of P-M interaction curve

Axial load-Moment (P-M) interaction diagram for a reinforced concrete section is developed by satisfying strain compatibility. It is generated by dividing the section into a number of finite strips and estimating the response of every finite strips of the section for given neutral axis depth and stress-strain profile of the materials constituting the section. The number of finite strips that the section is divided into, is based on sensitivity analysis of the section. The stress of the section for a given neutral axis depth is evaluated at the centroid of the section and corresponding force generated due to the strip is calculated. After determining the stress values a numerical integration is performed over the depth of the member so as to get the overall response of the section for the particular neutral axis depth. The procedure is followed for a number of neutral axis depths so that the entire failure envelope of the column section can be found out.

P-M diagrams for FRP-confined concrete is determined by satisfying strain compatibility and force equilibrium using the model proposed by Lam and Teng (2003a,b) (ACI 440.2R 2008).

### 3.3 Determination of Moment-Curvature diagram

The moment curvature (M-$\phi$) diagram of the section is determined to check the capability of the section to rotate or deform under the applied loading. The determination of moment-curvature relationship is approximately similar with that of deriving the axial load-moment interaction diagram, the only difference being in that, in case of P-M interaction, the strain at ultimate compression is known while in case of M-$\phi$ diagram the neutral axis depth is to be evaluated in terms of the axial loading on the member and an assumed value of the curvature for the section.

### 3.4 Determination of Moment-Rotation diagram

The moment-rotation diagram depicts the relation between the moment and the plastic rotation capacity of the member. Once the moment-curvature relation of the section is known, the yield and ultimate moment values and corresponding curvatures are calculated.

To evaluate the plastic rotation capacity of the member, the plastic hinge length of the member is taken as proposed by Pauley and Priestley. The plastic rotation of the member is determined as: (Paulay and Priestley 1993)

$$\theta_p = \phi_p \times L_p \qquad \text{Eqn. (3.14)}$$

Where $\phi_p$ = Plastic curvature of the section, which is obtained by using following equation

$$\phi_p = \left(\phi - \phi_y\right) \qquad \text{Eqn. (3.15)}$$

and, $L_P$ = Length of plastic hinge, which is defined as:

$$L_P = 0.08L + 0.022 d_b f_y \qquad \text{Eqn. (3.16)}$$

Where, L is the member length, $d_b$ = bar diameter, and $f_y$ = yield strength of the reinforcing bar.

# CHAPTER 4 WORK PLAN AND OBJECTIVE

## 4.1 Objective

It is evident that many existing buildings have yet to be retrofitted in order to stay reasonably undamaged and safe during severe earthquake than those they have been designed for. In addition to changes in seismic hazard levels, design methods, and serviceability requirements are amongst other reasons for retrofitting a deficient building subjected to an ordinary earthquake.

The primary aim of this study is to retrofit an existing reinforced concrete (RC) multi-storey building subjected to axial and lateral loads. And, also to evaluate the effect of retrofitting (FRP Jacketing) on strength and ductility parameter of RC building subjected to axial and lateral loads.

## 4.2 Methodology

The methodology for the proposed project includes the assessment of an existing building (G+6) which is to be retrofitted for seismic demand. The strength (in terms of axial load-moment interaction diagrams) and ductility (moment-curvature and moment-rotation diagram) parameters are compared at member level of the existing and retrofitted members. Nonlinear static analysis of the building is to be done before and after retrofitting so as to quantify the improvement in the seismic load carrying capacity of the building.

The local retrofitting technique chosen for this study is FRP wrapping of the deficient members. With this techniques the behavior of non-ductile members can be greatly enhanced, providing benefits to the global performance of the building frames and reducing the potential for collapse. At first, the local retrofitting techniques are applied on the members of building so as to safeguard the building against failure in seismic loading case. The strength and ductility parameters of the members will be compared in each of the local retrofitting technique applied.

## 4.3 Design and description of the Building

The structure considered is a 7-storey RC building representing a mid-rise building. The height is assumed to be equal to 3.30 m for all stories. The bay width for first and third bay along X- direction is 5.0 m and that for second bay is 3.0 m, while the bay width along Y-direction is 5.0 m. The design live load on the floors 4.0 kN/m$^2$ and on roof 1.5 kN/m$^2$ are assigned as per IS 875-2 (1987). In addition, the Grade of concrete and steel used are M25 and Fe415 respectively. The cross-section of columns and beams are (400 × 400) mm and (300 × 400) mm respectively. The cross-sectional details of columns and beams are plotted in Figure 4.1. The reinforcement details of the columns and beams are tabulated in Table 4.1.

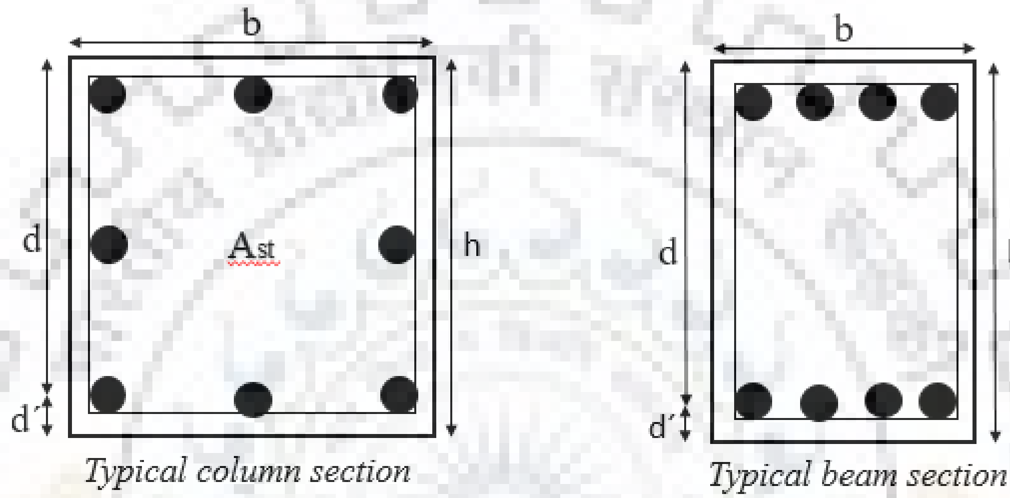



*Figure 4.1 Sectional details of the column and beam in existing building*

*Table 4.1 Reinforcement details of the beam and column*

| **Section** | **b** (mm) | **h** (mm) | **d** (mm) | **d´** (mm) | **Ast** | **As** | **As´** | **Transverse steel spacing (mm)** |
|---|---|---|---|---|---|---|---|---|
| Beam | 300 | 400 | 360 | 40 | - | 4-20Ø | 4-20Ø | 200 |
| Column | 400 | 400 | 360 | 40 | 8-20Ø | - | - | 200 |

The self-weight of the members is automatically assigned using a load case in SAP2000. The dead load due to slab weight, floor finish were assigned using a separate load case. The seismic weight of the building was considered as dead load plus 50% of live load as per Table 10 of IS 1893-1 (2016). The effective stiffness's of the beams and columns were taken as per Table 2 of IS 15988 (2013)

The nodes at each floor level is assigned rigid diaphragm constraint so as to simulate the effect of slabs at each floor levels. This constraint enables the nodes at each floor level to move together as a planar diaphragm. The connections at the beam column junctions were assumed to be semi-rigid and rigid offset length of 0.5 is taken.

### 4.3.1 Existing configuration

The existing configuration of the building is as shown in the Figure 4.2 – 4.4.

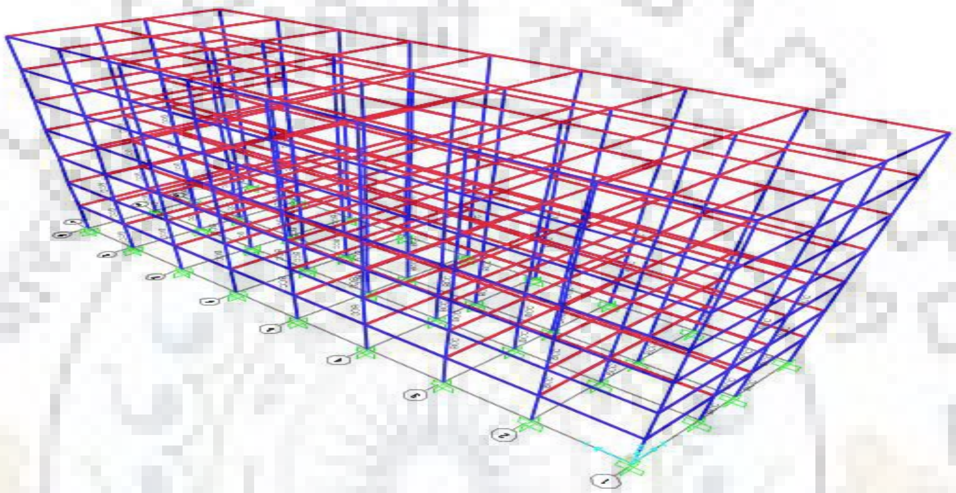


*Figure 4.2 Isometric view of the existing building*

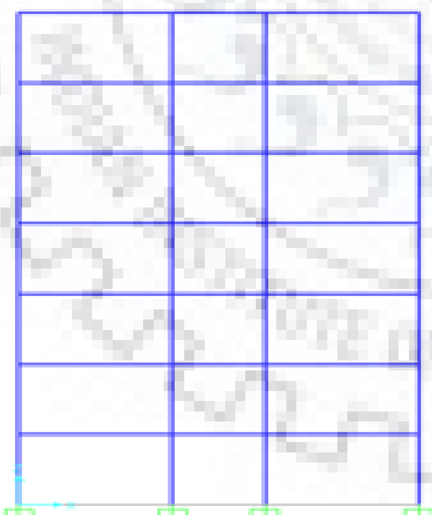


*Figure 4.3 Elevation of the existing building in X-Z plane*

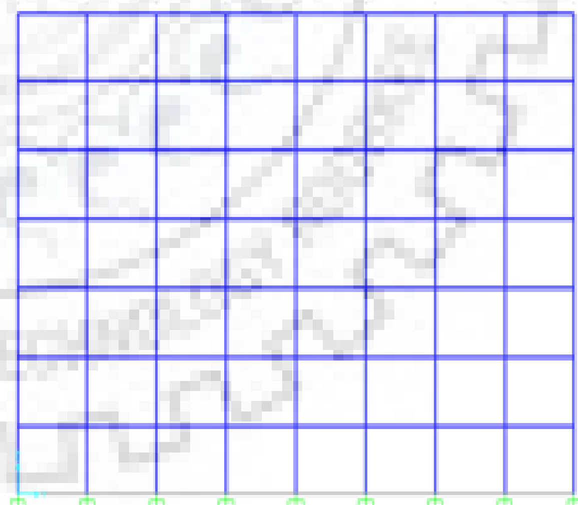


*Figure 4.4 Elevation of the existing building in Y-Z plane*

## 4.4 Modelling and Analysis

A 3d model of the building has been analysed for gravity loading using SAP2000, a general purpose finite element analysis software package. The demand points of the columns of the existing building such evaluated are plotted against the capacity curve of the respective member. The capacity curve of the member is evaluated in terms of P-M (axial load- bending moment) interaction curve. Thus the deficiency of the building, if any, is found out.

The building is then analysed for seismic loading in Zone-V, resting on medium or stiff soil. The demand points of the columns (for seismic loading) of the existing building such evaluated are plotted against the capacity curve of the respective member. If the building is found to be deficient in gravity/seismic loading, suitable retrofitting (local or global) measures is proposed. If the local retrofitting techniques is inadequate to enhance the seismic capacity of the building, a combination of local and global retrofitting technique will be proposed.

For determination of demand point on the building, the following load combinations were considered.

*Table 4.2 Load combinations for assessment of existing building (IS 1893-1 2016)*

| Load case type | Dead load | Live load | Earthquake load | |
|---|---|---|---|---|
| | | | $EQ_X$ | $EQ_Y$ |
| DL | 1.50 | 0.00 | 0.00 | 0.00 |
| DL + LL | 1.50 | 1.50 | 0.00 | 0.00 |
| DL+ LL+ $EQ_X$ | 1.20 | 1.20 | (+/-) 1.20 | 0.00 |
| DL+ LL+ $EQ_Y$ | 1.20 | 1.20 | 0.00 | (+/-) 1.20 |
| DL+ $EQ_X$ | 1.50 | 0.00 | (+/-) 1.50 | 0.00 |
| DL+ $EQ_Y$ | 1.50 | 0.00 | 0.00 | (+/-) 1.50 |
| DL+ $EQ_X$ | 0.90 | 0.00 | (+/-) 1.50 | 0.00 |
| DL+ $EQ_Y$ | 0.90 | 0.00 | 0.00 | (+/-) 1.50 |

The existing strength and ductility parameters of the members are found out using recommendations provided in IS 456 (2000), Plain and Reinforced Concrete–Code of Practice. The seismic demand of the building is determined using CQC rule of response spectrum method as recommended in IS 1893-1 (2016), Criteria for Earthquake Resistant Design of Structures-General Provisions and Buildings.

During the response spectrum analysis of the structure, the calculated base shear by SAP 2000 was compared to the base shear obtained from equivalent static method, and suitable base shear correction factor applied to the load case as recommended in clause no. 7.7.3 of IS 1893 (Part 1):2016. The design base shear obtained from linear analyses is shown in Table 4.3.

*Table 4.3 Design base shear of the existing building*

| **Direction** | **Design base shear (kN)** | |
|---|---|---|
| | Response spectrum method ($V_B$) | Equivalent static method ($V_B'$) |
| X-direction | 1408 | 1760 |
| Y-direction | 1416 | 1771 |
| Z-direction | 5278 | 5572 |

Since the design base shear estimated from response spectrum method is less than that obtained from equivalent static method, hence base shear multiplying factor ($V_B'/V_B$) 1.24, 1.25 and 1.25 is applied to the response spectrum load case along X, Y and Z direction respectively.

While applying response spectrum load on the existing building, a lateral load modification factor is also applied so as to incorporate the remaining reduced design life of the structure as per IS 15988 (2013).

Design useful life, $T_{design}$ = 100 Years

Remaining useful life, $T_{rem}$ = (100-25) = 75 Years

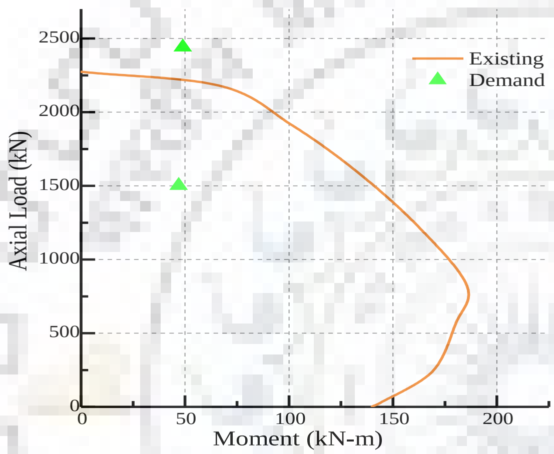
Lateral load modification factor, U= $(T_{rem}/T_{design})^{0.5}$ = **0.87**

# CHAPTER 5 RESULTS AND DISCUSSION

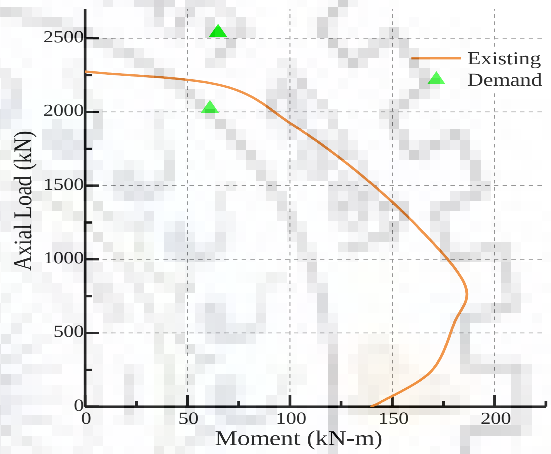
The results of linear and non-linear analysis of the building is presented in this chapter.

## 5.1 Linear analysis for Gravity loading case

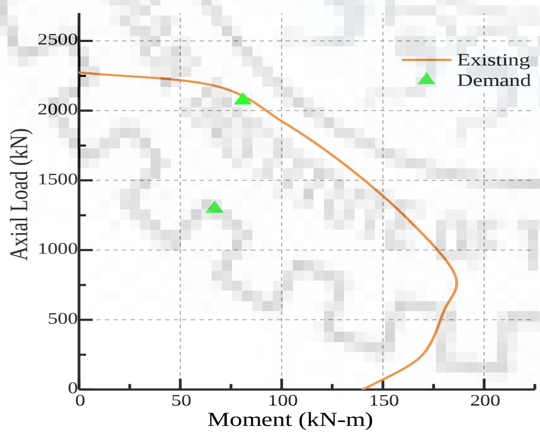
After analysing the building for gravity loading case, the interior columns at the ground and first storey and exterior columns at ground storey are found to be slightly deficient. The existing capacities of the member along with demand points in gravity load case at various floor levels are shown in Figure 5.1.



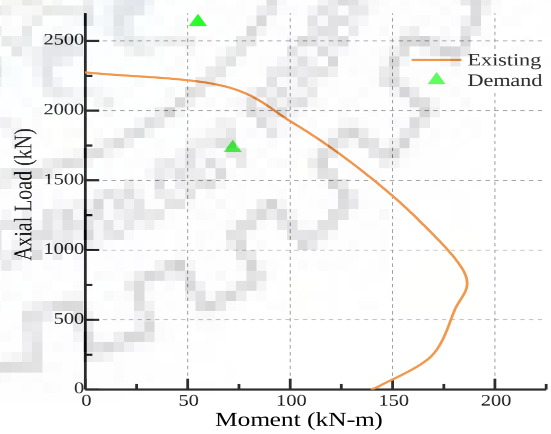

*(a) Ground storey exterior column for gravity loading*



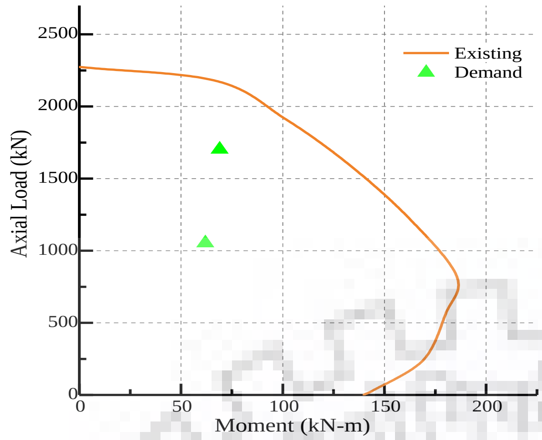

*(b) Ground storey interior column for gravity loading*



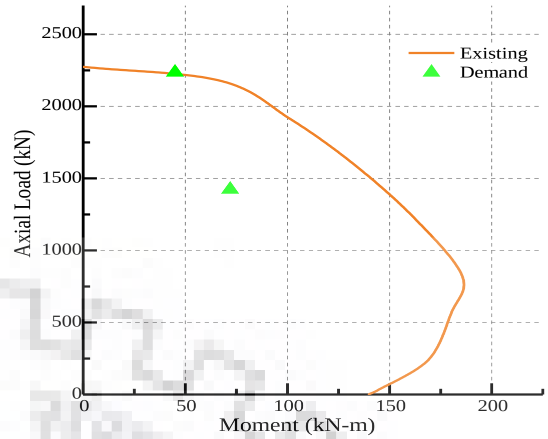

*(c) First storey exterior column for gravity loading*



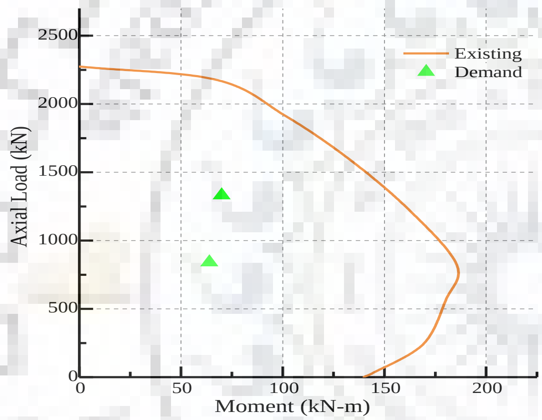

*(d) First storey interior column for gravity loading*



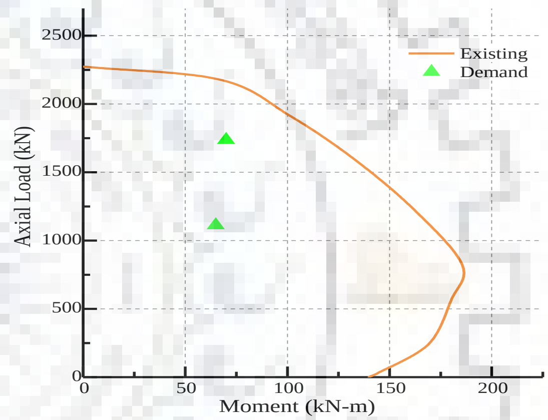

*(e) Second storey exterior column for gravity loading*



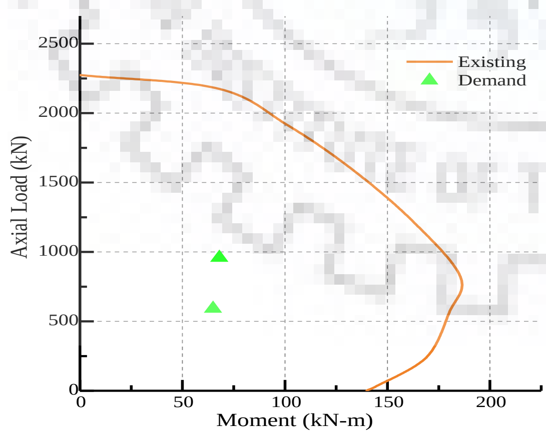

*(f) Second storey interior column for gravity loading*



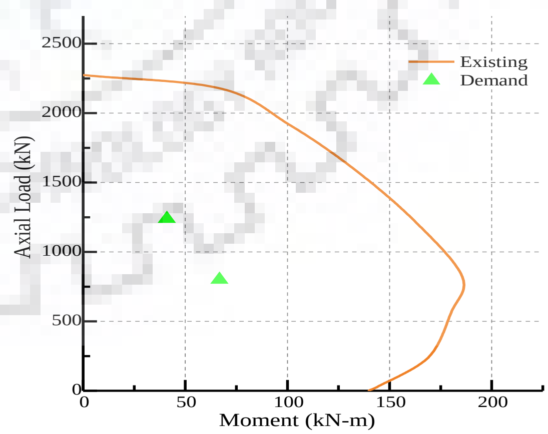

*(g) Third storey exterior column for gravity loading*



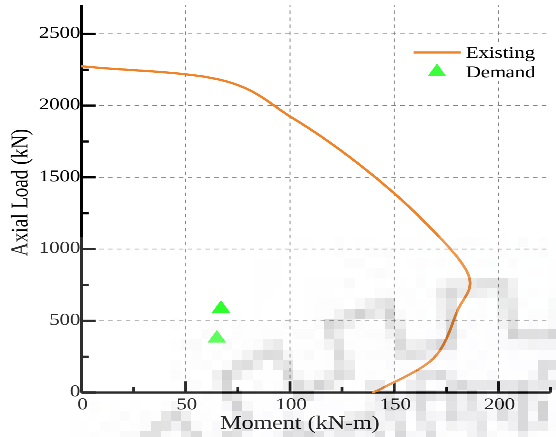

*(h) Third storey interior column for gravity loading*



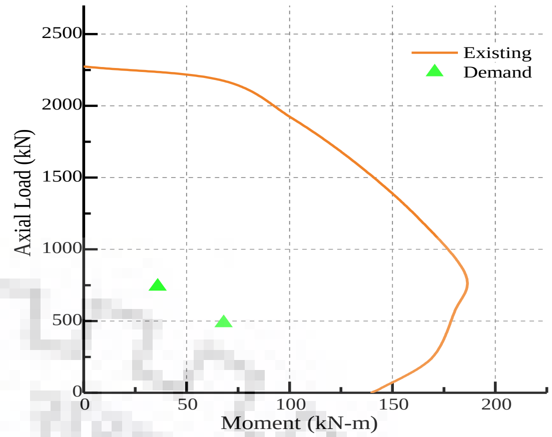

*(i) Fourth storey exterior column for gravity loading*



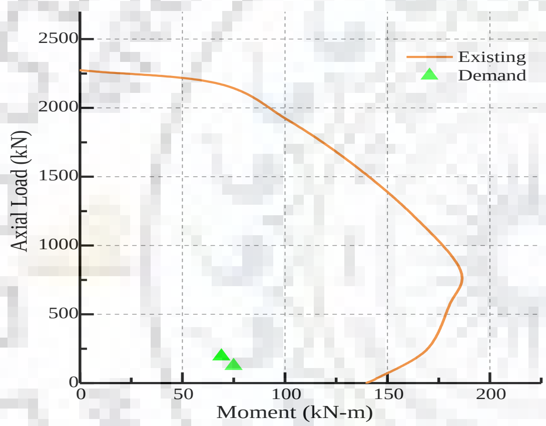

*(j) Fourth storey interior column for gravity loading*



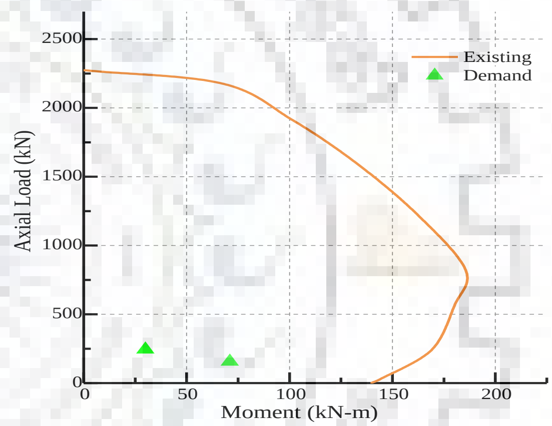

*(k) Fifth storey exterior column for gravity loading*




*(l) Fifth storey interior column for gravity loading*




*(m) Sixth storey exterior column for gravity loading*

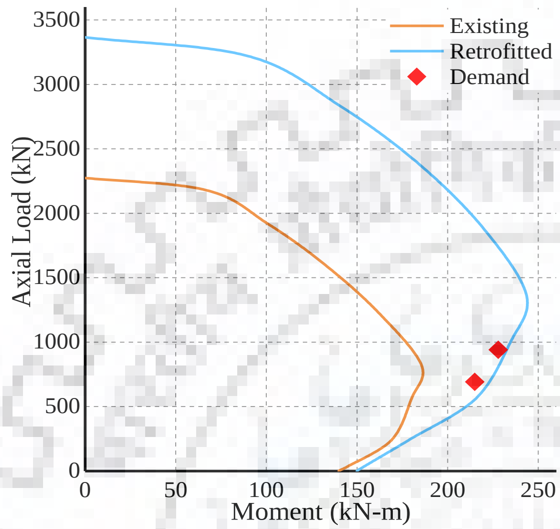



*(n) Sixth storey interior column for gravity loading*

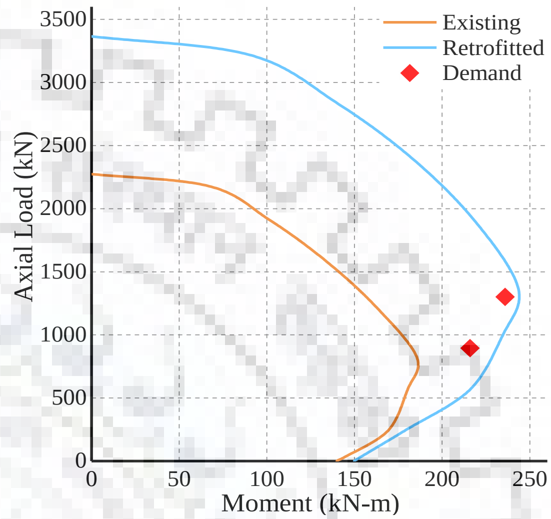
*Figure 5.1 Axial load-Moment interaction diagram for dead load (DL) and live load (LL) case*

## 5.2 Linear analysis for Seismic loading case with CFRP jacketing

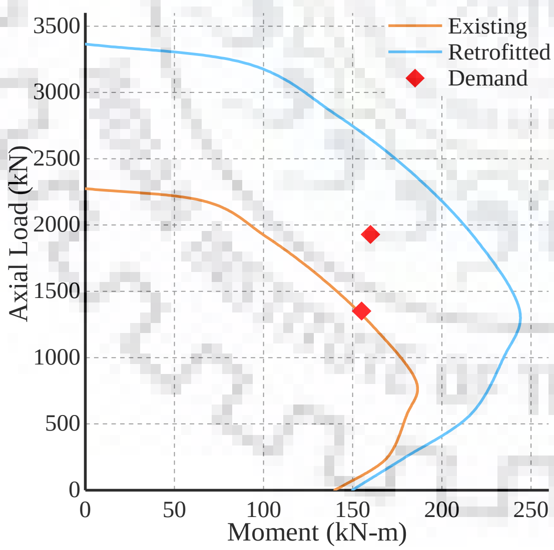
After analysing the building for gravity loading case, the building is analysed for seismic loading. Both the exterior and the interior columns of the building (ground, first and second storey) are found to be deficient. The interior columns at third storey are also found to be deficient under seismic loading. The interior beams up to fourth storey are also failing in seismic loading. The failure members are now wrapped with 1, 3 and 5 layers of CFRP strips respectively. The retrofitted structure is analysed successively, and found to be safe after retrofitting with 5 layers of CFRP strips. The existing and retrofitted capacities of the members along with the demand points in seismic loading cases at various floor levels are shown in Figure 5.2.



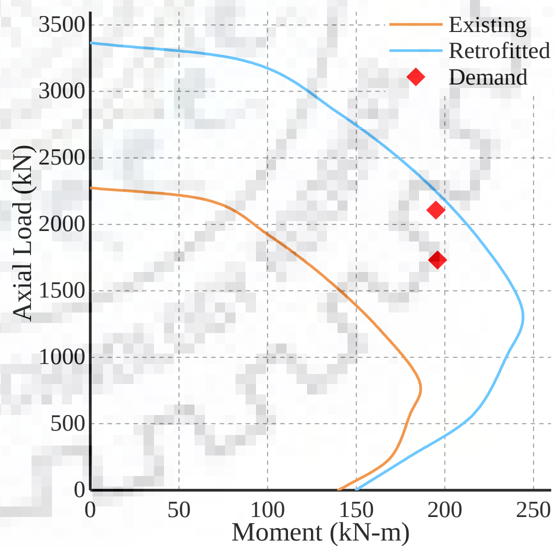

*(a) Ground storey exterior column for seismic loading case*



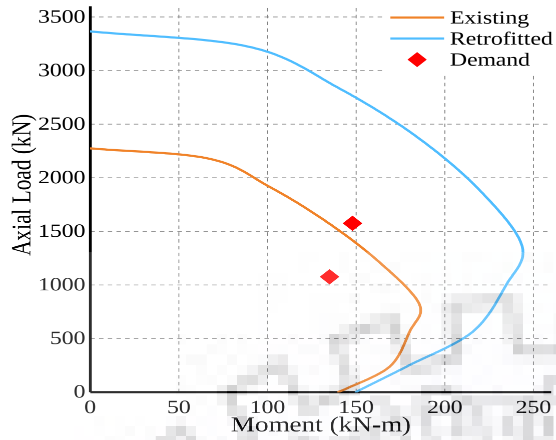

*(b) Ground storey interior column for seismic loading case*



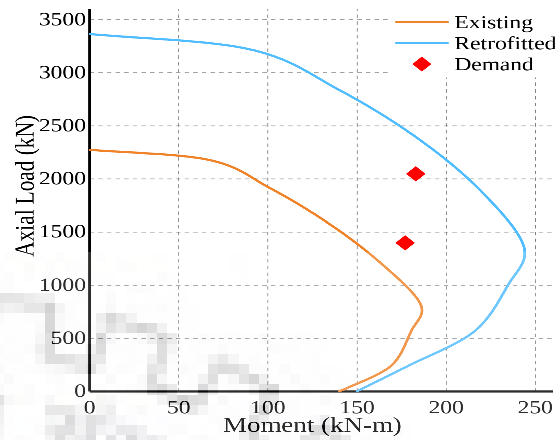

*(c) First storey exterior column for seismic loading case*



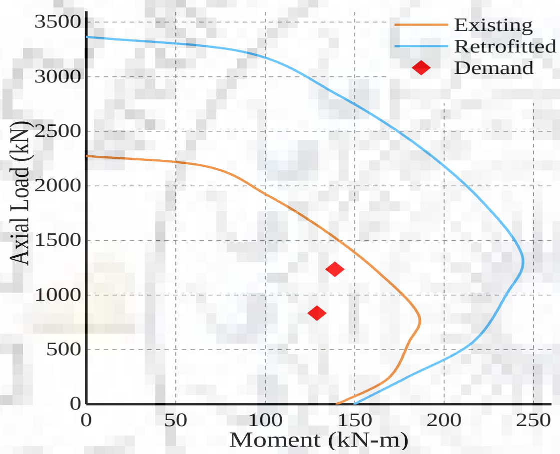

*(d) First storey interior column for seismic loading case*



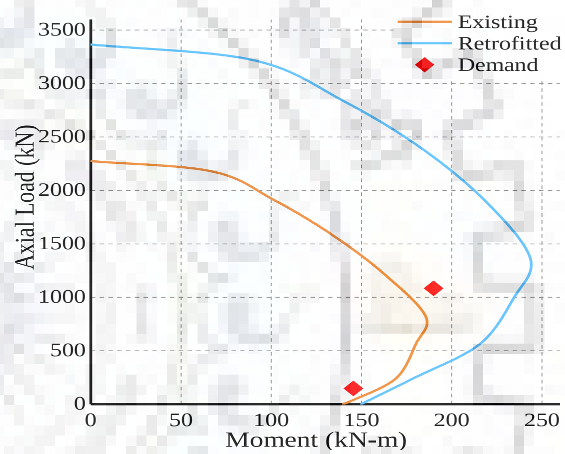

*(e) Second storey exterior column for seismic loading case*



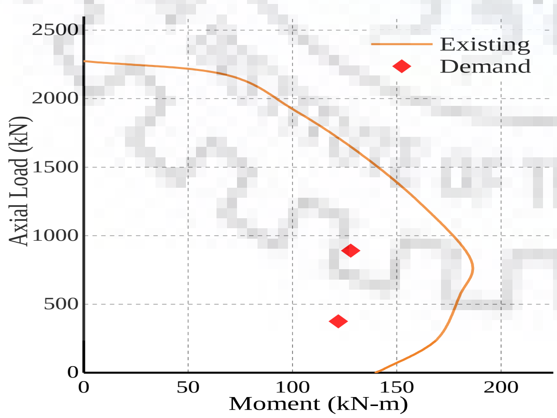

*(f) Second storey interior column for seismic loading case*



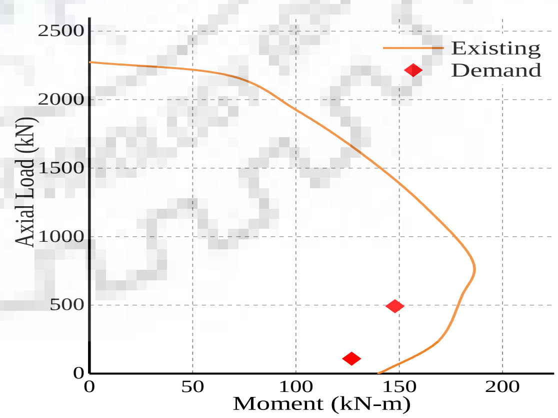

*(g) Third storey exterior column for seismic loading case*



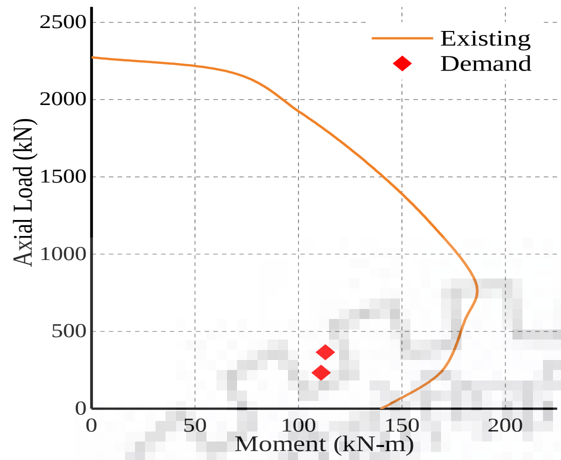

*(h) Third storey interior column for seismic loading case*



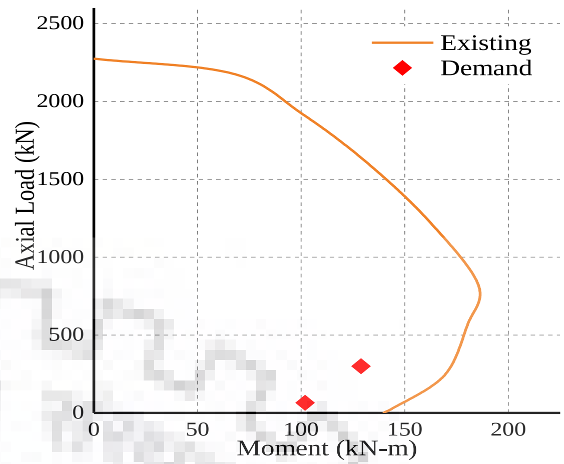

*(i) Fourth storey exterior column for seismic loading case*



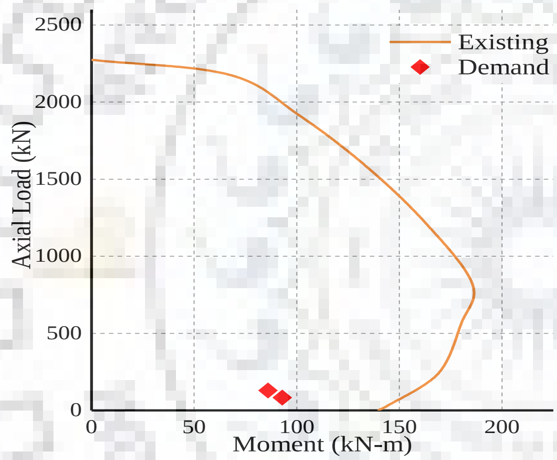

*(j) Fourth storey interior column for seismic loading case*



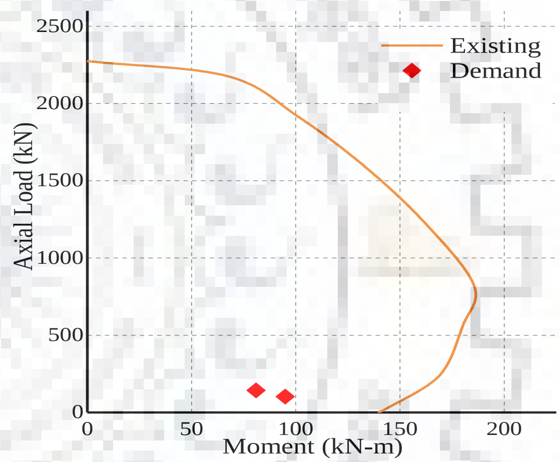

*(k) Fifth storey exterior column for seismic loading case*




*(l) Fifth storey interior column for seismic loading case*




*(m) Sixth storey exterior column for seismic loading case*




*(n) Sixth storey interior column for seismic loading case*

*Figure 5.2 Axial load- Moment interaction diagram for seismic loading case*

After analysing the building for seismic loading case, the beams at various floor levels are also found to be deficient in seismic loading case. The interior beams along transverse direction up to fourth storey are lacking in both shear and flexure demand. While the exterior beams are found to be safe in both shear and flexure.

In beam where an integral slab makes it difficult to completely wrap the member, the strength can be enhanced by wrapping the CFRP strips around three sides of the member (U-wrap) or bonding to two opposite sides of the member. In this study the deficient beams are wrapped with CFRP strips on three sides, and the enhancement in its strength is evaluated. The shear capacity of the interior and exterior beams before and after retrofitting along with demand is shown in Figure 5.3 and Figure 5.4 respectively:

*Table 5.1 Shear demand-capacity ratio for interior beams along transverse (X) direction*

| Floor level | Shear demand (kN) | Existing shear capacity (kN) | Demand-capacity (D/C) ratio | Retrofitted shear capacity (kN) | Modified demand-capacity (D/C) ratio |
|---|---|---|---|---|---|
| 1 | 140.69 | 110.09 | 1.27 | 160.00 | 0.87 |
| 2 | 158.60 | 110.09 | 1.44 | 160.00 | 0.99 |
| 3 | 149.63 | 110.09 | 1.36 | 160.00 | 0.93 |
| 4 | 130.30 | 110.09 | 1.18 | 160.00 | 0.81 |
| 5 | 99.83 | 110.09 | 0.90 | 110.09 | 0.90 |
| 6 | 59.59 | 110.09 | 0.54 | 110.09 | 0.54 |
| 7 | 22.27 | 110.09 | 0.20 | 110.09 | 0.20 |

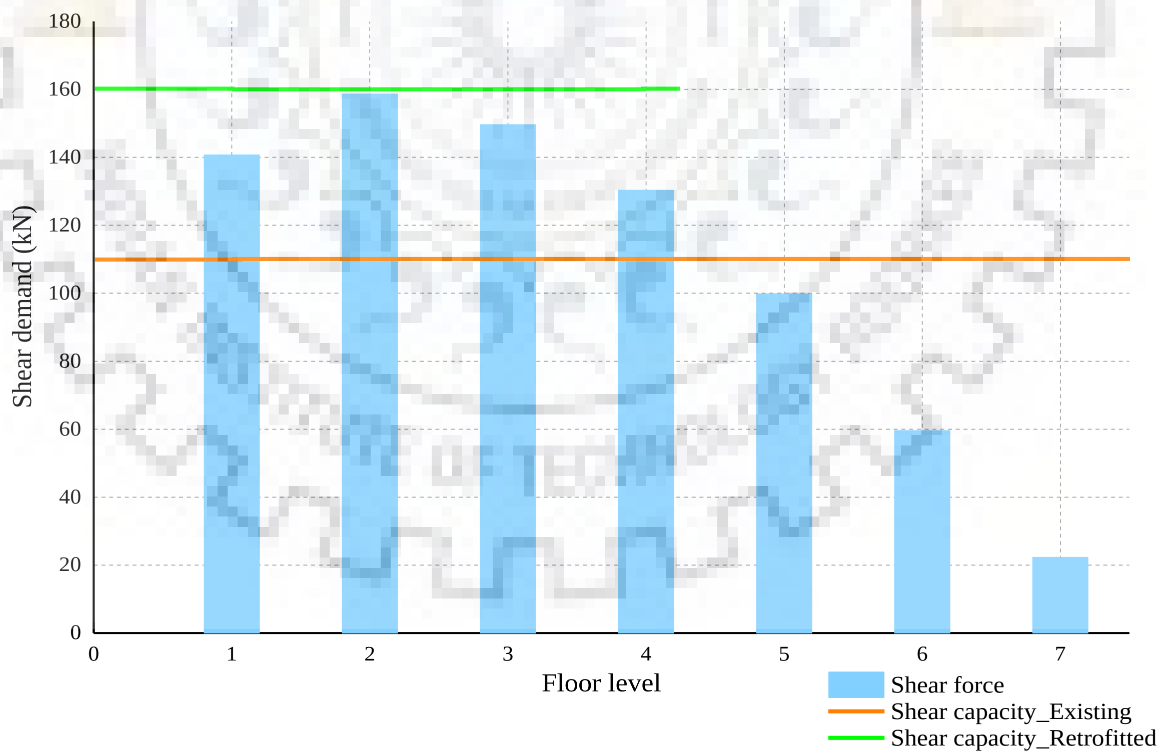



*Figure 5.3 Shear force variation for interior beams at various floor level*

Table 5.1 shows the interior beams of first, second, third and fourth floor are deficient in shear. After wrapping with CFRP the shear capacity of the beams are increased by 45%. Further the demand-capacity ratio of each member is calculated and found to be less than one.

*Table 5.2 Shear demand-capacity ratio for exterior beams along transverse (X) direction*

| | Shear demand (kN) | Existing shear capacity (kN) | Demand-capacity (D/C) ratio |
|---|---|---|---|
| 1 | 66.55 | 110.09 | 0.60 |
| 2 | 73.96 | 110.09 | 0.67 |
| 3 | 72.12 | 110.09 | 0.65 |
| 4 | 65.51 | 110.09 | 0.59 |
| 5 | 54.22 | 110.09 | 0.49 |
| 6 | 38.57 | 110.09 | 0.35 |
| 7 | 24.27 | 110.09 | 0.22 |

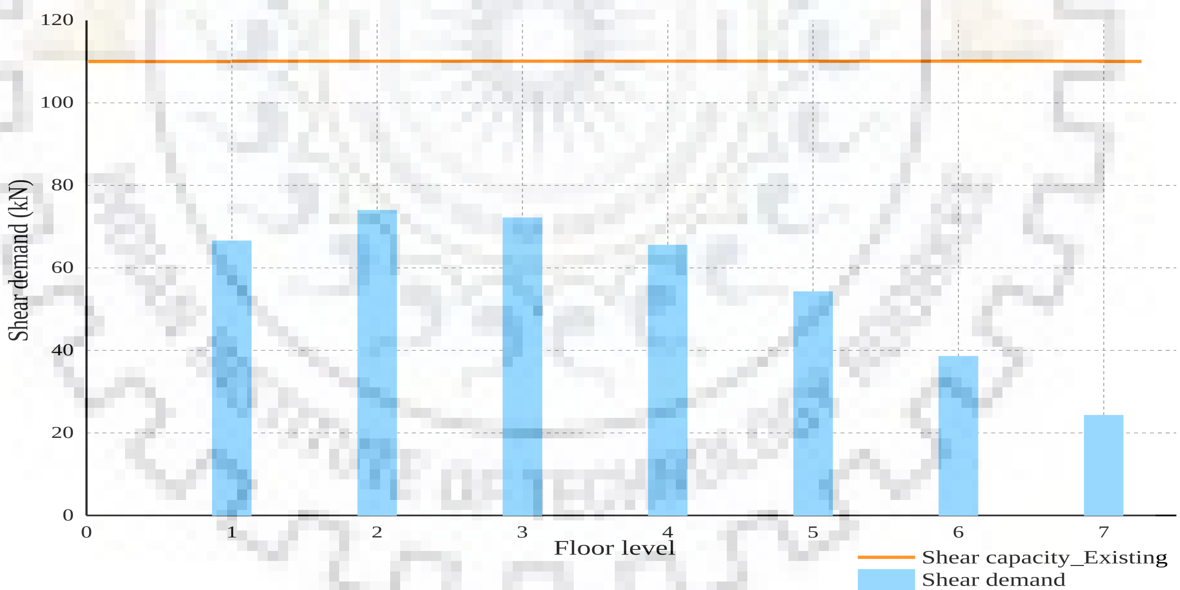



*Figure 5.4 Shear  force variation for exterior beams at various floor levels*

Table 5.2 shows that, the shear demand-capacity ratio of exterior beams are found to be less than one. Hence all the exterior beams are found to be safe in shear.

The flexure capacity of the beam before and after retrofitting along with demand is shown in the Figure 5.3 and Figure 5.4 respectively:

*Table 5.3 Flexure demand-capacity ratio for interior beams along transverse (X) direction*

| Floor level | Flexure demand (kN-m) | Existing flexure capacity (kN-m) | Demand-capacity (D/C) ratio | Retrofitted Flexure capacity (kN-m) | Modified demand capacity (D/C) ratio |
|---|---|---|---|---|---|
| 1 | 178.27 | 169.31 | 1.05 | 198.09 | 0.90 |
| 2 | 195.37 | 169.31 | 1.15 | 198.09 | 0.98 |
| 3 | 189.74 | 169.31 | 1.12 | 198.09 | 0.95 |
| 4 | 164.59 | 169.31 | 0.97 | 198.09 | 0.83 |
| 5 | 125.02 | 169.31 | 0.73 | 169.31 | 0.73 |
| 6 | 72.51 | 169.31 | 0.42 | 169.31 | 0.42 |
| 7 | 24.76 | 169.31 | 0.14 | 169.31 | 0.14 |

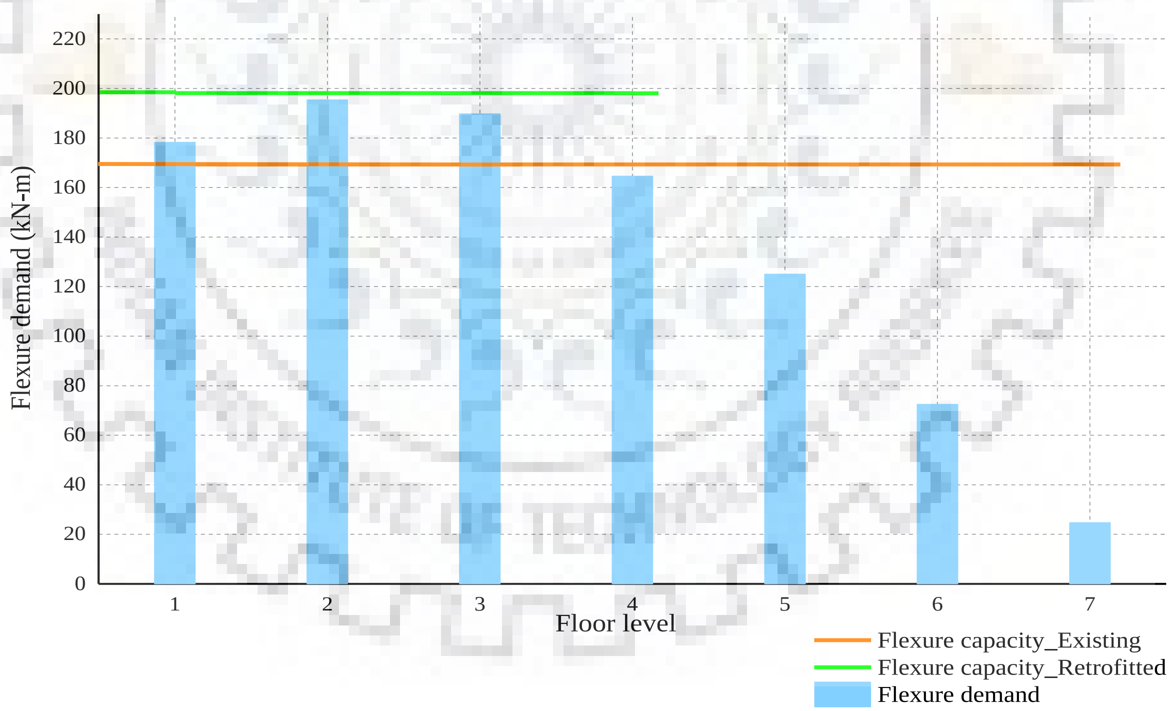



*Figure 5.5 Bending moment variation for interior beams at various floor level*

From Table 5.3, it can be concluded that the interior beams at first, second and third floor are found to be deficient in flexure also. After wrapping with CFRP, the flexure capacity of wrapped member is increased by 17% approximately. Further the demand-capacity ratio is calculated, and found to be less than one.

*Table 5.4 Flexure demand-capacity ratio for exterior beams along transverse (X) direction*

| Floor level | Flexure demand (kN-m) | Existing flexure capacity (kN-m) | Demand-capacity (D/C) ratio |
|---|---|---|---|
| 1 | 142.82 | 169.31 | 0.84 |
| 2 | 159.44 | 169.31 | 0.94 |
| 3 | 154.98 | 169.31 | 0.91 |
| 4 | 138.98 | 169.31 | 0.82 |
| 5 | 111.83 | 169.31 | 0.66 |
| 6 | 74.22 | 169.31 | 0.43 |
| 7 | 39.54 | 169.31 | 0.23 |

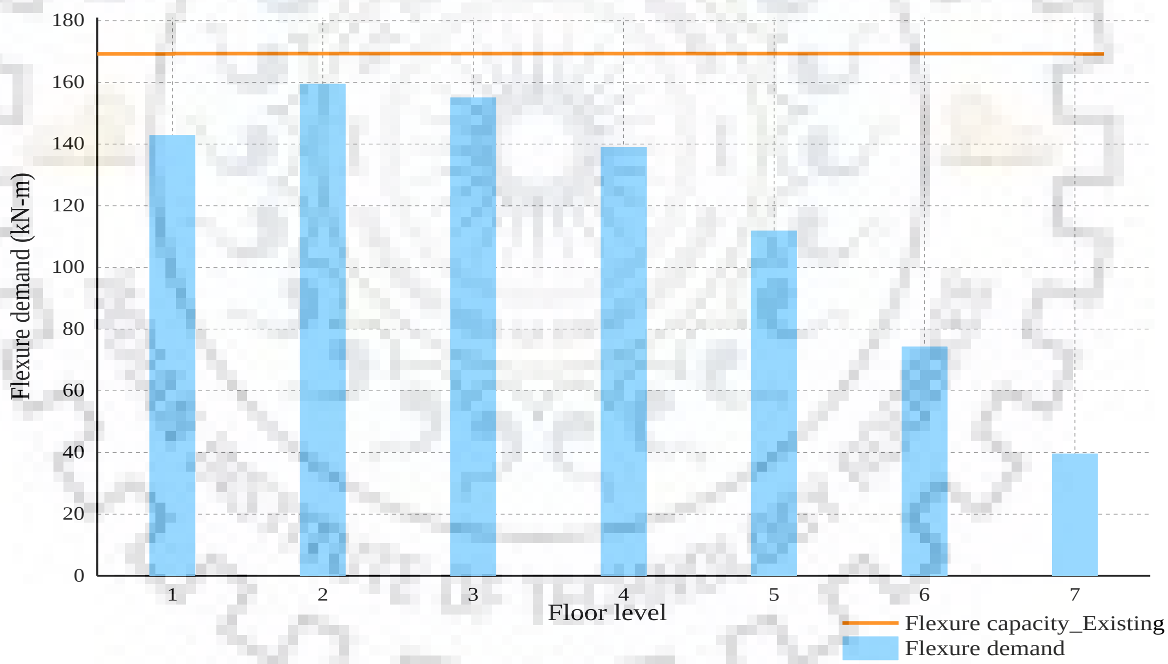



*Figure 5.6 Bending moment variation for exterior beams at various floor level*

Table 5.4 shows that, the flexure demand-capacity ratio of the exterior beams are less than one. Hence all exterior beams are found to be safe in flexure

### 5.3 Nonlinear static analysis of the existing building

Pushover analysis is basically a non-linear static analysis which is to be performed so as to estimate the structure's behavior under dynamic load conditions e.g. seismic load cases. This analysis method has been proven its effectiveness over the period of time in estimating the response under such extreme load cases. Though the accuracy of the result obtained from Time-history analysis are more, the static method has been performed so as to get the approximate results in much lesser time and computational cost.

Pushover analysis of the existing structure has been performed. The analysis has been done for both the direction of the building. The plastic hinges are auto assigned to beams and columns as per ASCE 41-13 in SAP2000, taking expected strengths of concrete and steel into account. The beams are assigned M3 moment and V2 shear hinges as it is expected to form hinges either in bending or in shear and the columns are assigned P-M2-M3 i.e. force-moment hinges. The pushover analysis reveals that the building fails by forming hinges in the beam first of the first storey and then in the columns. Most of the hinges formed in the beams and columns were in the collapse prevention limit. The local failure in the structure starts occurring before reaching its performance point.

The peak displacement and base shear force of the existing building is found to be 0.25 m and 4286.71 kN respectively along X- direction. In Y-direction, the peak displacement and base shear force is found to be 0.30 m and 4763.71 kN respectively.

***Pushover analysis results in X direction***

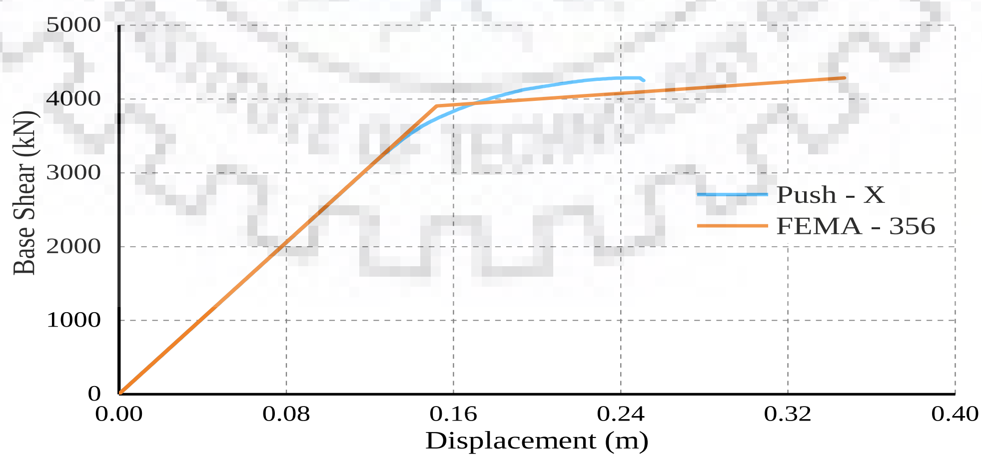



*Figure 5.7 Pushover curve of the existing building along transverse direction.*

*Table 5.5 Pushover analysis results along transverse direction.*

| | | | |
|---|---|---|---|
| Design base shear ($V_d$) | 1760.00 kN | | |
| Base shear at yield (Vy) | 3906.84 kN | Yield disp. (Δy) | 0.15 m |
| Base shear at failure | 4286.71 kN | Maximum disp. (Δmax) | 0.25 m |
| Overstrength factor (Vy/ $V_d$) | 2.22 | Ductility reduction factor (Rμ) | 1.64 |

***Pushover analysis results in Y direction***

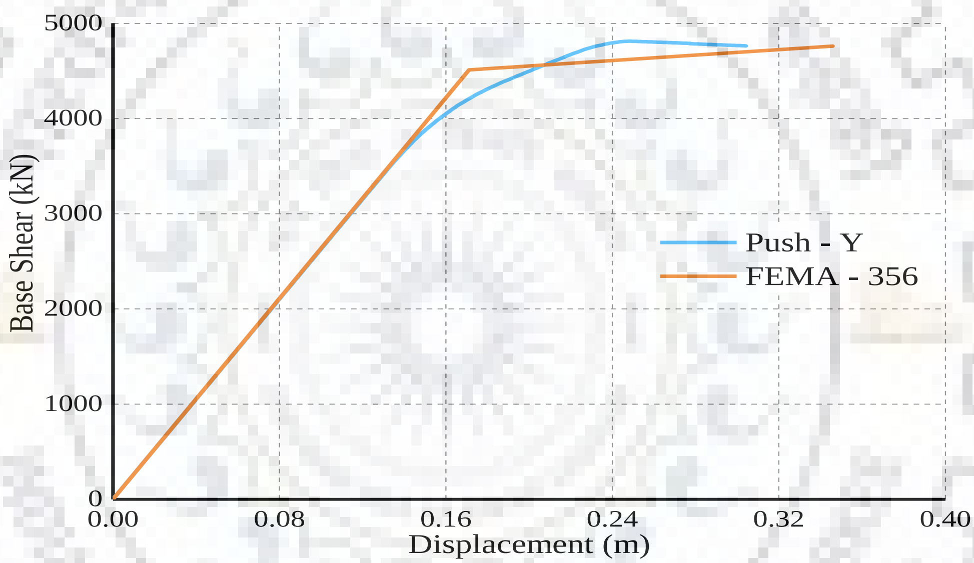



*Figure 5.8 Pushover curve of the existing building along longitudinal direction.*

*Table 5.6 Pushover analysis results along longitudinal direction.*

| | | | |
|---|---|---|---|
| Design base shear (Vd) | 1771.00 kN | | |
| Base shear at yield (Vy) | 4511.28 kN | Yield disp. (Δy) | 0.17 m |
| Base shear at failure | 4763.71 kN | Maximum disp. (Δmax) | 0.30 m |
| Overstrength factor (Vy/ $V_d$) | 2.54 | Ductility reduction factor (Rμ) | 1.77 |

## 5.4 Nonlinear static analysis of CFRP wrapped building.

Pushover analysis has been done for four states of the building, i.e., the existing one, second is the existing with failure members retrofitted with 1 layers of CFRP strips, third and last one is the existing building with failure members retrofitted with 3 and 5 layers of CFRP strips respectively. The analysis were done on the 2d frame structure of the building in both longitudinal and transverse direction. The effect of P-Δ has also been considered in all nonlinear analysis.

Analysis of a typical 2d frame of the building shows that there is very less increase in the strength of the retrofitted frame over existing one. Whereas the ductility of the frame was observed to be greatly enhanced. The peak displacement capacity of the retrofitted frame has been increased by 85.80% along transverse direction while 85.75% along longitudinal direction. The increase in base shear carrying capacity of the retrofitted frame along transverse and longitudinal directions are 5.29% and 9.17% respectively. The initial stiffness of the frame was observed to be identical for all the four cases analysed, whereas for post yielding performance of the building frame, a slight enhancement was observed.

The results of the nonlinear static analysis of the building frames are presented below in the Table 5.7.

*Table 5.7 Comparison of pushover results for existing and retrofitted frames*

| **Structure type** | | **Transverse dir. (X)** | | **Longitudinal dir. (Y)** | |
|---|---|---|---|---|---|
| | | *Disp. (mm)* | *Base shear (kN)* | *Disp. (mm)* | *Base shear (kN)* |
| **Existing structure** | | 409.66 | 518.69 | 401.44 | 1279.10 |
| **Retrofitted structure** | 1 Layer wrapping | 604.26 | 530.96 | 581.52 | 1293.87 |
| | 3 Layers wrapping | 700.34 | 544.44 | 671.85 | 1321.70 |
| | 5 Layers wrapping | 761.10 | 554.90 | 745.68 | 1396.90 |

The pushover curve for the frame along transverse and longitudinal directions are plotted as shown in Figure 5.9 and Figure 5.10 respectively.

***Pushover analysis results in transverse (X) direction***

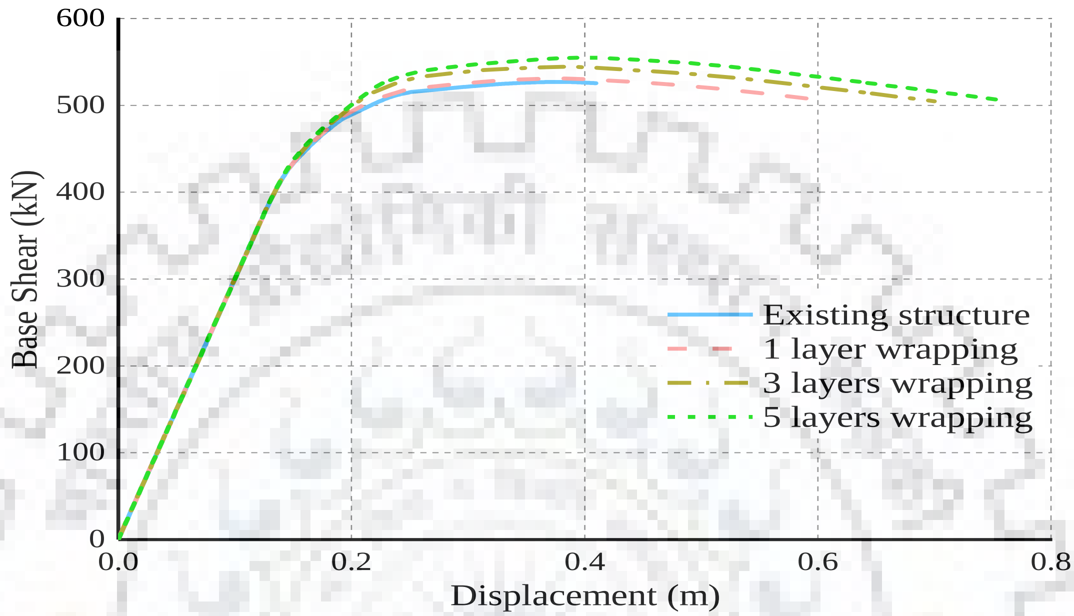



*Figure 5.9 Pushover curve for the CFRP wrapped transverse building frame*

***Pushover analysis results in longitudinal (Y) direction***

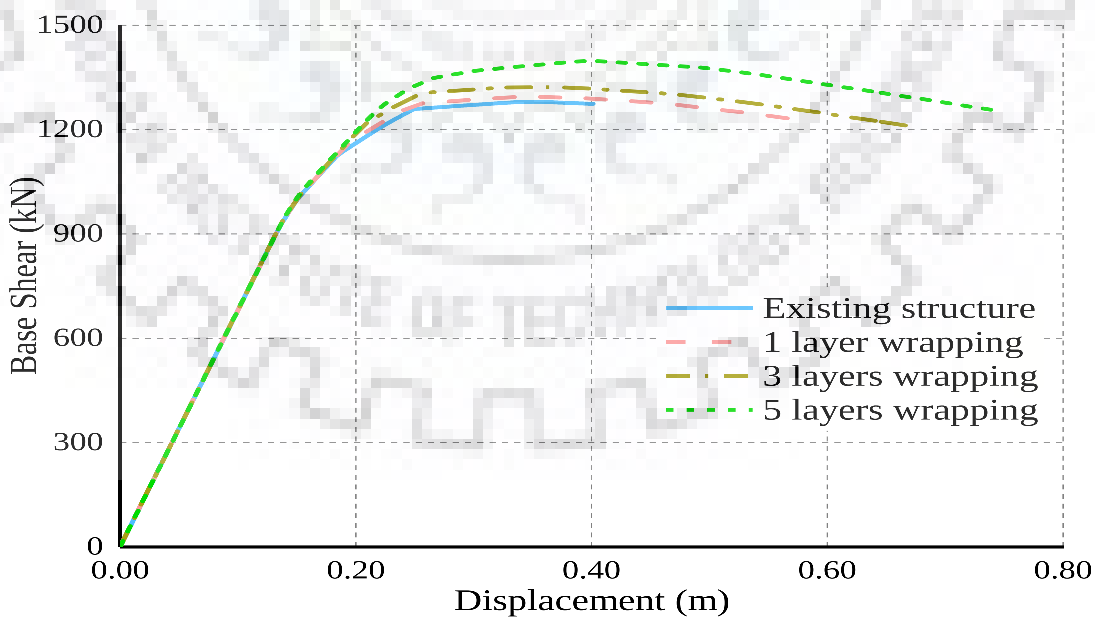



*Figure 5.10 Pushover curve for the CFRP wrapped longitudinal building frame*

### 5.4.1 Energy dissipation capacity

The maximum energy dissipated by existing and retrofitted frames along transverse and longitudinal directions are shown in Figure 5.11 and Figure 5.12 respectively. It can be seen that the energy dissipation capacity of the frames increased when they are rehabilitated using FRP wrapping.

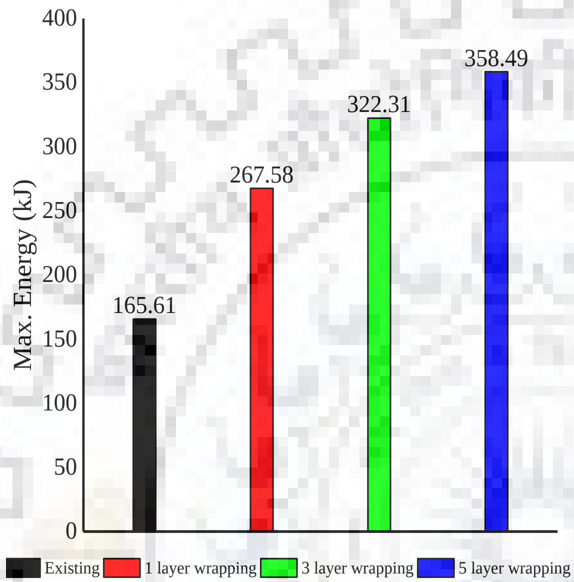



*Figure 5.11 Maximum energy dissipated for existing and retrofitted frames along transverse direction*

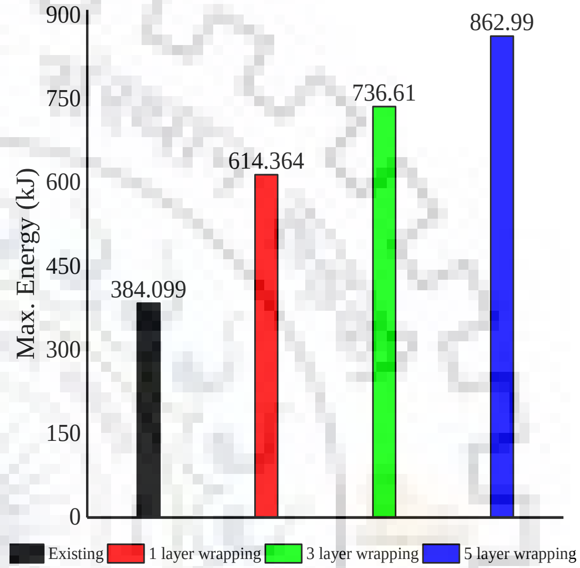



*Figure 5.12 Maximum energy dissipated for existing and retrofitted frames along longitudinal direction*

The energy dissipation capacity of the retrofitted frame along transverse direction has been increased by 61.58%, 94.62% and 116.48% for 1, 3 and 5 layers of CFRP wrapping respectively. While along longitudinal direction frame it has been increased by 59.94%, 91.77% and 124.68% for 1, 3 and 5 layers of CFRP wrapping respectively.

Thus the nonlinear static analysis conducted on existing and retrofitted frames rehabilitated using CFRP showed that this rehabilitation method is very efficient in increasing the energy dissipation capacities of the frames.

## 5.5 Member level analysis for CFRP wrapping

The enhancement in seismic capacity (i.e. strength and ductility) of the members wrapped with CFRP strips are compared in this section. The strength and ductility parameters which are compared are peak axial strength of the column, moment rotation and moment curvature of the section.

### 5.5.1 Comparison of strength parameter

The CFRP strips are much efficient in enhancing the peak axial strength of the columns. The increase in peak axial strength of the columns after CFRP wrapping was found to be 9.63%, 28.79% and 48.02% for 1, 3 and 5 layers respectively.

### 5.5.2 Ultimate Curvature and Plastic Rotation capacity of the members

The ultimate rotation and plastic rotation capacity of the member represents the capability of the member to undergo deformation beyond yielding. The ultimate curvature and plastic rotation capacity of the member is characterized by means of moment-curvature (M-$\phi$) and moment-rotation (M-$\theta$) diagrams.

To evaluate the plastic rotation capacity of the member, the plastic hinge length of the member is required to be known. The required mathematical expression to calculate the plastic hinge length of the member is taken as proposed by Paulay and Priestley.

Figures below shows the comparison of the ductility parameter of the columns:

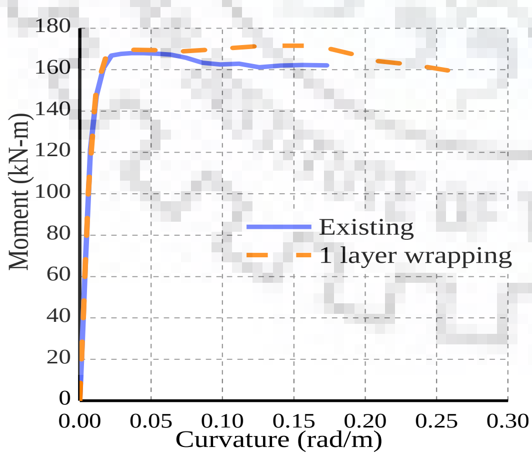



*Figure 5.13 Moment curvature diagram for column with 1 layer of CFRP*

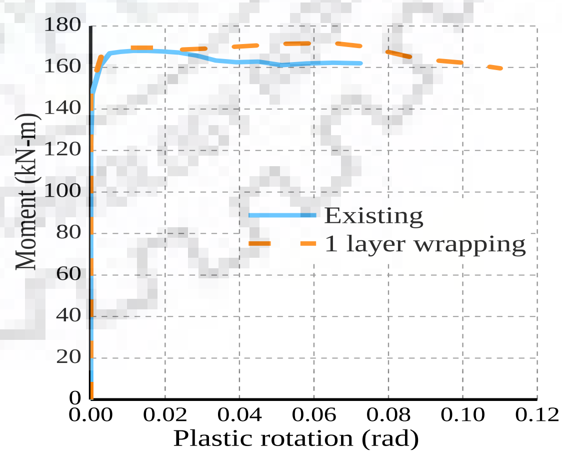



*Figure 5.14 Moment rotation diagram for column with 1 layer of CFRP*

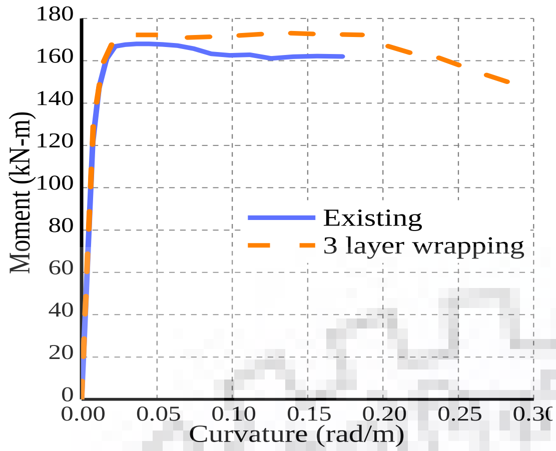



*Figure 5.15 Moment curvature diagram for column with 3 layer of CFRP*

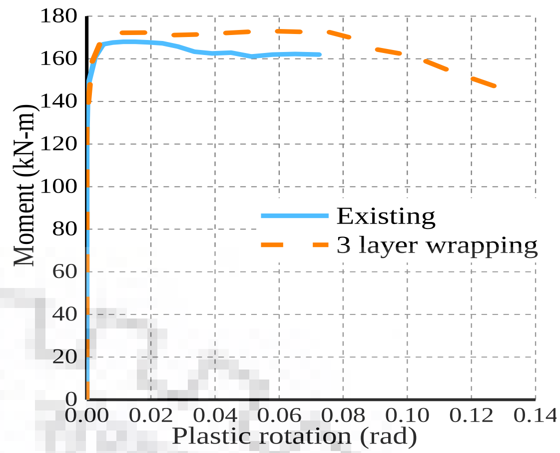



*Figure 5.16 Moment rotation diagram for column with 3 layer of CFRP*

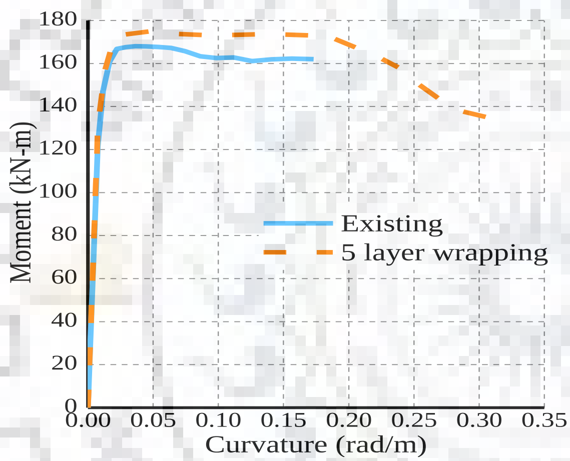



*Figure 5.17 Moment curvature diagram for column with 5 layer of CFRP*

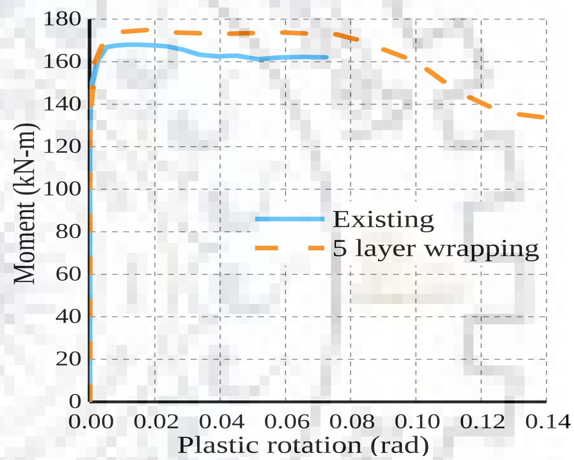



*Figure 5.18 Moment rotation diagram for column with 5 layer of CFRP*

Table 5.8 shows the increase in the ductility parameter of the members:

*Table 5.8 Comparison of ductility parameters of the columns*

| **No. of layers** | ***Ultimate curvature*** | ***Plastic rotation capacity*** |
|---|---|---|
| 1 layer wrapping | 49.13% | 52.00% |
| 3 layers wrapping | 70.69% | 75.70% |
| 5 layers wrapping | 86.76% | 93.03% |

# CHAPTER 6 CONCLUSIONS

In this dissertation, an existing building has been analysed for gravity and seismic loading, and suitable retrofitting technique has been proposed to strengthen the same. The local retrofitting technique i.e. CFRP Jacketing has been used for adequately strengthening the building for the imposed seismic demand on the structure. Nonlinear static analysis has been performed on the existing and retrofitted frames so as to estimate the structure behavior under dynamic loading condition. The conclusions drawn form this studies are as follow:

1) The results obtained from linear analysis methods shows that, the FRP wrapping is sufficient to increase the seismic capacity of the existing building.

2) As observed from the results obtained from the nonlinear analysis of the retrofitted frame, the FRP wrapping of the deficient members shows the ductility of the structure has increased significantly. The enhancement in the ultimate displacement is found to be approximately 47.50%, 70.9 % and 85.80% for 1, 3 and 5 layers of CFRP wrapping for the building frames in transverse direction; whereas in longitudinal direction it is found to be 44.85%, 67.36% and 85.75% for 1, 3 and 5 layers of CFRP wrapping respectively. The increase in base shear carrying capacity of the FRP wrapped building is found to be 5.29% and 9.17% higher than the existing one along transverse and longitudinal direction respectively. Hence from nonlinear analysis results, it can be concluded that, with increasing the number of layers of CFRP wrapping, the ductility is enhanced enormously as compared to strength.

3) The initial stiffness of the frame is being observed to be identical for all the four cases analysed, whereas in post yielding performance of the building frame, a slight enhancement is observed. Hence it can be concluded that FRP wrapping does not contribute to the structure strength significantly, but it increases the structures ductility.

4) The nonlinear analysis conducted on the frames rehabilitated using FRP wrapping shows that, this rehabilitation scheme is very efficient in increasing the energy dissipation capacity.

5) The results obtained from member level analysis of FRP wrapped column shows that, FRP wrapping is efficient in increasing the axial strength and plastic rotation capacity i.e. it enhances both the strength and ductility parameters of the member.

6) From the results of this analytical investigation in this study, it can be concluded that, though the member ductility and strength parameters can be greatly enhanced from FRP wrapping, but their effect in the global performance of the structure is less effective; particularly in terms of enhancing the base shear carrying capacity.

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