EFFECT OF FRP WRAPPING ON STRENGTH AND DUCTILITY PARAMETER OF RC MEMBERS SUBJECTED TO AXIAL AND LATERAL LOADS

A DISSERTATION

Submitted in partial fulfilment of the
Requirements for the award of the degree
of
MASTER OF TECHNOLOGY
in
EARTHQUAKE ENGINEERING
(With Specialization in Seismic Vulnerability and Risk Assessment)

By
SUBHAJIT DE

DEPARTMENT OF EARTHQUAKE ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY ROORKEE
ROORKEE-247667, INDIA
MAY, 2016

CANDIDATE'S DECLARATION

I hereby declare that the work which is presented in this report , entitled "**Effect of FRP Wrapping on Strength and Ductility Parameter of RC Members Subjected to Axial & Lateral Loads**", in partial fulfillment of the requirement for the award of the degree of Master of Technology in Earthquake Engineering with specialization in Seismic Vulnerability and Risk Assessment submitted in Earthquake Engineering Department, Indian Institute of Technology Roorkee, is an authentic record of my own work carried out during the period from January 2015 to May 2016 under the supervision and guidance of Dr. Pankaj Agarwal, Professor, Earthquake Engineering Department, Indian Institute of Technology Roorkee, Roorkee (India).I also declare that I have not submitted the matter embodied in this thesis for award of any other degree.

Date: 20th May, 2016
Place: Roorkee

(SUBHAJIT DE)

CERTIFICATE

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

Dr. PANKAJ AGARWAL
Professor
Department of Earthquake Engineering
Indian Institute of Technology Roorkee
Roorkee-247677

ACKNOWLEDGEMENT

I feel much honoured in presenting this report in such an authenticable form of sheer endurance and continual efforts of inspiring excellence from various coordinating factor of cooperation and sincere efforts drawn from all sources of knowledge. I express my sincere gratitude to Dr. Pankaj Agarwal, Professor, Earthquake Engineering Department, Indian Institute of Technology Roorkee for his valuable guidance and infilling support for the completion of seminar work.

I am also grateful to all faculty members and staff of Earthquake Engineering Department, Indian Institute of Technology, Roorkee.

Date: 20th May, 2016
Place: Roorkee

(SUBHAJIT DE)

ABSTRACT

Lessons learnt from past & most recent earthquakes, continuously call for the seismic strengthening of the structures. As a matter of fact, the number of lives & economic losses claimed by a disaster like earthquake isn't due to the earthquake itself, it is primarily due to the collapse of civil engineering structures which are unable to withstand the ground motion induced due to earthquake. There is a famous quotation that "Earthquakes don't kill people, buildings do". This depicts the uttermost necessity to check for the seismic vulnerability of the structures & to make them able to withstand the earthquake forces that it may be subjected to throughout its remaining life or increased design period. In a country like us, where thick population can cause a magnified effect on the earthquake hazard, it is really of prime importance to strengthen the seismically vulnerable structures.

Seismic retrofitting has been an interesting topic for researchers for quite some time. However, for our country hardly anything has been done in terms of code prescription or special design requirements for seismic load cases. An attempt has been made through this research procedure so as to represent seismic retrofitting technique a dearer one to the design offices and for practical implementation.

In this study, an existing building which is primarily a Moment resisting frame of five storeys, has been analysed for gravity loading and seismic loading. The capacities of the existing and retrofitted buildings has been evaluated and compared through Non-linear static analysis of the frames. The local and global retrofitting techniques which has been used are FRP jacketing and installation of additional steel bracings in the building frame. The viability of the local retrofitting technique such as FRP jacketing of the columns and beams so as to boost the overall structural capacity of the structure has been investigated. 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 using the non-linear hinge parameters developed manually with the help of Microsoft Excel and feeding them into SAP2000. Also, the global performance of the structure after installation of additional steel bracings with and without FRP wrapping has been investigated through nonlinear static analysis of the retrofitted frames. The study shows that the enhancement in the global performance of the structure is insignificant when only columns are retrofitted with FRP strips, however, the ductility of the structure can be enhanced significantly when both the beams and columns are wrapped.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 General

In the wake of the past and most recent earthquake disasters, the retrofitting of structures has become an inevitable issue in the modern days. Retrofitting typically means alteration in the structure so as to better accommodate the catastrophe it is retrofitted for. This alteration may be done by installing additional members or by increasing the member sizes or confining them in a better way than the existing. Although the term is primarily used for seismic mayhems, it is equally applicable for any natural disaster such as tornadoes, severe winds, tropical cyclones etc.

The awareness for the seismic protection was mainly adopted in design provisions only in the era of sixties. Before their introduction, most structures were designed without proper seismic detailing. Particularly for these structures seismic retrofitting in the only way to prevent them from being collapsed due to earthquakes.

With the increasing problem of aging infrastructures & buildings worldwide, the structural retrofit work has come to the forefront of the industry practice. This problem, coupled with revisions in the structural codes to better accommodate natural phenomenon, creates the need for the development of successful structural retrofit technologies. Solution of this can be primarily done through two ways, i.e. strengthening or demolition. The latter option has its own limitation of becoming economically viable. This fact necessitates the establishment of systematic structural strengthening measures. A prospective system that has been booming for use in retrofitting techniques involves lateral confinement of the concrete in the structural member. Lateral confinement provides following property enhancement in the concrete member: increases compressive strength & ultimate strain (pseudo-ductility), provides a mechanism for shear resistance & inhibits longitudinal steel reinforcement buckling. The concept of utilizing FRP materials in strengthening structural systems was first evaluated in Japan as an alternative to steel systems.

Very recently, in 2013, Bureau of Indian Standards (BIS) has published a guideline for seismic evaluation and retrofitting of structures, IS 15988:2013 [7]. In this code, various retrofitting techniques has been discussed and protocol to use them have been issued. However, when compared to the other similar documents from around the world such as ASCE 41-13, FEMA 356 etc., it is felt that more elaborate discussion on the topics are necessary for more accurately evaluate the structure. The retrofitting techniques that are proposed in the code require more clarity when applying them in the real world situation.

1.2 Retrofitting Methods

Method of retrofitting that should be applied to a structure depends on the level of deficiency of the structure itself, required performance, availability of space 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. The local retrofitting techniques are mainly done through confining the concrete members which is discussed in more detail in subsequent sections of this dissertation report. In global retrofitting techniques, additional members are installed in the existing system aimed primarily to transfer the lateral loads to the foundation through an alternate path. Global retrofitting techniques often require additional foundation for supporting the newly installed members or strengthening of existing one. Global retrofitting can also be done by decreasing the seismic demand on the structure which is known as unconventional method of retrofitting. The various types of retrofitting method is described in the tree diagram below:

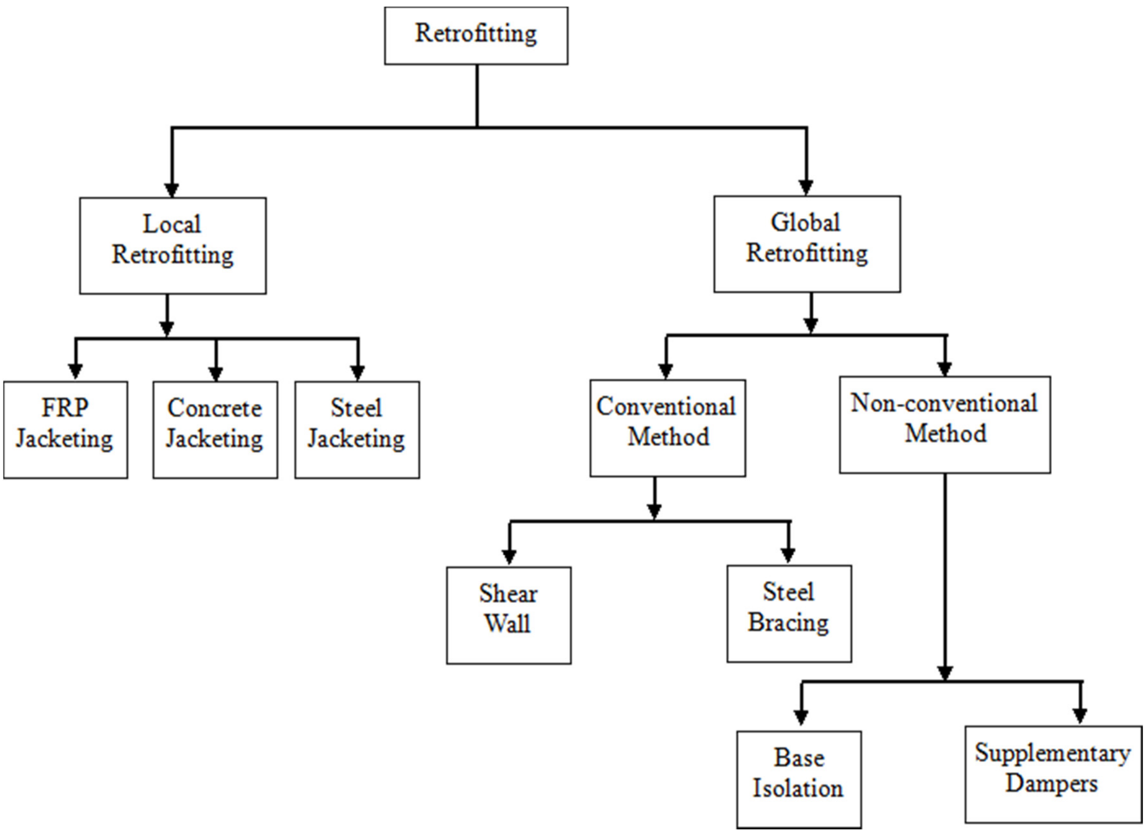


Fig. 1.1: *Retrofitting Types*

1.3 Confinement of Concrete

The applicability of lateral confinement in enhancing the mechanical properties of the reinforced concrete members has already been proved by many researchers [2-7]. The performance of the concrete member is dependent on the interaction between the confined concrete & the confining material. The enhancement in the material behaviour of the concrete is due to the development of a triaxial stress field in it and the restriction in lateral dilatation of the member after its cracking (Nanni & Bradford, 1995, [4]). However, the confinement due to the internal transverse reinforcements & the external FRP sheets are somewhat different in a manner that in the first case, a constant confining pressure can be assumed as the steel can be in plastic flow, whereas, in the latter one, the confining pressure increases continuously with the applied load owing to the fact that the FRP composite remain linear elastic until the final rupture. The stress-strain models of FRP confined concrete can broadly be classified into two major categories i.e., i) Design oriented models & ii) Analysis oriented models [4]. In the first category, the material behaviour, i.e. the stress-strain relationship of the FRP confined concrete are predicted using closed-form equations based directly on the interpretation of the experimental results, whereas, on the other hand the latter category correspond to an incremental numerical procedure. Also, in case of FRP wrapped concrete, the resultant stress-strain relationship can be classified into three categories. The first category corresponds to a bi-linear monotonically increasing type. Most of the tests on FRP confined concrete falls under this category. However, this is also recognized that this type of stress-strain relationship is obtainable only if the volume of FRP material exceeds certain threshold value. There are also another type which depicts a descending branch of Stress-strain relationship for confined concrete beyond the peak confined strength of the concrete. These can be further divided into two more categories; one where the ultimate concrete strength lies below the unconfined strength of concrete & another where the ultimate strength is greater than the unconfined specimen. The first type where the monotonically ascending branches are observed & the second type where the ultimate strength of the concrete is greater than the unconfined strength, is called sufficiently confined concrete whereas, the remaining one is formally known as insufficiently confined concrete (Lam & Teng, 2003, [6]).

The lateral confinement of the concrete can be done in two ways i.e. i) Active confinement of the concrete & ii) Passive confinement.

Active confinement of concrete refers to the scheme of Retrofitting/Repairing which exerts initial tensile stresses in the confining material. An example of the active confinement can be stated as a column wrapped by over-sized composite confining material with the gap in between the concrete & the confining material filled with pressurized epoxy resin. In this system, the pressurized epoxy resin exerts initial tensile stress in the composite straps which in turn creates an active pressure on the column concrete which reduces the radial dilation & cracking of the core concrete (Saadatmanesh et al. 1996, [7]).

On the other hand, the Passive confinement of the concrete indicates gradual increase in confining pressure with increase in the outward expansion of the concrete. However, the passive confinement of the concrete becomes relevant only after extensive micro-cracking of the confined concrete. After initiation of cracking in the concrete, the transverse to axial strain ratio is no longer remains a material property, & its variation is the result of crack opening. That is why, some of the researcher prefer terming this ratio as "Strain Ratio" (Nanni & Bradford 1995, [4]). Previous research has shown that the strain ratio remains almost constant up to approximately 85% of f_{co}' at 0.20, increases to a value of 0.50 between 85% & 100% of f_{co}'. Beyond f_{co}', the value can increase up-to values 1.20 or higher [4]. The passive confinement of the concrete can be done providing lateral ties or hoop reinforcements or wrapping up the concrete externally with composites with epoxy resins.

The failure modes of the FRP wrapped RC members is also an important parameter to be identified. The modes of failure of a FRP wrapped RC member is primarily due to 1) Steel yielding followed by FRP fracture; 2) Steel yielding followed by concrete crushing; 3) Concrete compressive crushing; 4) FRP peel off at the termination or cut-off point due to shear failure of the concrete; 5) Debonding at the concrete-FRP interface in areas of concrete surface unevenness or due to faulty bonding [15]. Out of these failure modes, the most desirable one is the one in which first the reinforcing steel is yielded and then the concrete is crushed [15] which is also compliant with the normal design procedure of the under-reinforced RC sections.

1.4 Fiber Reinforced Polymer

To start with, let us first define the term "composites", composites may be defined as, a combination of two or more materials (reinforcement, resin, filler, etc.), differing in form or composition on a macro scale. The constituents retain their identities, i.e., they do not dissolve or merge into each other, although they act in concert. Normally, the components can be physically identified and exhibit an interface between each other.

Fiber reinforced polymer (FRP) is a composite made of high-strength fibers and a matrix for binding these fibers to fabricate structural shapes. Common fiber types include Aramid, carbon, glass, and high-strength steel; common matrices are epoxies and esters. Inorganic matrices have also been evaluated for use in fire-resistant composites. The binder matrix in the composites has two primary functions:
a) Transferring loads among the fibers &
b) Protecting the fibers from the environmental effects.

Originally developed for aircraft, FRP is particularly suitable for structural repair and rehabilitation of reinforced and prestressed concrete elements. The evaluation of the FRP composites started in the early 1980s in Japan. Before this, the applications of these materials were limited to the specialized application of aerospace & defence industries owing to their high cost. During these periods, the retrofitting schemes of the structures were also emphasized & seismic retrofitting being at least important state, the emergence of FRP for application in the construction industry became obvious. The salient properties of the FRP which particularly enables its use in the latter category can be summarized as:

a) High strength to weight ratio.
b) Ductility for improved seismic performance.
c) High fatigue resistance.
d) Excellent tensile strength in the direction of Fibers.
e) Do not yield, but instead are elastic up to failure.
f) Low modulus of elasticity in tension.
g) Exceptionally corrosion resistant.

Fibers give the composite high tensile strength & rigidity along their longitudinal direction. In the subsequent section of this report a review for the effects of orientation of the fibers has been discussed. Several types of fibers have been developed for use in FRP composites. For structural applications, research & development has been conducted using carbon, Aramid & glass fibers. These fibers exhibit an ultimate strain range of 1 to 4%, with no yielding occurring prior to failure. The ultimate strength range is approximately 5700 to 3300 MPa, & elastic moduli range from 270 to 70 GPa.

1.5 Type of Fibers used in Composites:

There are commonly three types of fibers used in the structural applications. They may be used in the form of sheets, FRP bars or combining the raw materials, they can be manufactured at the site also. Generally, three types of composite fibers are used i.e. Carbon fiber, Glass fiber & Aramid fibers. A brief description of each of the type is described in the following tree diagram:

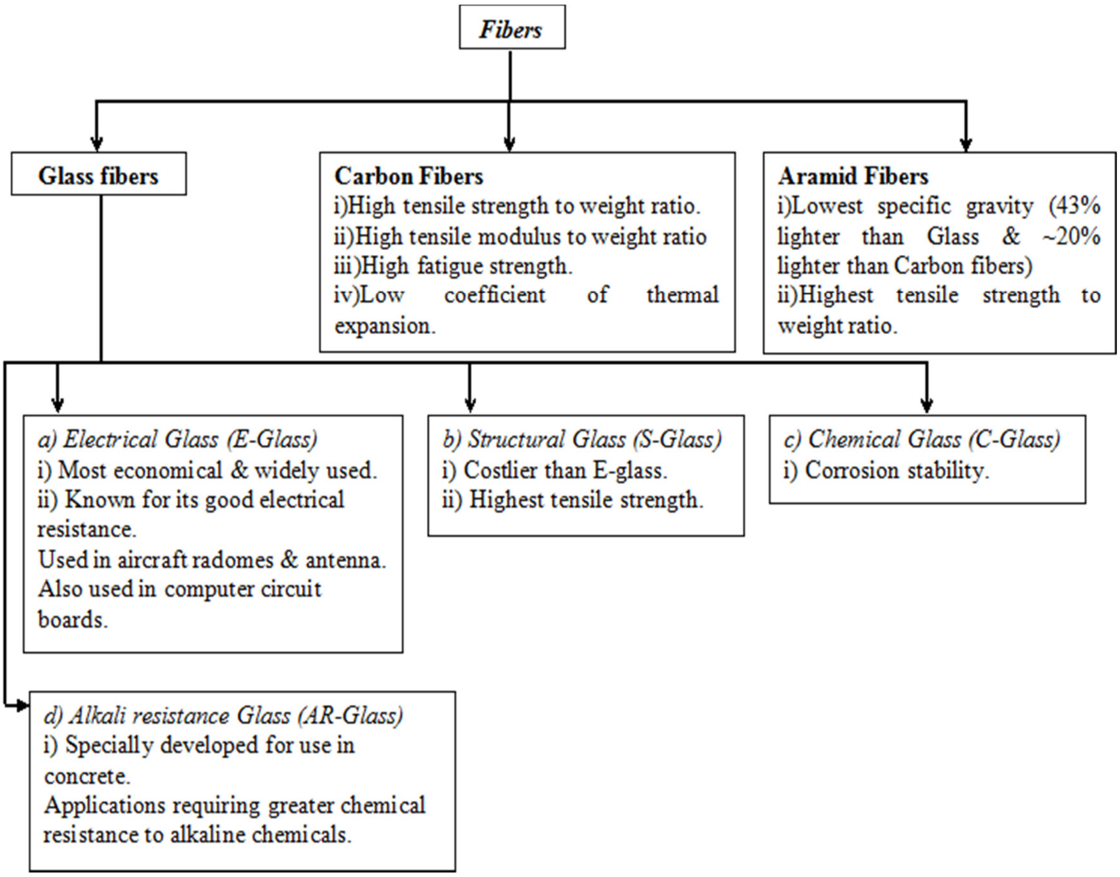


Fig. 1.2: *Type of Fiber Composites.*

A comparison of typical properties for different FRP composites, i.e., E-glass, S-glass, High strength Carbon fiber, High modulus & ultra-high modulus carbon fiber is shown in the table below:

Table 1.1: Comparative properties for various FRP Composites (*Source: ACI 440.2R-08 [17] Table A1.1)*

Fiber Type	**Elastic Modulus (GPa)**		**Ultimate Strength (MPa)**		**Rupture Strain (min)**
	Min	**Max**	**Min**	**Max**	
Carbon fibers					
General purpose	220	240	2050	3790	1.2
High strength	220	240	3790	4820	1.4
Ultra High strength	220	240	4820	6200	1.5
High Modulus	340	520	1720	3100	0.5
Ultra High modulus	520	690	1380	2400	0.2
Glass fibers					
E Glass	69	72	1860	2680	4.5
S Glass	86	90	3440	4140	5.4
Aramid fibers					
General purpose	69	83	3440	4140	2.5
High performance	110	124	3440	4140	1.6

CHAPTER 2

A BRIEF REVIEW OF THE PREVIOUS WORKS

This section of the report primarily deals with the models that the researchers has proposed for the evaluation of the strength & ductility parameters of the concrete confined by various confinement methods i.e., Confinement due to internal transverse reinforcement, confinement properties of FRP wrapped RC members etc. The section has been subdivided into five parts. In the first three parts, the numerical models proposed by various researchers for unconfined, stirrup confined and FRP confined concrete has been presented. In the fourth part, nonlinear analysis procedure for such FRP wrapped frames has been discussed. The comparison between various confinements in concrete has been presented in the fifth part of this section.

2.1 Unconfined Concrete Model

Wan T. Tsai (1988) [1] proposed a model for unconfined concrete which takes into account both the ascending & descending branches of concrete in a single numerical relationship. The numerical relationship that had been proposed by Tsai can be considered as a generalized equation for the similar relationships proposed by Popovics (1970) & Saenz (1964). The new relationship considered two parameters, m & n which control the ascending & descending branches of the stress strain relation respectively. The relationship particularly performs well in cases where concrete strength is higher than 34.5 MPa (5 ksi). The recommended stress-strain relationship for the unconfined concrete as proposed by Tsai is as follows:

$$y = \frac{mx}{1 + \left(m - \frac{n}{n-1}\right)x + \frac{x^n}{n-1}} \text{.............(2.1.1)}$$

Where, $y = f_c/f_c'$, Ratio of concrete stress to ultimate strength;

$x = \varepsilon/\varepsilon_c$, Ratio of concrete strain to the strain at y = 1;

$m = E_0/E_c$, Ratio of initial tangent modulus to the secant modulus at y = 1. (Factor for controlling steepness of the ascending branch of the stress- strain relationship.

n = Factor to control the steepness rate of the descending branch of the stress-strain relationship.

The factors for steepness control i.e., m & n were given by,

$m = 1 + 17.9/f_c'$; & ....(2.1.2)

$n = f_c'/6.68-1.85$................. (2.1.3)

The secant modulus of concrete $E_c = E_0/m$ & the strain at peak stress, $\varepsilon_c = f_c'/E_c$.

2.2 Confined Concrete model

Mander, Priestley & Park (1988) [5] 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 & corresponding strain of concrete confined by circular, spiral or rectangular hoops in a single mathematical relationship under uni-axial loading. They confirmed that the confinement of concrete by suitable arrangements of transverse reinforcement results in a significant increase in both the strength & ductility of the material under the mentioned conditions. In particular, the strength enhancement & the slope of the descending branch of the concrete stress-strain relationship have a considerable influence on the flexural strength & ductility of the reinforced concrete columns.

For a slow (quasi-static) strain rate & monotonic loading, the longitudinal compressive stress fc is given by,

$$f_c = \frac{f'_{cc}\, xr}{r-1+x^r} \;........................(2.2.1)$$

$$\varepsilon_{cc} = \varepsilon_{co}\left[1+5\left(\frac{f'_{cc}}{f'_{co}}-1\right)\right] ....(2.2.2)$$

Where, f_{cc}' = Peak strength of the confined concrete; f_{co}' = Peak strength of the unconfined concrete; $x = \varepsilon/\varepsilon_{cc}$, ε_{co} = Strain at peak stress of the unconfined concrete, &

$$r = \frac{E_c}{E_c - E_{sec}}\;, \quad E_c = 5000\sqrt{f'_{co}}, E_{sec} = \frac{f'_{cc}}{\varepsilon_{cc}}$$

The peak confined strength of concrete (f_{cc}') is given by,

$$\frac{f_{cc}'}{f_{co}'} = \left(-1.254 + 2.254\sqrt{1+\frac{7.94 f'_L}{f'_{co}}} - 2\frac{f'_L}{f'_{co}}\right) ....(2.2.3)$$

Where, f_l' is the effective lateral confining stress from the transverse confining reinforcements.

Determination of effective confining stress for *Circular section*:

The effective lateral confining stress due to transverse reinforcement is given by,

$$f'_L = f_l k_e ....(2.2.4)$$

Where, f_l is the lateral pressure from the transverse reinforcement, assumed to be uniformly distributed over the surface of the concrete core; & k_e is the confinement effectiveness coefficient which is defined as the ratio of the area effectively confined by reinforcement & the gross concrete area in the core. Thus,

$$k_e = \frac{A_e}{A_{cc}} ....(2.2.5)$$

For A_e, the area at midway between the levels of transverse reinforcement to be considered, as the area of ineffectively confined concrete will be maximum there. A representative diagram of the longitudinal section of the column confined with transverse reinforcements is shown below:

The arching action is assumed to occur in the form of a second degree parabola with an initial tangent slope of 45^0, the area of an effectively confined concrete core at midway between the levels of transverse reinforcements is,

$$A_e = \frac{\pi}{4}\left(d_s - \frac{s'}{2}\right)^k = \frac{\pi}{4}d_s^2\left(1 - \frac{s'}{2d_s}\right)^k \text{.........(2.2.6)}$$

Where, s' is the clear vertical spacing between spiral or hoop bars. k is exponent which is 1 for spiral & 2 for circular hoop reinforcement.

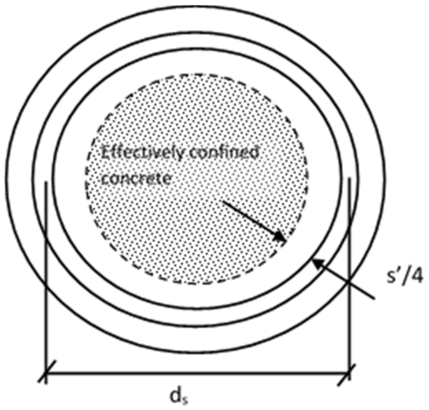


Fig. 2.1: *Effectively confined core for Circular hoop reinforcement, showing Transverse confinement [5].*

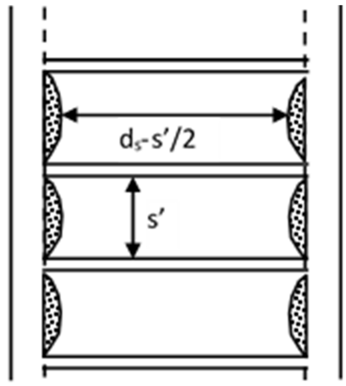


Fig. 2.2: *Effectively confined core for Circular hoop reinforcement, showing longitudinal confinement [5].*

The area of confined concrete within the center lines of the perimeter spiral or hoop, A_{cc} is given by,

$$A_{cc} = A_c(1 - \rho_{cc})\text{...........(2.2.7)}$$

Where, A_c is the concrete area enclosed by the center lines of the perimeter spiral or hoop; & ρ_{cc} is the ratio of the area of longitudinal reinforcements to the area of core section.

Thus, the effective confining lateral pressure works out to be, assuming that the confining effect of the concrete diminishes at the first fracture of the lateral confinement reinforcement by yielding in tension,

$$f_L' = \frac{1}{2}k_e \rho_s f_{yh}\text{..............(2.2.8)}$$

Where, ρ_s is the ratio of the volume of transverse confining steel to the volume of confined concrete core = $4A_{sp}/(sd_s)$ & f_{yh} is the yield strength of the lateral confining reinforcement.

Determination of effective lateral confining pressure for *rectangular* section:

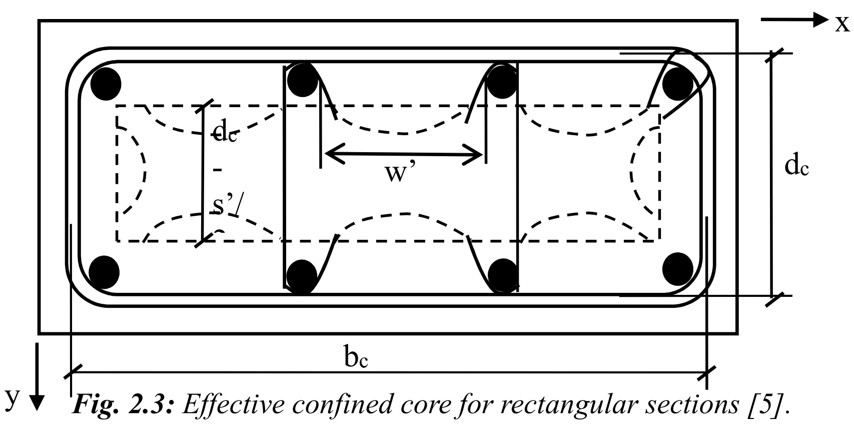


Fig. 2.3: *Effective confined core for rectangular sections [5].*

In case of a rectangular section, the arching action of the concrete remains same in the Longitudinal & transverse direction i.e. in the form of second degree parabolas with an initial tangent slope of 45^0. Only difference is that the area of effectively confined concrete in case of rectangular column is less than the effectively confined concrete area of a circular column having same gross area. The total plan area of ineffectively confined concrete at the level of hoops where there are n longitudinal bars is,

$$A_i = \sum_{i=1}^{n} \frac{(w_i^{'})^2}{6} \text{..............(2.2.9)}$$

Incorporating the influence of the ineffective areas in the elevation the confinement effectiveness coefficient for rectangular hoops becomes,

$$k_e = \frac{\left(1 - \sum_{i=1}^{n} \frac{(w_i^{'})^2}{6 b_c d_c}\right)\left(1 - \frac{s'}{2b_c}\right)\left(1 - \frac{s'}{2d_c}\right)}{\left(1 - \rho_{cc}\right)} \text{..............(2.2.10)}$$

As in the case of rectangular sections, the area of transverse reinforcements can differ in either direction, the confining pressure on the directions may be different. The effective lateral confining pressure in the X & Y direction is given by,

$$f_{Lx}^{'} = k_e \rho_x f_{yh} \text{..............(2.2.11)}$$

$$f_{Ly}^{'} = k_e \rho_y f_{yh} \text{..............(2.2.12)}$$

Where, ρ_x & ρ_y are the transverse steel ratios for the respective directions.

2.3 Experimental Evaluations of FRP wrapped RC Members

A.Nanni & Norris (1995) [3] carried out experimental studies to evaluate the behaviour of laterally confined FRP wrapped concrete members. A total of twenty six specimens were loaded quasi-statically under cyclic flexure, with or without axial load placed on them. Two different types FRP composites (braided aramid FRP tape & pre-formed glass-aramid shells) were used in the evaluation process. Two type of cross sections were used as well for the experiments. The hysteresis & moment- force interaction diagrams were used to study the confinement effects on the specimens. The results of their experiment showed that the jacketed specimens with no axial load placed on them behave similar to the corresponding unconfined specimens. The small strength enhancements were limited to additional longitudinal reinforcing effect provided by the FRP fibers. The wrapped specimens developed vertical flexural cracks only and no diagonal shear failure was observed in the same. It was also observed that the specimens subjected to various levels of axial loading placed on them, showed higher flexural rigidity proportional to the axial force placed on them. They concluded that the enhancement due to FRP confinement is expected under load situations in which the members are predominantly in compression, as the confinement pressure becomes relevant only after the concrete has already undergone a certain amount of micro-cracking. Where the member fails due to tension, the presence of FRP confinement should have a less dominant effect on the performance of the member. Also, the tape wrapping method & the circular sections performed more effectively in the test than the preformed shell confinement & the rectangular section.

Saadatmanesh, Ehsani & Jin (1996) [7] investigated many bridge failures for the then earthquakes by poor performance of concrete columns primarily due to inadequate lateral reinforcement & insufficient lap of starter bars. They proposed an effective & economical alternative for seismic retrofitting of those substandard columns. They tested five scaled down, circular reinforced concrete bridge column footing assemblages retrofitted with both active & passive confinement procedures.

For the first three specimens, they have used starter bars projecting from the footing and lapped for 20 bar diameters with the main longitudinal bars of the column. The remaining two columns, the column reinforcements were made continuous to the footing & a standard 90 degree hook was used for anchoring the bars. One from each type of column has been used as control specimen. The remaining columns were retrofitted using active & passive confinement. The retrofitted columns were wrapped with six layers of 0.80 mm thick composite strap within the potential plastic hinge zone of the columns. The columns were prestressed to simulate the dead load on them. The results of the tests were depicted through load vs displacement, curvature with respect to height & load vs. strain plots. The load versus displacement curves showed that the retrofitted columns carried significantly larger lateral load with stable hysteresis loop. The lateral load carried by the column with active confinement was maximum. For obvious reasons, the non-retrofitted column with lap splicing in the plastic hinge region showed least ductility and failed by degradation of lap splicing. The brittle failure was due to the debonding of lapped bars resulting from insufficient transverse reinforcement. The control column with continuous bars without retrofitting also showed significant ductile performance than the lapped spliced one demarcating the potential adverse effects of lapping bars in the plastic hinge regions.

The curvature versus height plots confirmed the significant increase in the rotational capacities of the sections due to confinement provided by the composite straps than the stock one. However, any improvement due to the active confinement wasn't revealed in the test.

The load vs. strain had been recorded for the longitudinal reinforcement, transverse reinforcement as well as the composite straps. The results showed that the confinement by the composite straps allowed higher strain capability in longitudinal & transverse bars before failure, resulting in higher overall ductility & energy absorption capacity of the retrofitted columns.

They concluded that the columns lap-spliced longitudinal reinforcement in the potential plastic hinge regions, appear to fail at low ductility levels of u=1.2 to 1.5. Whereas, the columns with continuous reinforcements through the plastic hinge region improves moderately & the degradation is delayed until ductility levels of u = ±4 is reached. The retrofitted columns developed a very stable load-displacement hysteresis loops up-to a displacement ductility of u = ±6, without evidence of significant structural deterioration associated with the bond failure of lapped starter bars or longitudinal reinforcement buckling.

Banzaid & Mesbah (2013) [14] conducted a test series to evaluate performance of circular & square column strengthened with carbon fiber reinforced polymer (CFRP) sheets. One of the interesting findings of their results is that, in the existing models for FRP confined concrete, it is commonly assumed that the FRP ruptures when the hoop stress in the confining FRP jacket reaches its ultimate tensile strength, however, the tests indicated that at failure, the ultimate strength of the FRP material wasn't reached. This phenomenon considerably affected the accuracy of the existing models for FRP confined concrete. Based on the effective lateral confining pressure & the effective circumferential failure strain, they had proposed numerical relationships so as to predict the strength of FRP confined concrete & the corresponding failure strain in the CFRP sheet. They concluded with a new confinement model for CFRP wrapped square & circular which takes into account the ratio of actual failure strain to the ultimate strain of the composite. The model proposed is presented in the subsequent paras of this document. Evidently, the circular section showed a better performance for strength & ductility as compared to the square sections with similar confining arrangement. The failures of the specimens were observed to be brittle & sudden owing to the material properties of the confining materials. The CFRP confinement yielded higher results in terms of strength & ductility for lower strength specimens than in specimens of higher in-situ concrete grade.

The numerical relationships as proposed by the author is presented below:
For *circular* sections wrapped with CFRP sheets,

$$f_{L,eff} = \frac{2t_{frp}E_{frp}\varepsilon_{h,rup}}{d} = \frac{2t_{frp}E_{frp}\eta\varepsilon_{fu}}{d} \text{.........(2.3.1)}$$

Where, η = Effective FRP strain coefficient; $\varepsilon_{h,rup}$ = Strain at rupture of CFRP sheet ; E_{frp} = Young's modulus of FRP ; t_{frp} = Total thickness of FRP wrapping ; d = Diameter of the circular column section. $f_{L,eff}$ = Effective lateral confinement pressure; & ε_{fu} = Ultimate strain of FRP.
The relation between the unconfined concrete to that of the confined one was proposed as:

$$\frac{f_{cc}'}{f_{co}'} = 1 + 2.20\frac{f_{L,eff}}{f_{co}'} = 1 + 1.60\frac{f_L}{f_{co}'} \text{..........(2.3.2)}$$

Where, f_{cc}' = Peak strength of the confined concrete; f_{co}' = Peak strength of the unconfined concrete; $f_{L,eff}$ = Effective lateral confinement pressure = $\eta\, f_L$; η = Effective FRP strain

coefficient = 0.73 (as evaluated in the investigation);
The strain at peak stress were given as,

$$\varepsilon_{cc} = \varepsilon_{co}(2 + 7.6\frac{f_{L,eff}}{f_{co}'}) = \varepsilon_{co}(2 + 5.55\frac{f_L}{f_{co}'}) \text{..........(2.3.3)}$$

Where, ε_{co} = Strain at peak stress of the unconfined concrete.
For *square* sections wrapped with CFRP sheets,

$$f_{L,eff} = \frac{2t_{frp}E_{frp}\varepsilon_{h,rup}}{\sqrt{2}b} = \frac{2t_{frp}E_{frp}\eta'\varepsilon_{fu}}{\sqrt{2}b} \text{..........(2.3.4)}$$

For square section without any corner radius. b is the side length of the square sections. The other parameters in the equation are as defined earlier. For square sections with corner radius R_c, the equation modified as:

$$f_{L,eff} = \frac{2t_{frp}E_{frp}\varepsilon_{h,rup}}{\sqrt{2}b - 2R_c(\sqrt{2}-1)} = \frac{2t_{frp}E_{frp}\eta'\varepsilon_{fu}}{\sqrt{2}b - 2R_c(\sqrt{2}-1)} \text{......(2.3.5)}$$

Unlike circular sections, the square sections do not show uniform confinement of the concrete. The core confined concrete in case of a square can be considered as the area confined by four second degree parabola along the edges of the square section whereas, in case of a circular section, the entire concrete section can be considered as uniformly confined with equal lateral confining pressure, as shown in the figure below :

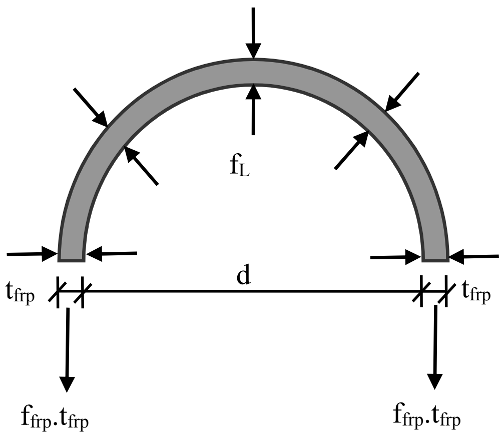


Fig.2.4: *Confinement action of FRP Jacket in circular sections [6].*

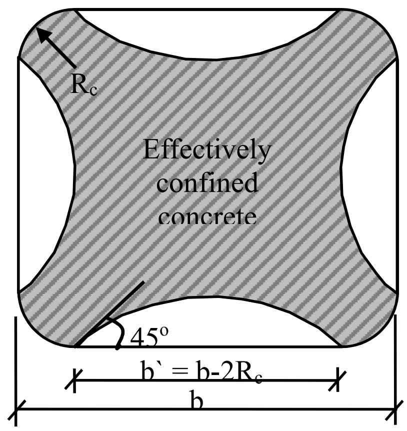


Fig. 2.5: *Confinement action of FRP Jacket in square sections [14].*

The relation between the strength of unconfined concrete & the confined concrete in a FRP wrapped square column as proposed by the author,

$$\frac{f_{cc}'}{f_{co}'} = 1 + k_1 k_e \frac{f_{L,eff}}{f_{co}'} = 1 + 0.58\frac{f_L}{f_{co}'} \text{........(2.3.6)}$$

Where, k_1 = 1.60, as defined in case of uniformly confined concrete; k_e was found to be 0.36.
The strain at peak stress was given as,

$$\varepsilon_{cc} = \varepsilon_{co}(2 + k_2 k_{e2}\frac{f_L}{f_{co}'}) = \varepsilon_{co}(2 + 4\frac{f_L}{f_{co}'}) \text{.......(2.3.7)}$$

Where, k_2 = 5.55, as defined in case of uniformly confined concrete & k_{e2} = 0.72.

2.4 Nonlinear Analysis of Fiber wrapped frames

Eslami and Ronagh (2013) [16] carried out an analytical study to investigate the enhancement of the structure at their global level when subjected to dynamic loadings. They had analysed two 8 storey 2D RC frames with different stirrup confinement before and after retrofitting with GFRP laminates. The original structure was retrofitted with GFRP sheets wrapped around the members at their critical regions for energy dissipation. Nonlinear static analysis of all the frames, i.e., the existing one and the retrofitted one has been performed using SAP2000 with manually defined hinge parameters and lumped plasticity approach. The moment curvature and moment rotation relationships for the members were evaluated with the help of XTRACT software and stress strain relationship proposed by Lam and Teng [6] was used for deriving the FRP confined properties of the concrete. The retrofitting strategy used was focussed on increasing the ductility of the structure 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.

2.5 Comparative study of Confined Concrete

Stress-strain relation for various confined concrete is depicted here in this section of this report. Primarily, three types of concrete properties are discussed as mentioned in the previous sections. First, typical stress strain behaviour of unconfined concrete is depicted for various concrete grades, then, confinement effect of stirrups and hoops are discussed for rectangular, square and circular sections and finally FRP wrapped square, rectangular and circular section properties are presented.

2.5.1 Unconfined Concrete stress strain

The figure below represents the stress strain behaviour of unconfined concrete for three grades i.e., M20, M30 and M40. As we can see from the graph, the slope of the post peak descendent branch of the concrete increases as the grade is increased. The numerical relationship that has been used in deriving the concrete properties as presented in the graph is the one proposed by Wan T.Tsai [1] as described in section 2.1.

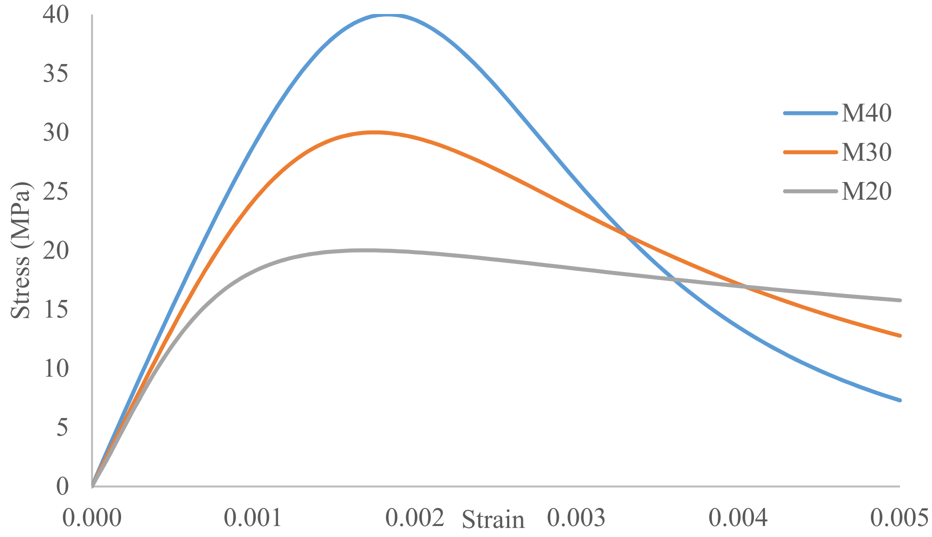


Fig. 2.6: *Stress-Strain relationship for Unconfined concrete*

2.5.2 Confined Concrete stress strain

The effect of stirrup spacing and section geometry on confinement of concrete and its stress strain relationship in depicted here in this section. Mander's confinement model [5] has been used in deriving the stress strain properties of the confined concrete.

2.4.2.1 Circular section

For determination of confined stress strain properties in a circular section with circular hoop or spiral reinforcement, a circular section with 500 mm diameter has been chosen. The longitudinal steel considered to be 8 no.s 20 dia bar of Fe415 grade. Transverse bar diameter of 10mm of the same grade has been taken in deriving the properties of the confined concrete. Spacing of the transverse reinforcements varied for 150mm, 100mm and 75mm. It was found that with spiral reinforcement, the peak confined strength of the member can be increased. Also, it was observed that the initial portion of the curve was unaffected by type of transverse reinforcement provided and their spacing. The stress strain relationship for confined concrete such derived is presented in figure 2.6 and 2.7.

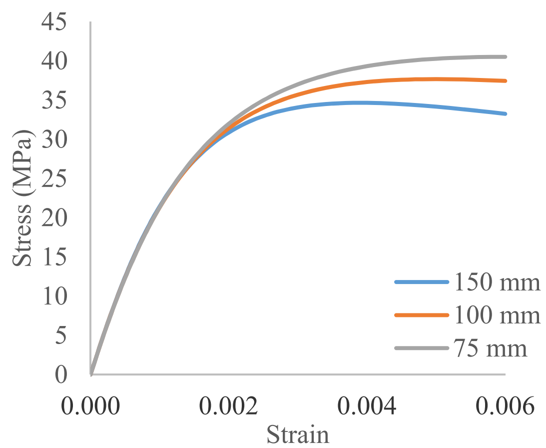


Fig. 2.7: *Stress-Strain relationship of confined concrete for circular section with various spacing of hoop reinforcement.*

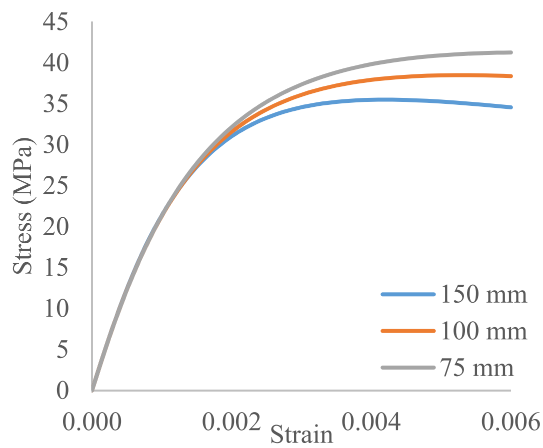


Fig. 2.8: *Stress-Strain relationship of confined concrete for circular section with various pitch of spiral reinforcement.*

2.4.2.2 Rectangular and Square section

For rectangular section, D/b ratio of 1.50 is chosen with 1.7% longitudinal reinforcement and 2 legged Fe415 grade transverse reinforcement. Stirrup spacing has been varied from 150mm to 75 mm. For square section same parameters are chosen except that the D/b ratio is taken as 1.0. For comparison with circular section, area and total longitudinal reinforcement of the sections has been taken as defined for circular section.

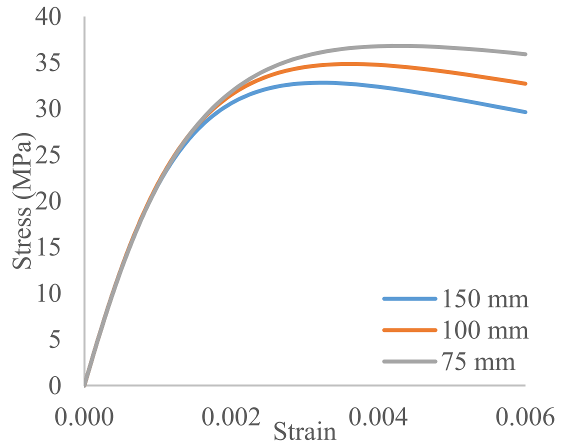


Fig. 2.9: *Stress-Strain relationship of confined concrete for rectangular section with various spacing of transverse reinforcement.*

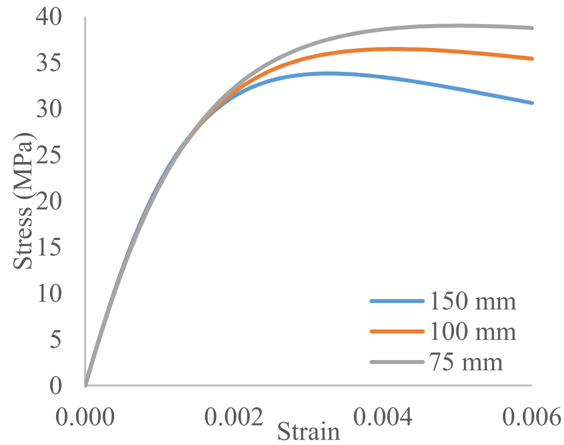


Fig. 2.10: *Stress-Strain relationship of confined concrete for square section with various spacing of transverse reinforcement.*

Comparison of peak strength of the sections

Table 2.1: *Comparison of Peak Strength for Circular, Rectangular and Square sections.*

	Circular Section						Rectangular Section			Square Section		
	Hoop Reinf.			Spiral Reinf.								
	150 mm	100 mm	75 mm	150 mm	100 mm	75 mm	150 mm	100 mm	75 mm	150 mm	100 mm	75 mm
Peak Strength	34.66	37.68	40.53	35.49	38.47	41.27	32.82	34.85	36.81	33.82	36.47	39.00

As seen from Table 2.1, for same concrete area, longitudinal and transverse reinforcement, the circular columns behave better than the square and rectangular columns.

2.5.3 Carbon Fiber wrapped Concrete stress strain

In this section, the effect of Carbon fiber wrapping has been discussed for Rectangular, square and circular section of same area. The D/b ratio for the rectangular section has been taken as 1.50. All the sections were retrofitted with 2 layers of CFRP strips. The confinement effect was observed to be most efficient followed by square and rectangular section. The stress strain relationship for the various sections are plotted below:

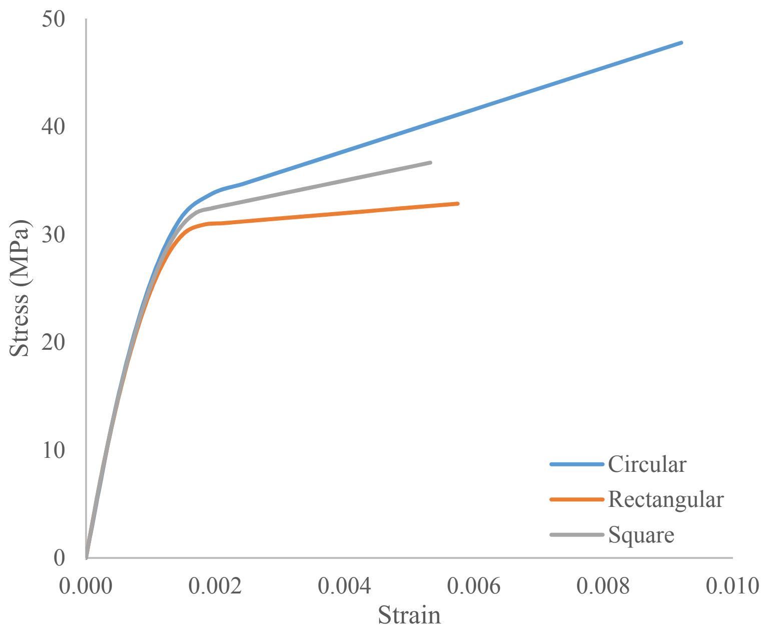


Fig. 2.11: *Stress-Strain relationship of carbon fiber confined concrete for Circular, Rectangular and Square section wrapped with 2 layers of CFRP strips.*

The peak strength of the Carbon fiber wrapped concrete is presented in the Table below:

Table 2.2: *Comparison of Peak Strength for CFRP wrapped circular, rectangular and square circular sections.*

	Circular Section	Rectangular Section	Square Section
Peak Strength (MPa)	47.79	32.86	36.67

As shown in the figure above, the confinement effect in circular sections are much more dominant than in square or rectangular shapes.

CHAPTER 3

WORK PLAN & OBJECTIVE

3.1 Objective

In the current scenario, where the number of non-engineered buildings are much more than the engineered ones, the poor construction qualities & the seismo-tectonic vulnerability of the regions calls for the utmost necessity of the strengthening measures to be taken for the existing structures. Though many research works have been done in the field of retrofitting and strengthening of existing structures, researchers are yet to concur on an appropriate and authentic yet simple design procedure for the same. Here in this study an attempt has been made to bridge the gap between the theoretical rigorous calculations and their practical application in the real life situation.

The primary aim of this study is focussed on deriving an efficient methodology for evaluating parameters of a retrofitted structure which could be applied in design office applications.

3.2 Methodology

The methodology for the proposed numerical study includes an assessment of an existing building which is to be retrofitted for seismic demand. 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. Also, the strength & ductility parameters are compared at the member level on the basis of the axial load-moment interaction diagram, Moment-curvature relationship & Moment-rotation diagram of the existing & retrofitted members.

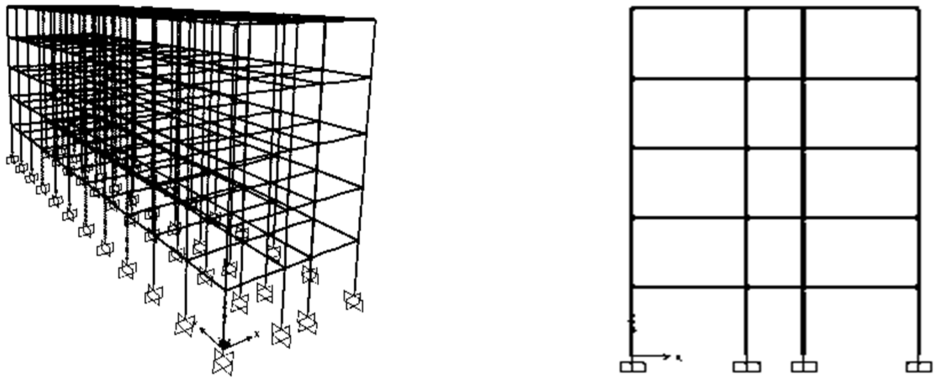
At first, the local retrofitting techniques are applied on the members of the building so as to safeguard the building against failure in seismic loading cases. The local retrofitting technique that is chosen here for the study is FRP wrapping of the members. The strength & ductility parameters of the members are compared in each of the local retrofitting technique applied. Global retrofitting techniques such as addition of steel bracings combined with local FRP jacketing of the members were also investigated upon so as to strengthen the structure efficiently.

3.3 A Numerical Study

In the present study, an existing G+4 residential building in Seismic Zone-V has been analysed for gravity & earthquake loading. Suitable measures are suggested for successfully retrofitting the structure for resisting seismic loads.

3.3.1 Existing Configuration

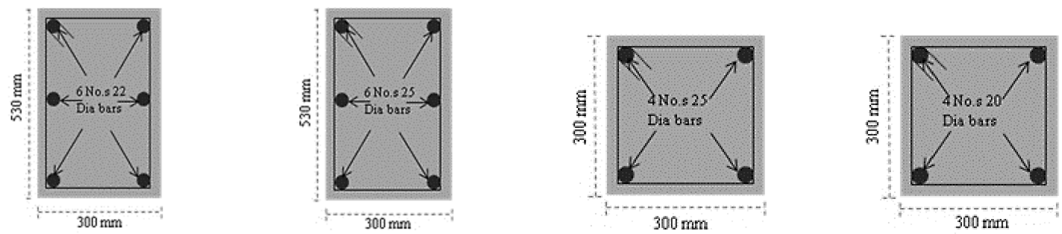
The existing configuration of the building is shown in the figure below:



(a) Isometric view of the existing building (b) Elevation of the existing building

Fig. 3.1: *Configuration of the existing building*

The sectional details of the columns are as shown below:



(a) Exterior column at lower levels. (b) Exterior column at upper levels. (c) Interior column at lower levels. (d) Interior column at upper levels.

Fig. 3.2: *Sectional details of the columns of the existing building*

3.3.2 Modelling & Analysis

To determine the seismic loading on the structure, a 3d model of the building has been analysed 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 interaction curve at failure with the help of MS Excel software package. Thus the deficiency of the building, if any, is found out. If the building is found to be deficient in Gravity/Seismic loading, suitable retrofitting measures in terms of local retrofitting (i.e. FRP wrapping) is proposed. The local retrofitting technique was found to be inadequate for the building. Next a combination of local and global retrofitting technique is proposed by addition of steel braces to the structure. The deficient columns are identified & retrofitting measure using the same approach has been proposed. The Strength (in terms of P-M interaction) & ductility parameters (in terms of Moment-Curvature & Moment rotation curves) of the existing members & the retrofitted members are compared to ascertain the increase in the parameters due to the particular retrofitting measure used.

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* [12]. The seismic demand of the structure is determined using CQC rule of response spectrum method as recommended in IS 1893-1 (2002), *Criteria for Earthquake Resistant Design of Structures, Part 1: General Provisions and Buildings* [11]. The stress strain response of the FRP confined members are ascertained using ACI 440.2R-08 [17] which uses Lam & Teng's [6] numerical model for predicting the quantity. A brief description of the building model is given in the following section.

The storey height of the building is 3.35m for each & every floor. The bay width for first & third bay is 4.60m & that is for the second bay is 2.30m. The self-weight of the members is automatically assigned using a load case in SAP2000 [8]. The dead load due to slab weight, flooring & other immovable objects were assigned using a separate load case. The seismic weight of the building was considered as Dead load plus 50% live load as per Table 8 of IS 1893:2002[11]. The effective stiffness's of the beams & columns were taken as per Table 2 of IS 15988:2013[9].

The nodes at each floor level were assigned rigid diaphragm constraint so as to simulate the effect of slabs at the 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 rigid & a length of intersection between the two members was taken as the rigid offset length.

For determination of the demand points, the following load combinations were considered,

Table 3.1: *Load combinations for assessment of existing building.*

Load Case Type	Dead Load		Live Load	Response Spectrum Load	
	Self-Weight	Dead(Addi)		EQ_x	EQ_y
DL	1.50	1.50	0.00	0.00	0.00
DL + LL	1.50	1.50	1.50	0.00	0.00
DL+LL + EQx	1.20	1.20	1.20	1.20	0.00
Dl+LL+EQy	1.20	1.20	1.20	0.00	1.20
DL+ EQx	1.50	1.50	0.00	1.50	0.00
DL+ EQy	1.50	1.50	0.00	0.00	1.50
DL+ EQx	0.90	0.90	0.00	1.50	0.00
DL+ EQy	0.90	0.90	0.00	0.00	1.50

During the response spectrum analysis of the structure, the calculated base shear by SAP2000 was compared to the manual calculation of base shear and suitable base shear correction factor applied to the load case as recommended in clause no. 7.8.2 of IS 1893:2002 [11]. Also, when applying response spectrum load on the building, a lateral load modification factor of 0.80 is applied so as to incorporate the remaining reduced design life of the structure as per IS 15988:2013 [9].

During the modal analysis, it was found that the fundamental mode shape of the building is in the longitudinal direction. The mode in the transverse direction was slightly longer in period than the first one. The reason behind this can be attributed to the asymmetric orientation of the edge columns which has a greater moment of inertia in the transverse direction of the building.

CHAPTER 4

EVALUATION OF STRENGTH & DUCTILITY PARAMETERS

The strength & ductility parameter that are compared for the study are in terms of P-M interaction, Moment-curvature & moment rotation diagram. The determination of the said sectional properties is done manually with the help of MS Excel 2007 software package. One of the primary input data that is required to determine the strength & ductility parameters of any section, is the peak strain for the materials. The peak strain data for the existing can be easily selected from the code of practices issued by BIS [12]. But, for the determination of the P-M interaction curve of a member wrapped with FRP laminates, it is of 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 in the FRP wrapped members are sudden if they are governed by the rupture of the FRP itself.

4.1 Estimation of Peak Strain of FRP wrapped members

Many researchers ([2], [4], [7]) have already proved that the peak strain capacity of the concrete material increases due to confinement actions of the FRP on the member. Many such mathematical models are also available in the research publications by these authors. However, for practical application of the particular technique, a reliable estimate of the quantity to be used in design calculations has not been provided.

The report by ACI committee 440, i.e., ACI 440.2R-08 [17], Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures provides guidelines for designing retrofitting solutions using FRP. The report provides recommendation for peak strain to be used in design calculations. It has recommended a peak strain value of 0.004 for bending members which seemed to be too conservative in lieu of the reported values of peak strain in research publications. The peak strain value for axially loaded members as recommended in the code is 0.01 which is also a bit conservative to the reported values.

Another important aspect of choosing the peak strain value for the FRP wrapped concrete material is that the internally cracked state of the member. Research activities show that with increase in strain on the sample, the internal integrity of the concrete in the member diminishes & if unwrapped at this stage, the residual strength of the concrete becomes very much less than the peak strength of the concrete even in the unconfined state. So, if very high peak strain is considered in designing the FRP wrapping it could affect the performance of the structure, as the failure mode of the FRP wrapped members are sudden.

So, the peak strain to be taken into consideration for designing a retrofitting solution using FRP composites is really a difficult task as far as the reliability of the quantity is concerned. In this study, in absence of proper codal guidelines, the peak strain of the FRP wrapped members are taken as predicted by mathematical model of Lam & Teng [6]. However, to predict the confining lateral pressure on the members, the factors such as Environmental factors, effective strain in FRP as specified in ACI 440.2R-08 [17] has been incorporated so as to make a reliable estimate of the quantities.

4.2 Determination of P-M Interaction Curve

The Axial load-Moment interaction curve of the section has been generated by dividing the section into a number of finite strips and estimating the response of every finite strip of the section for given neutral axis depth & stress-strain profile of the materials constituting the section. The number of finite strips that the section is divided into, based on a sensitivity analysis of the section. As a general rule, the size of the strips shall be such that it can be justifiably assumed as a rectangular/trapezoidal section. The stress of the section for a given neutral axis depth is evaluated at the centroid of the section & 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 that 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.

4.3 Determination of Moment Curvature Diagram

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

4.4 Determination of Moment-Rotation Diagram

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

To evaluate the plastic rotation capacity of the member, the plastic hinge length of the member is required to be known. The required mathematical relationship to predict the plastic hinge length of the member is taken as proposed by Paulay and Priestley [18]. The plastic rotation of the member is determined as:

$$\theta_p = \phi_p * L_p$$

Where, ϕ_p is defined as the plastic curvature of the section $= (\phi - \phi_y)$, ϕ_y is the yield curvature of the section.

The plastic hinge length, L_p is defined as, $L_p = 0.08L + 0.022\ d_b\ f_y$ [18]; where, L is the member length, d_b is the bar diameter & f_y is the yield strength of the reinforcing bar.

4.5 Determination of Non-linear Hinge parameters

For determination of non-linear hinge parameters, firstly, the actual stress strain behaviour of the materials has to be used. Code defined stress strain relationship of the materials are based on the lower bound values, i.e., for an exceedance probability of 5% during its lifetime. However, for nonlinear load cases, such huge factor of safety is unnecessary. For determining the nonlinear response of the structure, it is conventional to use the expected values of the materials. Various codes such as ASCE 41-13 recommends that the expected strength of the existing concrete and reinforcing steel shall be taken as 1.50 and 1.25 times the characteristic strength specified at the time of construction (Table 10.1 of ASCE 41-13). However, in the actual case, the strength of the concrete and the reinforcing steel is dependent upon the curing method, exposure conditions etc. and in some case it may well be established that the strength may get reduced from the original specified one. So, it is better to take the properties from the non-destructive or core testing results of the structure. Here, in this study, keeping both the things in mind, the actual specified properties of the materials has been used for performing nonlinear analysis of the structure.

For existing concrete, the Mander's model has been used to calculate the nonlinear member parameters, whereas, for steel reinforcing bars, a model with yield plateau and strain hardening is used. The peak strain capacity of the reinforced concrete members were limited to a strain of 0.005. Linear strain profile for the entire failure envelope of the member has been considered. The strain at peak stress of the concrete grade has been taken as 0.001789 instead of code specified 0.002. For steel, yield strength of 415 MPa and ultimate strength of 485 MPa has been considered. The failure strain of the steel reinforcing bars were taken as 0.12 in tension, whereas in compression, an elastic perfectly plastic steel model has been used with ultimate strain of 0.05. The plasticity of the members were lumped at the mid-point of the calculated plastic length. For characterization of the nonlinear hinge parameters of the CFRP wrapped columns, recommendations given in ACI 440-2R-08 [17] has been utilized. The peak strain capacity of the CFRP wrapped reinforced concrete members were taken as 0.01 as specified in the code. The stress strain relationship for the CFRP wrapped concrete has been taken considering an effective rupture strain coefficient of 0.55 and the numerical formulation given in the code which implements the relationship proposed by Lam and Teng [6].

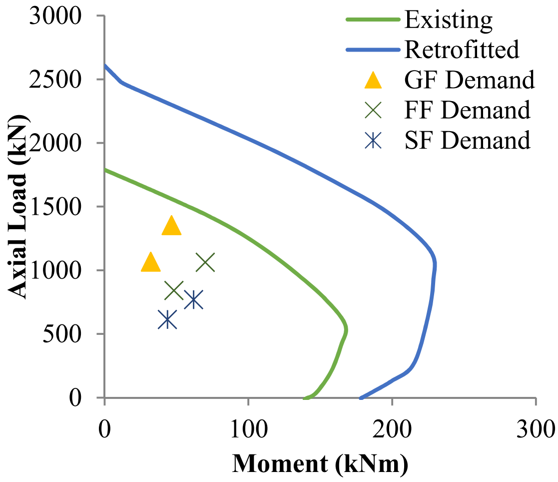
CHAPTER-5

RESULTS & DISCUSSION

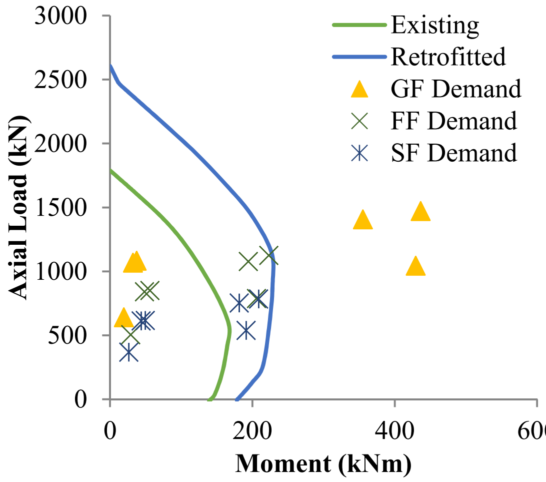
The results of the linear and nonlinear analysis of the building is depicted here in this chapter. The chapter has been subdivided into five parts. The first and the second part describes the linear analysis results of the building retrofitted with CFRP wrapping and by installation of additional steel braces. The third and fourth part discusses the nonlinear static analysis results of the same building and the issues related to it. The fifth part of this chapter illustrates the member level enhancements due to CFRP wrappings.

5.1 Linear Analysis with FRP Jacketing

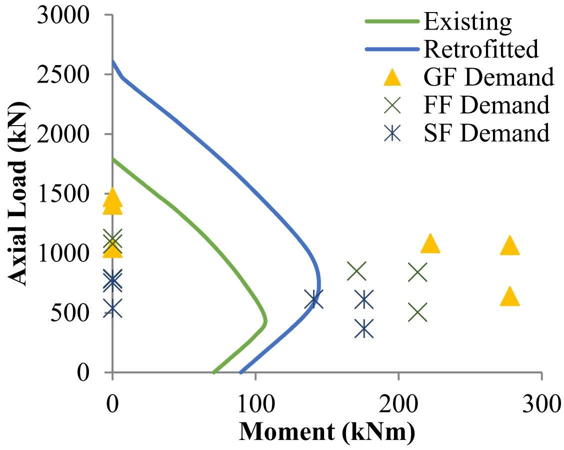
The exterior rectangular columns in lower floors are found to be safe under gravity loading combinations. However, the interior square columns are slightly deficient even in the gravity loading. After analysis of the building for seismic loading cases, both the interior and the exterior columns of the building at lower levels (ground, first and second floor) are found to be deficient. The columns are now retrofitted with 5 layers of CFRP strips and the results are plotted against the demand points of the building. It is observed that even with 5 layers of CFRP strip wrapping, the enhancement in the columns strength is not sufficient. The members at the lower levels are also deficient in the shear capacity. However, with CFRP strip wrapped around the members, the shear demand on these members is successfully mitigated. In the upper floor levels, i.e. in third and fourth floor levels, the existing columns are found to be sufficient as far as the gravity loading is concerned. However, in the seismic loading combinations, the columns are highly deficient both for shear and moment demand even retrofitted with 5 layers of CFRP wrapping. The existing and the retrofitted capacities of the member along-with the demand points in various gravity and seismic loading cases at various levels are shown in Figure 5.1:



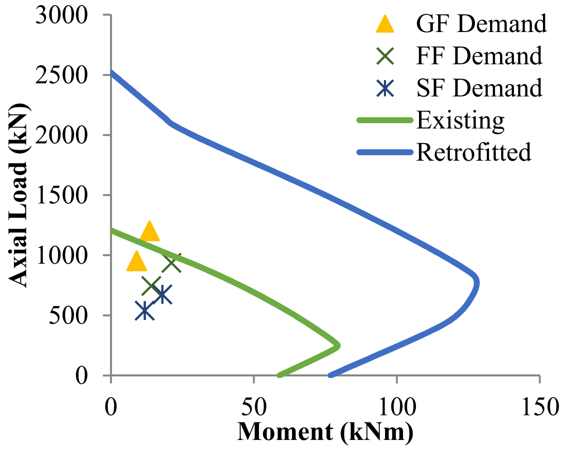
(a) *Axial load-Moment interaction for exterior column (Ground, First and Second floor) for DL and LL case.*



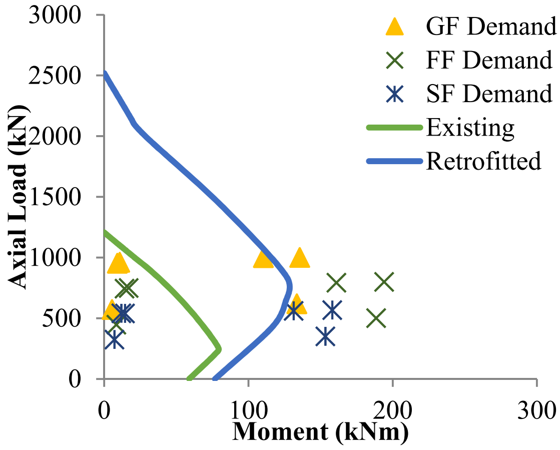
(b) *Axial load-Moment interaction for exterior column (Ground, First and Second floor) for Transverse seismic case.*



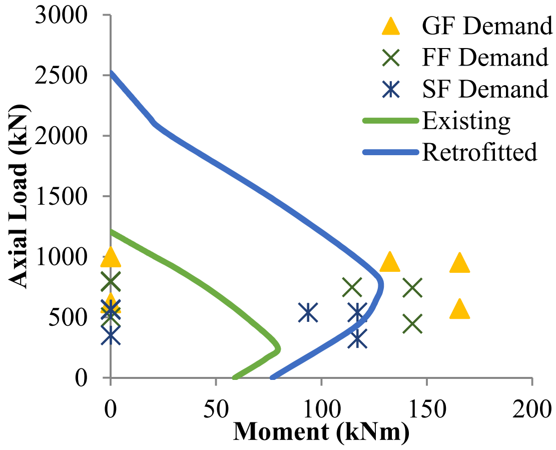
(c) *Axial load-Moment interaction for exterior column (Ground, First and Second floor) for Longitudinal seismic case.*



(d) *Axial load-Moment interaction for interior column (Ground, First and Second floor) for DL and LL case.*

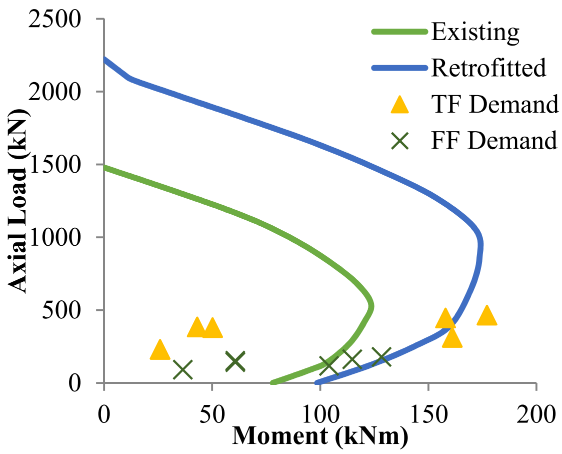


(e) *Axial load-Moment interaction for interior column (Ground, First and Second floor) for Transverse seismic case.*

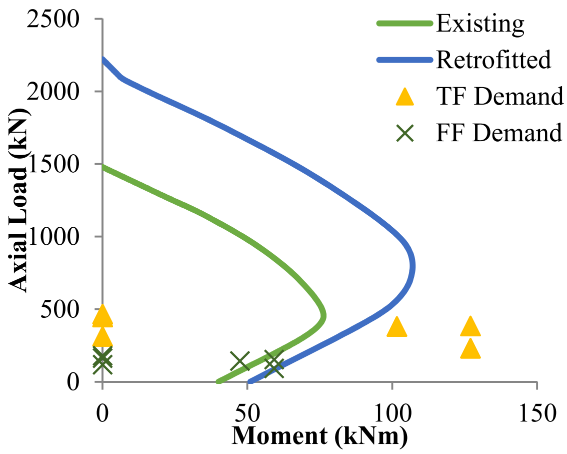


(f) *Axial load-Moment interaction for interior column (Ground, First and Second floor) for Longitudinal seismic case.*

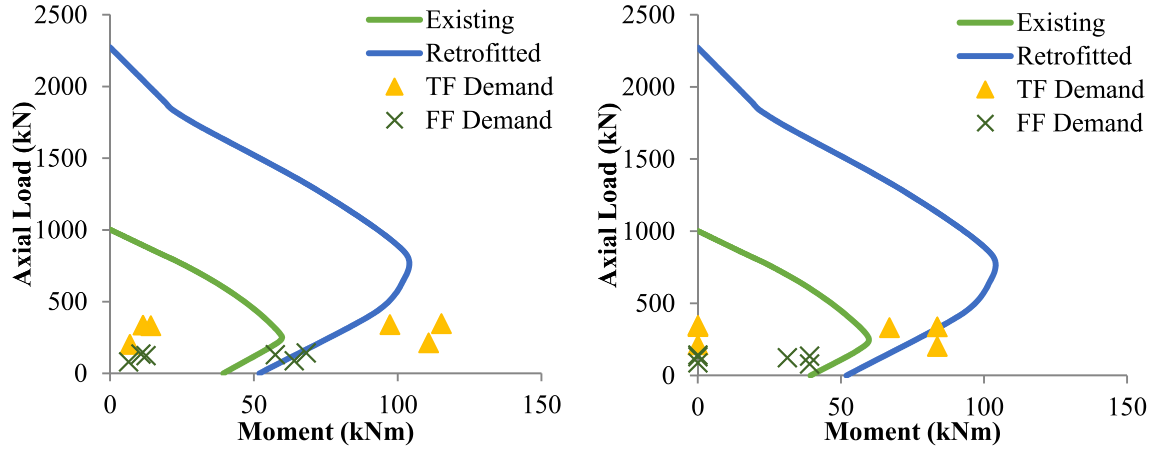
Fig. 5.1: *P-M Interaction diagram for Existing and Retrofitted columns at lower level with Demand points plotted for various load combinations.*



(a) *Axial load-Moment interaction for exterior column (Third and fourth floor) for Transverse seismic case.*



(b) *Axial load-Moment interaction for exterior column (Third and fourth floor) for Longitudinal seismic case.*



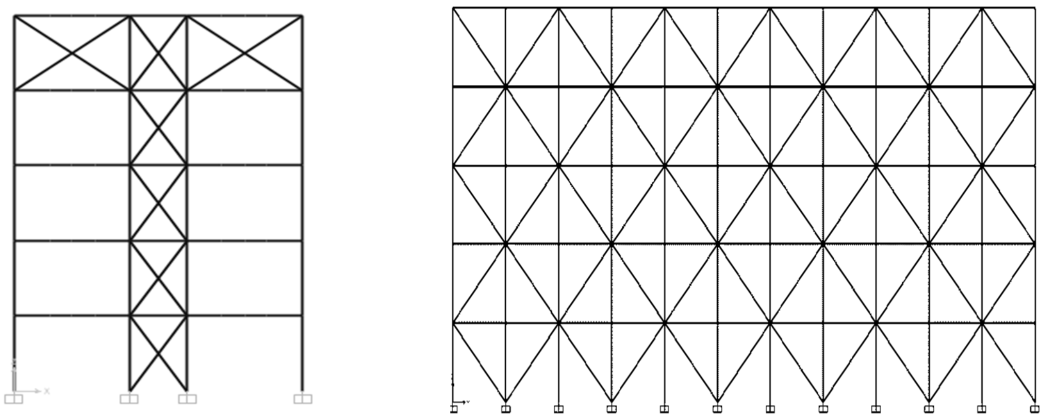
(c) *Axial load-Moment interaction for interior column (Third and fourth floor) for Transverse seismic case.*

(d) *Axial load-Moment interaction for interior column (Third and fourth floor) for Longitudinal seismic case.*

Fig. 5.2: *P-M Interaction diagram for Existing and Retrofitted columns at upper level with Demand points plotted for various load combinations.*

5.2 Linear Analysis with Steel Braced Building

From the results discussed in Section 5.1, it is quite evident that the mere use of FRP jacketing of member is not able to increase the strength and ductility of buildings at the desired level. An additional diagonal bracing system also needs to be used as retrofitting measure for the same building along-with FRP jacketing of member level. A diagonal bracing system in either direction is shown in Figure 5.3. Configuration of the bracing member is such that it increases the load on the interior columns to take maximum advantage of FRP jacketing.

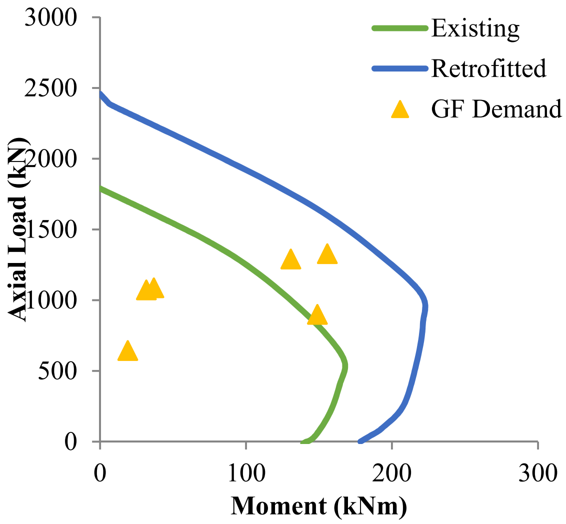


(a) Elevation of the building showing bracings in the transverse direction.

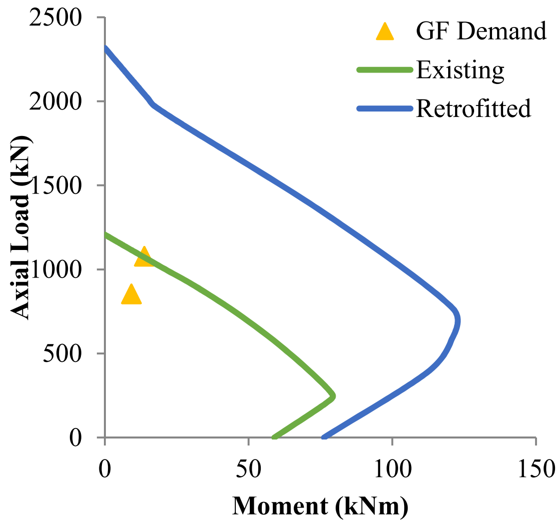
(b) Elevation of the building showing bracings in the longitudinal direction.

Fig. 5.3: *Configuration of building retrofitted with steel bracings.*

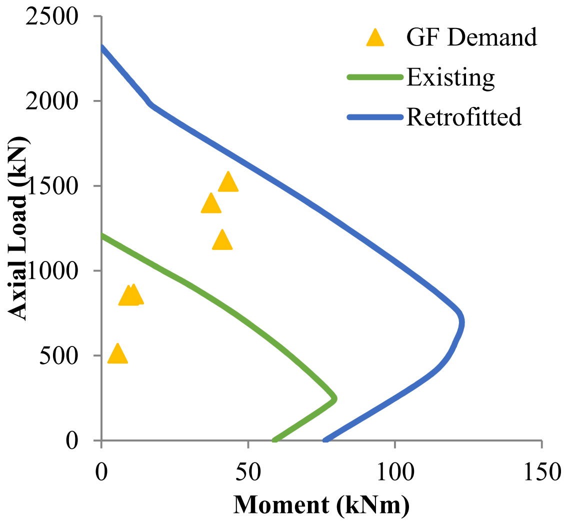
The ISMB 100 is used as the bracing member in the structure. After analysing the braced retrofitted structure, it is observed that the demand on the columns in terms of moment and shear forces are reduced. The exterior rectangular column at the lower floor levels except at the ground floor, is sufficient for the imposed gravity and seismic loading. The exterior column at the ground floor is still deficient under the case of transverse seismic loading and is subsequently retrofitted with 3 layers of CFRP strip. As anticipated, the interior columns at the ground floor level are more stressed in terms of axial loading and moment demand. The interior column at ground floor level are successfully retrofitted with 4 layers of CFRP wrapped around the member. The interior columns at the first floor level are also overstressed in braced configuration and retrofitted with 2 layers of CFRP wrapped. Both the interior and the exterior columns at upper levels i.e., at third and fourth floor levels are found to be safe under the given seismic loading conditions. The P-M interaction curves for columns which require additional jacketing of various level of the braced structure is depicted in the Figure 5.4.



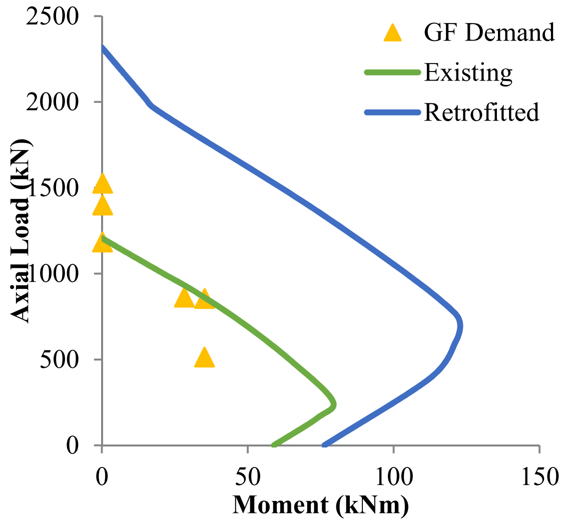
***(a)** Axial load-Moment interaction for exterior column (Ground floor) for Transverse seismic case.*



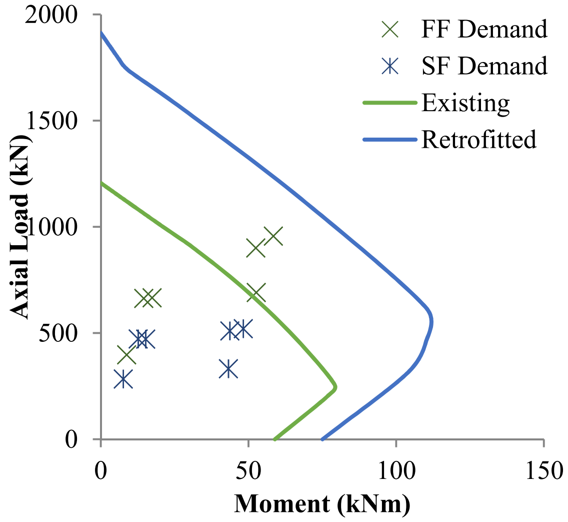
***(b)** Axial load-Moment interaction for interior column (Ground floor) for DL and LL.*



***(c)** Axial load-Moment interaction for interior column (Ground floor) for Transverse seismic case.*



***(d)** Axial load-Moment interaction for interior column (Ground floor) for longitudinal seismic case.*



***(e)** Axial load-Moment interaction for interior column (First and second floor) for Transverse seismic case.*

***Fig. 5.4:** P-M Interaction curve with demand points for columns of the braced structure.*

5.3 Nonlinear Static Analysis of FRP wrapped building

Pushover analysis is basically a Non-linear static analysis which is to be performed so as to estimate the structure's behaviour under dynamic load conditions e.g., in seismic load cases. The 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 Non-linear dynamic analysis or 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.

The non-linear static analysis of the FRP wrapped structure has been performed so as to ascertain the effectiveness of the retrofitting technique under global load applications. The non-linear hinge parameters for the existing as well as the retrofitted members of the structure were determined through manually developed Microsoft excel worksheets. Then the hinge parameters were given in SAP2000 & the non-linear static analysis of the structure has been performed.

Pushover analysis has been done for three states of the building, i.e., the existing one, second is the existing with its columns retrofitted with CFRP strips and the last one being the existing building with both its beams and columns retrofitted with CFRP jackets. The analysis has been done on the 2d frame structure of the building since the P-M2-M3 hinges were required for the complete 3d analysis of the building. The analysis were done for both the direction of the building. The pushover analysis reveals that the building fails by forming hinges in the beam first and then in the columns of the ground storey. Most of the hinges formed in the beams and columns were in the collapse prevention limit. A single hinge was observed to be in collapse prevention to collapsed state. The peak displacement and base shear force of the building was found to be 0.485 m and 277 kN. In the longitudinal direction, the peak displacement was observed to be 0.5075 m and maximum base shear as 834 kN.

Since the perimeter frames and the interior frames are different, analysis has been done for both type of frames. As an overall representation of the nonlinear analysis, it can be said that for typical building frames such as where strong column weak beam mechanism is expected at failure point, no significant increase in the pushover curve can be observed. Analysis has also been performed to observe the effect of frame size on the retrofitting technique.

Analysis of a typical transverse frame of the building shows that there is hardly any increase in strength and ductility of the retrofitted frame over the existing one where only columns are wrapped with CFRP strips. This can be attributed to the fact that, the nonlinear behaviour of the frame is primarily influenced by the yielding of the beams. In the second case of retrofitting, beams were also retrofitted with 2 layers of CFRP strips. As the behaviour of the frame is beam dominated, the ductility of the frame was observed to be greatly enhanced. An increase of 81% in the peak displacement of the frame was noticed. The increase in the strength of the frame in resisting lateral force however, was nominal. A mere 7% increase was recorded. As a matter of fact, the initial and the post yielding stiffness of the frame was same for all the three cases were identical. This is due to the property of FRP strips which doesn't impart any stiffness to the existing system, only increases its ductility. The results of the nonlinear static analysis of the building frames are presented in the table below:

Table 5.1: *Comparison of Pushover results for Existing & Retrofitted frames.*

Frame	Existing		Retrofitted columns		Retrofitted all	
	Disp. (mm)	Base Shear (kN)	Disp. (mm)	Base Shear (kN)	Disp. (mm)	Base Shear (kN)
Transverse Dir. (X)	484.90	277.00	503.40	278.40	878.90	297.40
Long. Dir. Peri. (Y)	507.50	834.10	509.50	850.30	843.10	898.00
Long. Dir. Inter. (Y)	482.10	780.60	478.80	816.10	816.20	853.90

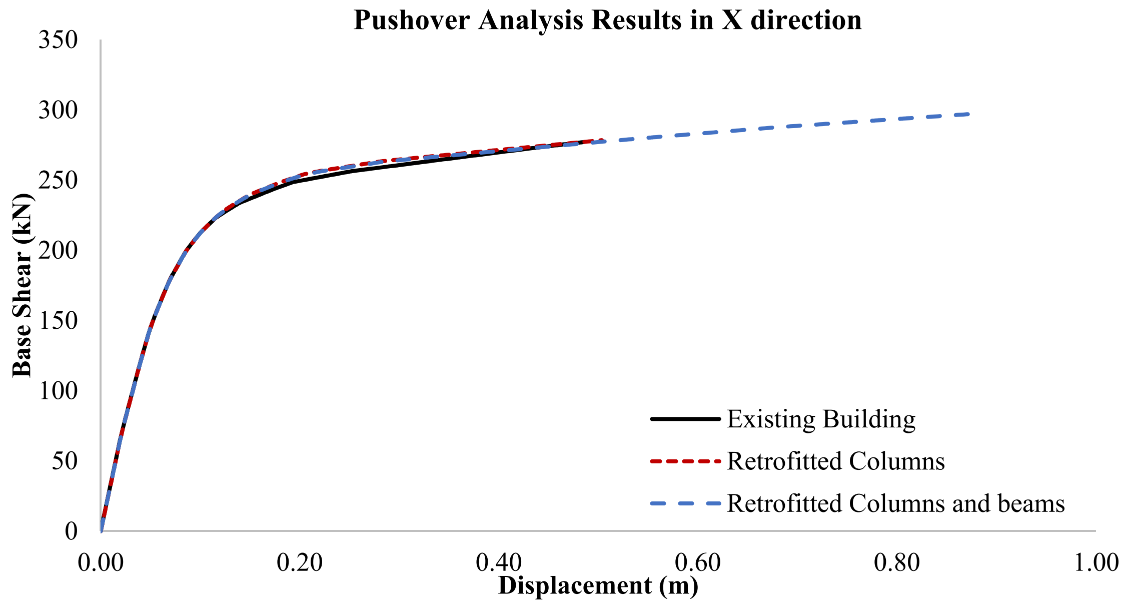


Fig. 5.5: *Pushover curve for the CFRP wrapped Transverse building frame.*

The analysis of the longitudinal frames both the perimeter and interior frames gives a slightly better though insignificant results for column retrofitting. However, as the number of columns participating in the failure mechanism is greater than the one in transverse frames, a small amount of ductility enhancement was observed in the frames. When the beams of the frames retrofitted with 2 layers of CFRP strip along with the wrappings in the column, their peak displacement was found to be significantly increased by 66-69% than the existing frame. The initial stiffness of the frame was observed to be identical for the three cases analyzed, whereas, for post yielding performance of the building frame, a slight enhancement was observed. The pushover curves generated for the perimeter and the interior frame of the structure are presented in the figure below:

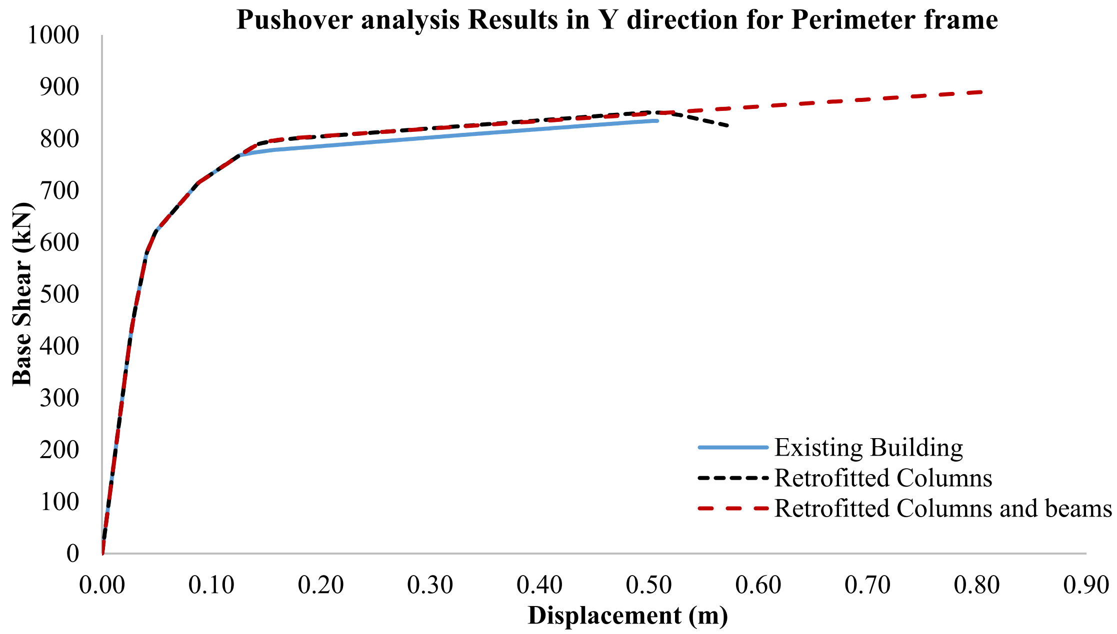


Fig. 5.6: *Pushover curve for the CFRP wrapped perimeter frame of the building.*

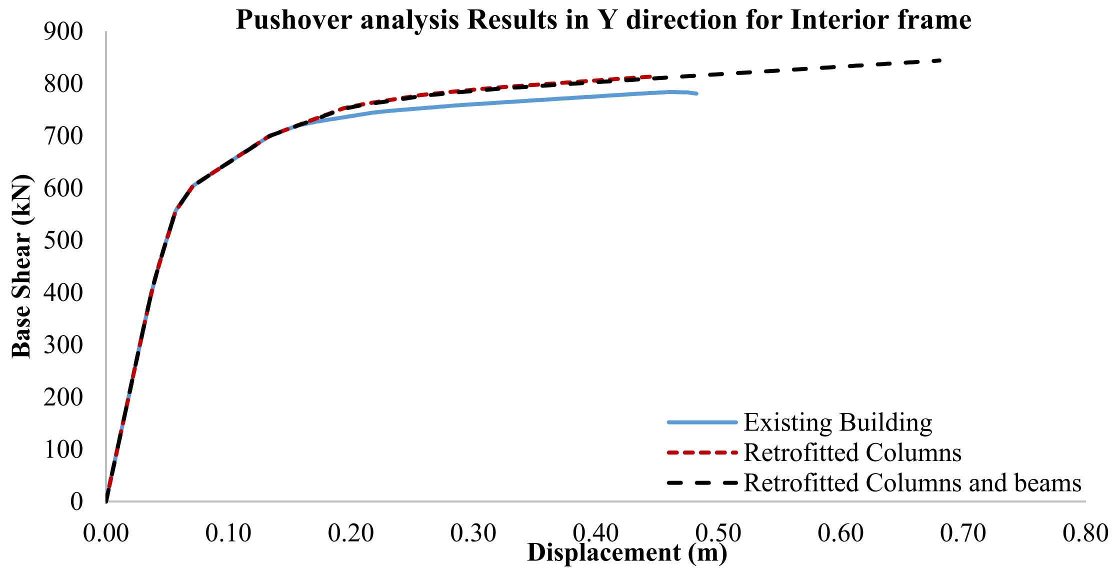
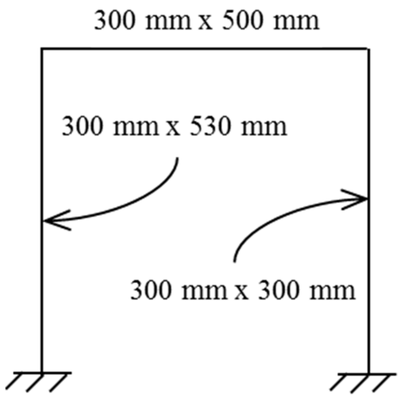
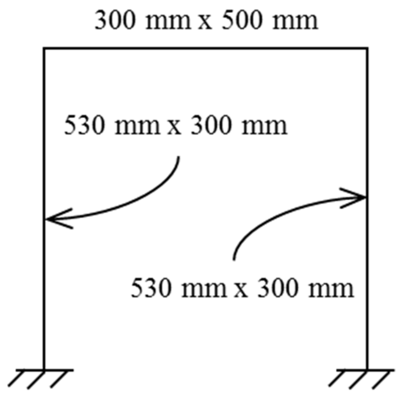


Fig. 5.7: *Pushover curve for the CFRP wrapped interior frame of the building.*

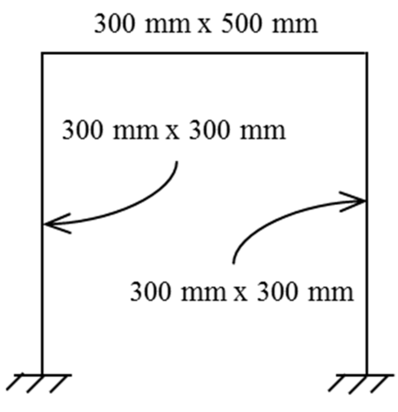
Simple portals has also been analyzed to see the effects of CFRP wrapping on various scales of the structures. Three types of portal has been examined. Each of which is a constituent of three building frames analyzed. The first portal that has been analyzed, consisted of a 530 mm x300 mm column on one side, a 300 mm x 300 mm column on other side. As per the results obtained, retrofitting the portal's columns with CFRP wrapping, can increase the overall ductility of the structure. This is due to the fact that columns participation in the nonlinear behaviour of the portal is more as compared to the building frame. The strength enhancement in all the cases are found to be insignificant the reason for which can be attributed to the low axial load on the columns of the Portal. Wrapping the beams with additional 2 layers of CFRP strips improved the ductility capacity of the frame by quite a significant amount. The peak displacement capacity of the portal increased by 23% and 97% when columns retrofitted and beams and columns both are retrofitted respectively. The figure below shows the pushover analysis results for the frame. The initial stiffness of the structure was found to be identical for all the three cases. However, the post yield stiffness of the retrofitted structure and the existing one was found to be a little different. The increase in base shear carrying capacity in columns retrofitting case is almost negligible. A minor increase of 9.25% was observed when both beams and columns of the portal were retrofitted. A schematic diagram of the portals are shown in the fig. below:



(a) Sectional details of Portal 1



(b) Sectional details of Portal 2



(c) Sectional details of Portal 3

Fig. 5.8: *Sections of the Portal frames analyzed.*

A comparison of the base shear and displacement values of the portals tested are depicted in the table below:

Table 5.2: *Comparison of Pushover results for Existing & Retrofitted Portals.*

Portal	Existing		Retrofitted columns		Retrofitted Columns and Beams	
	Disp. (mm)	Base Shear (kN)	Disp. (mm)	Base Shear (kN)	Disp. (mm)	Base Shear (kN)
Portal 1	109.90	186.60	135.50	186.90	217.50	203.90
Portal 2	130.00	136.10	134.70	138.20	211.30	144.90
Portal 3	141.70	122.37	139.22	131.22	221.50	139.45

The pushover plots of the Portal frames are shown in the figures below:

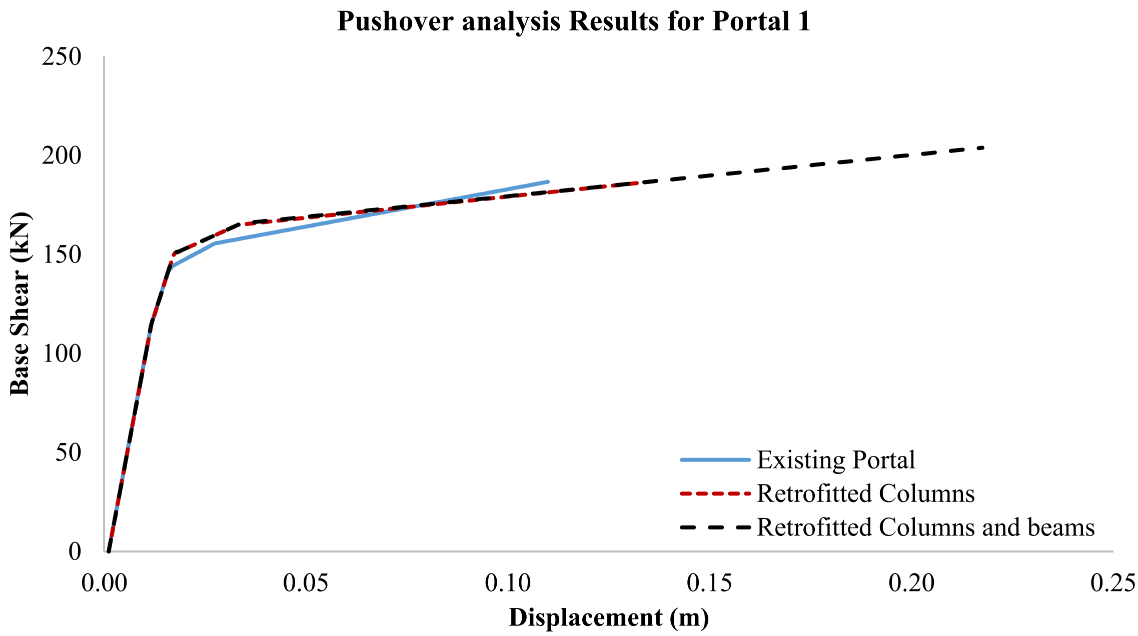


Fig. 5.9: *Pushover curve for the CFRP wrapped portal1.*

The other two portal that has been analyzed shows that the base shear capacity of the portal frames can be increased particularly for those structures where strong beam weak column mechanism is expected. The loading condition for the portals are same as the loading conditions of the main building frame. The portal frame with square columns on both side shows a substantial increase of 7.23% when only its columns were wrapped with CFRP strips. The increase in base shear capacity was enhanced by 14% when both its beams and columns were retrofitted with CFRP strips. Whereas, for Portal 2, the increase in base shear capacity when the columns and the beams and columns were retrofitted is insignificant. The base shear capacity of the Portal increased only by 6.4% when both the beams and columns are wrapped with CFRP strips. The pushover curves for both the portals are shown below:

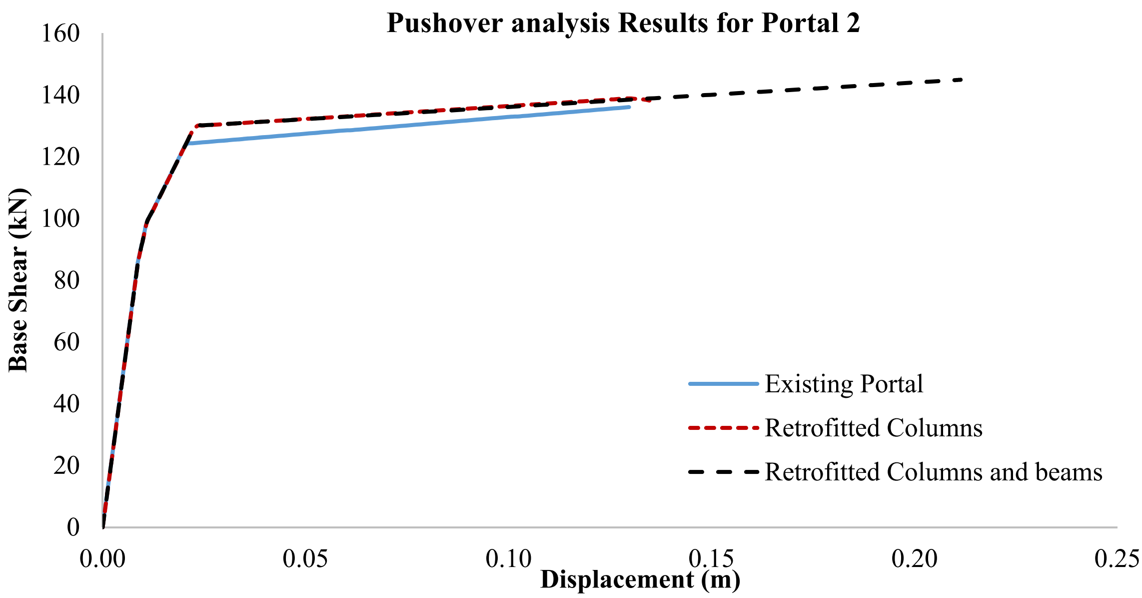


Fig. 5.10: *Pushover curve for the CFRP wrapped portal2.*

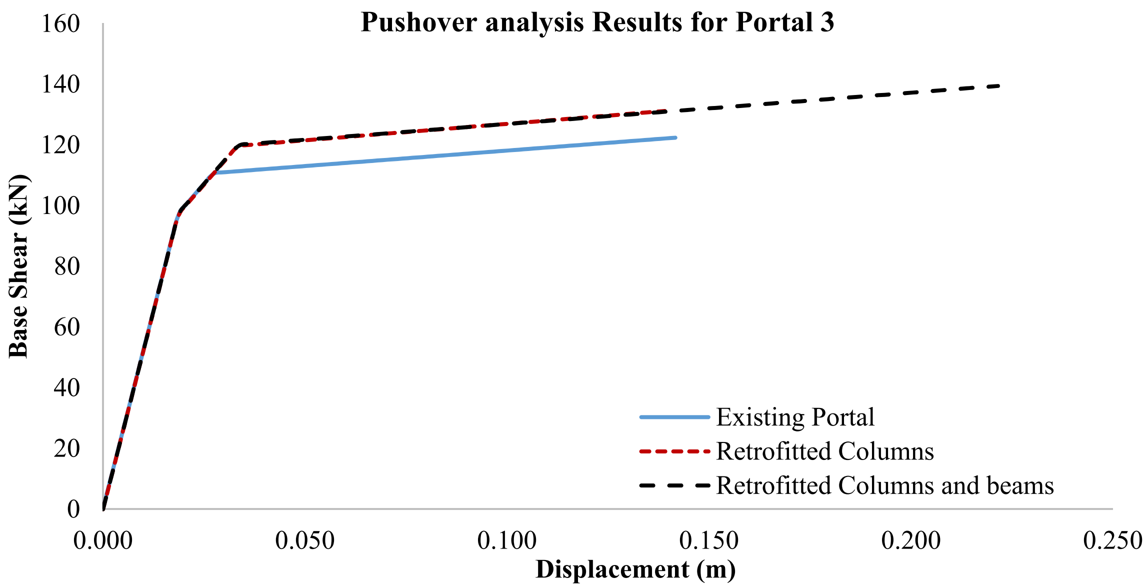


Fig. 5.11: *Pushover curve for the CFRP wrapped portal3.*

5.4 Nonlinear Static Analysis of Braced Building

Installation of additional steel braces in the building can retrofit the building in an efficient way. Linear analysis of the building with steel braces installed stiffens the structure and an alternate load path can be generated so that the lateral loads coming on the structure can be safely transferred to the ground through braces. When steel braces are installed in a building, the axial forces in the braces are too large and the other members get relieved from the loads. However, an inherent disadvantage of this type of retrofitting is that the members at the upper floor level are more stressed than the lower ones as the behaviour of the braced building is somewhat similar like shear wall buildings. As we know that for normal RC frame buildings, the structural members of the upper levels are designed for much lesser force and have much lesser reinforcing bars than the lower ones, additional measures are to be ensured so that these member do not get overstressed due to steel bracings.

As seen from the nonlinear load case analysis of the steel braced building, the failure mechanism start to form from the yielding of the upper level members. Preliminarily, steel braces only at the middle bay of the building frame was considered. But later, steel braces at the end bays of the building at the uppermost floor was also installed to prevent the failure of the upper floor beams. Installation of additional steel braces changes the behaviour of the moment resisting frame building as a whole. If very large section for braces has been chosen, the steel brace will never yield and the required ductile behaviour of the frame cannot be achieved. However, the base shear capacity of the building get increased by quite a significant amount. From the analysis it is seen that installing ISMB 100 enhances the base shear capacity of the frame by 65%. Whereas, the peak displacement capacity of the frame reduced to only 72 mm which is only 15% of the existing peak displacement capacity of 484 mm. This is due to the fact that the ground floor columns of the frame which are connected to the bracing members failed due to tensile forces and failure under tension for a RC member is always brittle. The limited ductility that is available is due to the yielding of braced members and the beams. The failure of the frame as seen from the pushover analysis starts from the yielding of the braces at the ground floor level. The formation of hinges in the beams stats only when most of the braces are yielded.

Pushover analysis of the braced building with FRP wrapped on ground and first floor columns were also investigated. The braced building with 3 layers, 4 layers and 2 layers of CFRP wrapped around ground floor exterior, ground floor interior and first floor interior columns was studied for this purpose. The failure initiation of the FRP wrapped braced building was observed to be somewhat similar to the braced one. The peak displacement capacity of the FRP wrapped building was found to be 82 mm and the peak base shear capacity to be 478 kN, which is around 73% enhancement over the existing RC frame. The pushover curves of the braced buildings are shown in the figure below:

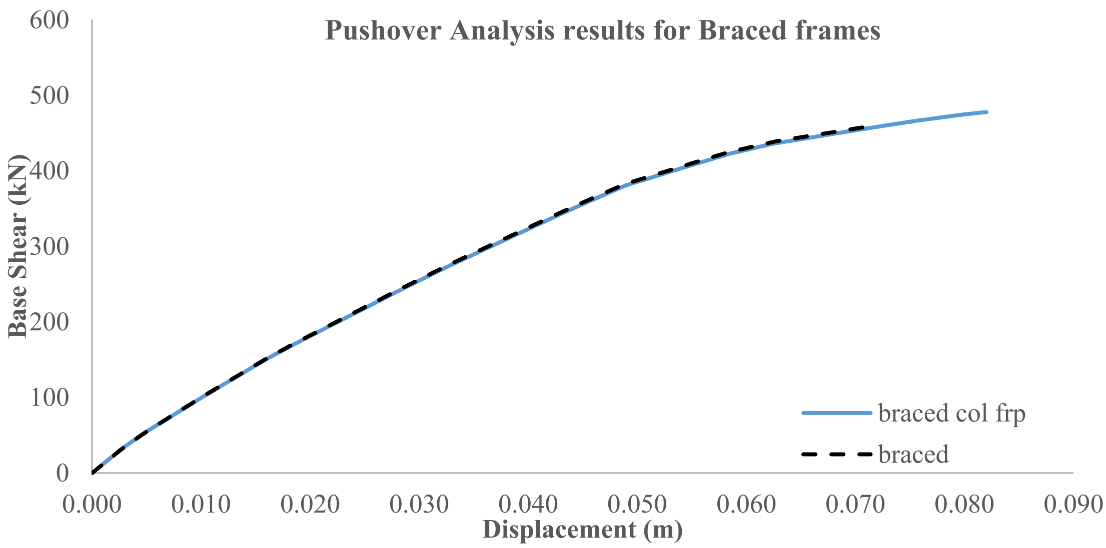


Fig. 5.12: *Pushover curve for the Braced and FRP wrapped braced building frame.*

5.5 Member level analysis for CFRP wrapping

The enhancement in strength and ductility parameters of the member wrapped with CFRP strips are compared here in this section. The strength and ductility parameters which are compared are, peak axial strength of the column, moment curvature, moment rotation properties and nonlinear hinge definition of the member.

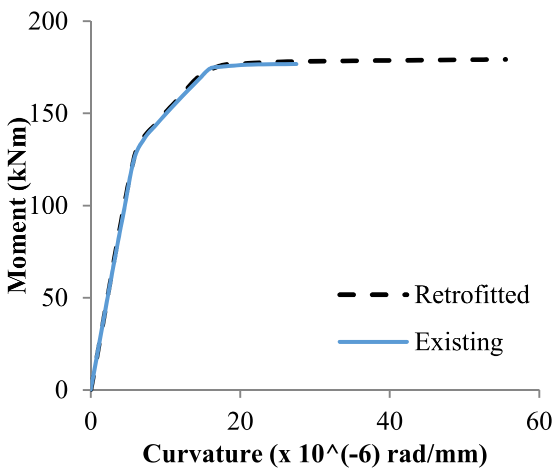
5.5.1 Strength Parameter comparison

The CFRP strips found to be very much efficient in enhancing the peak axial strength of the columns. However, the increase in strength capacity of the member was observed to be dependent on the sectional geometry of the member. In square columns, the effect of wrapping was much more pronounced than it was in rectangular columns. The increase in peak axial strength for rectangular exterior columns was found to be 29.13%, 37.41% and 45.68% for 1, 3 and 5 layers of CFRP wrapping over the member. Whereas, for interior square columns, the enhancement was 58.49%, 91.98% and 108.73% respectively for 2, 4 and 5 layers of FRP jacketing. The maximum layers of jacket was limited to five as the bonding failure of the FRP layers and the failure of the epoxy layer in between weren't taken into consideration.

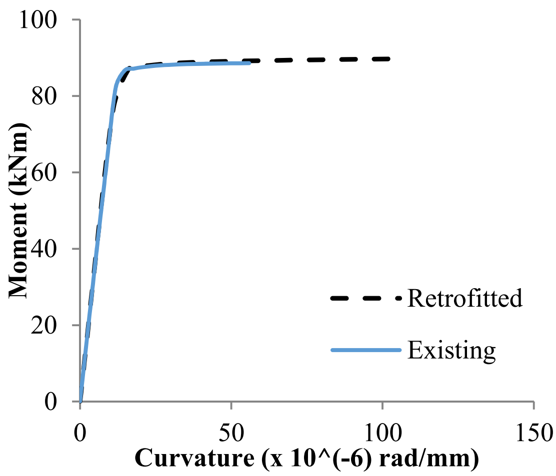
5.5.2 Ultimate Curvature and Plastic rotation capacity of the members

The ultimate curvature and the plastic rotational capacity of the members depicts the capability of the member to undergo deformation beyond yielding. The ultimate curvature and the plastic rotational capacity of the member is represented here by means of moment-curvature and moment rotation diagrams. Figure below shows the enhancements in the ductility parameters for the members:

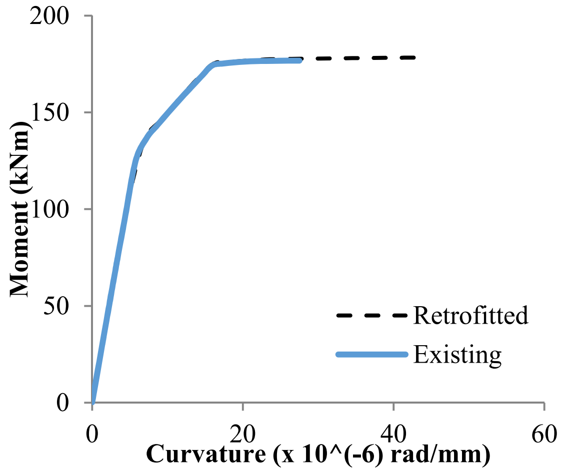
For exterior rectangular columns:



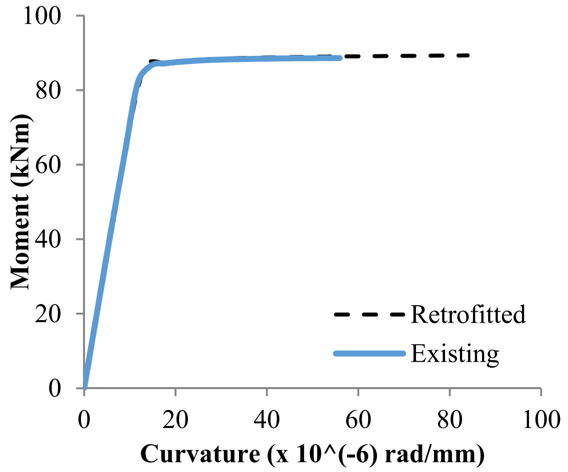
(a) Moment curvature diagram for rectangular column (Long direction) with 5 layers of CFRP.



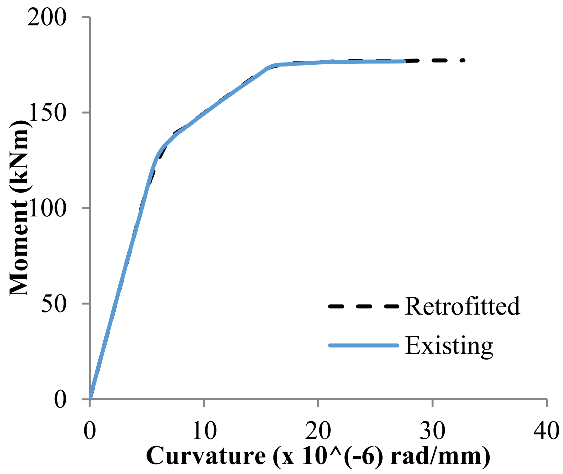
(b) Moment curvature diagram for rectangular column (Short direction) with 5 layers of CFRP.



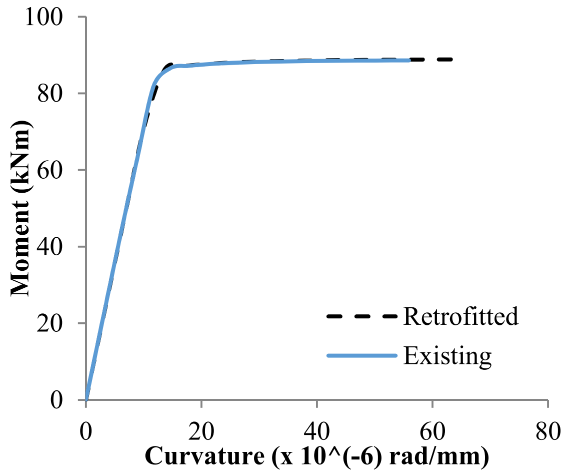
(c) Moment curvature diagram for rectangular column (Long direction) with 3 layers of CFRP.



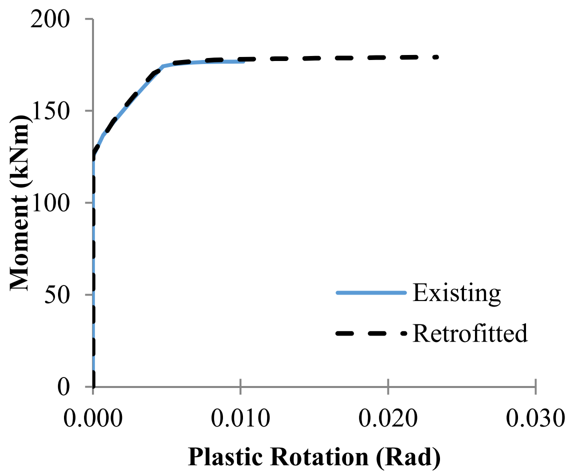
(d) Moment curvature diagram for rectangular column (Short direction) with 3 layers of CFRP.



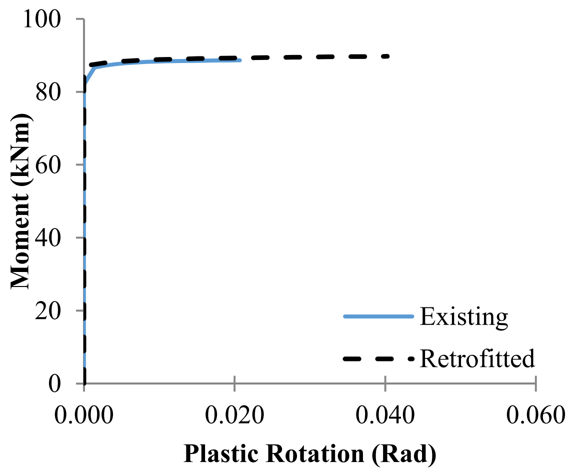
(e) Moment curvature diagram for rectangular column (Long direction) with 1 layer of CFRP.



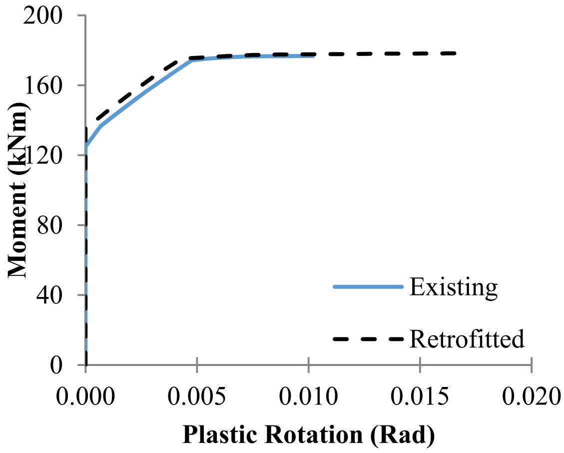
(f) Moment curvature diagram for rectangular column (Short direction) with 1 layer of CFRP.



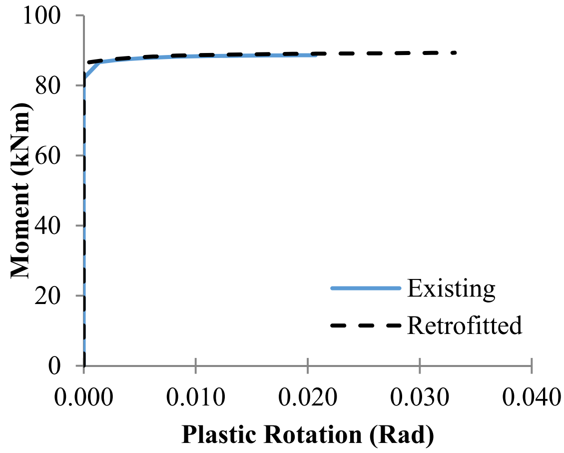
(g) Moment rotation diagram for rectangular column (Long direction) with 5 layers of CFRP.



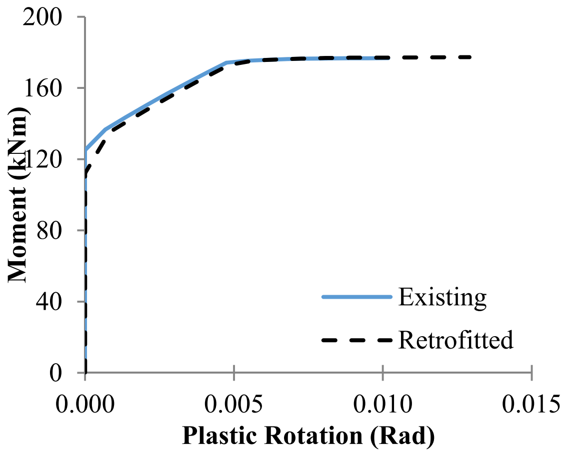
(h) Moment rotation diagram for rectangular column (Short direction) with 5 layers of CFRP.



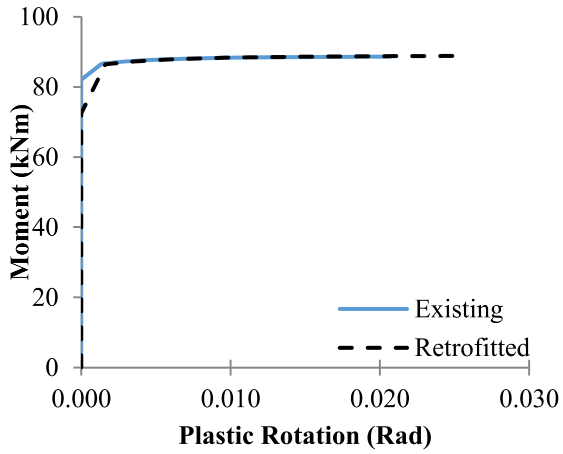
(i) Moment rotation diagram for rectangular column (Long direction) with 3 layers of CFRP.



(j) Moment rotation diagram for rectangular column (Short direction) with 3 layers of CFRP.



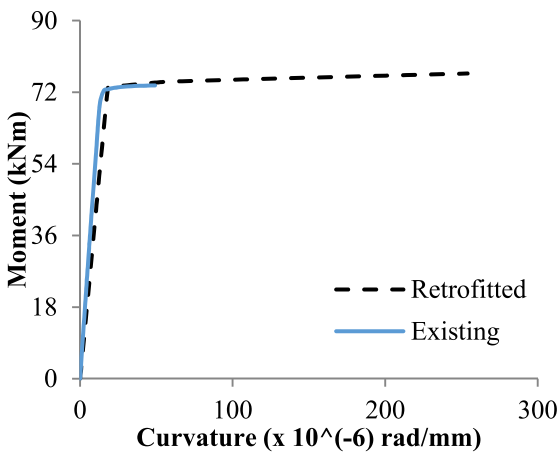
(k) Moment rotation diagram for rectangular column (Long direction) with 1 layer of CFRP.



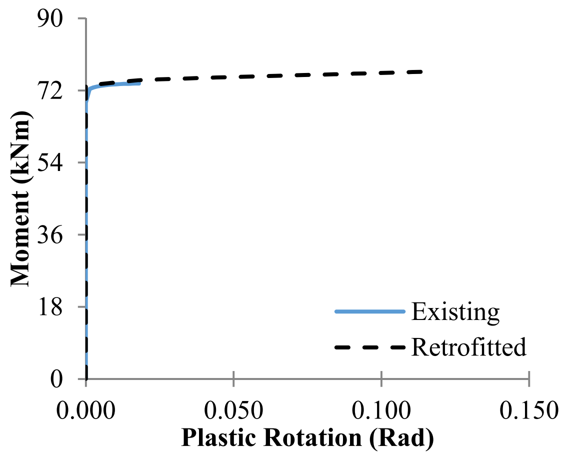
(l) Moment rotation diagram for rectangular column (Long direction) with 1 layer of CFRP.

Fig. 5.13: *Comparison of Ductility parameters for Exterior Rectangular Columns.*

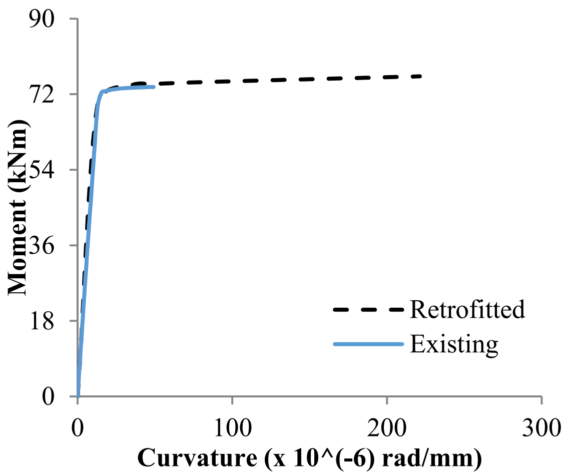
For interior square columns:



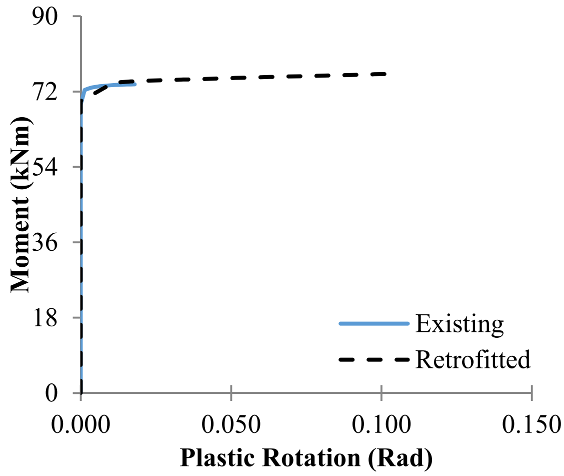
(a) Moment curvature diagram for square column with 5 layers of CFRP.



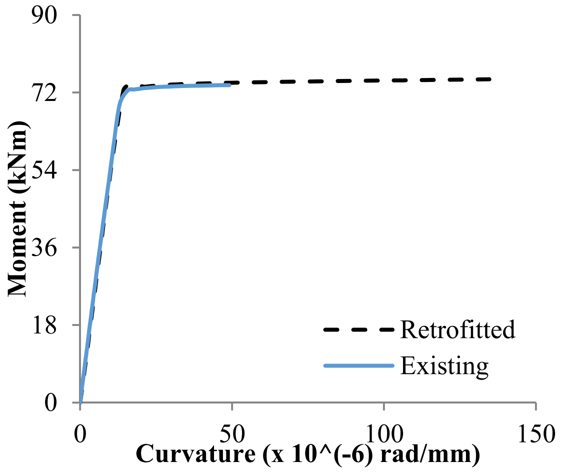
(b) Moment rotation diagram for square column with 5 layers of CFRP.



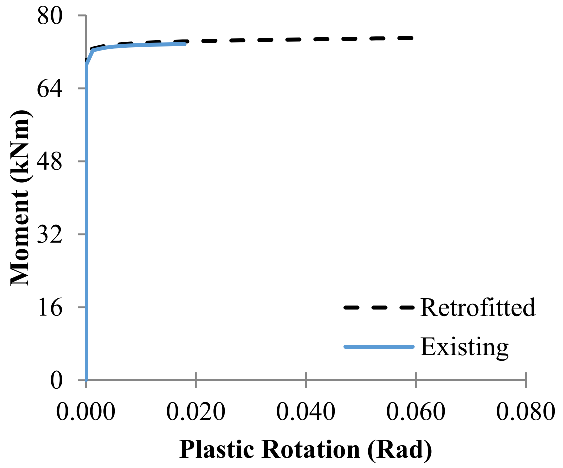
(c) Moment curvature diagram for square column with 4 layers of CFRP.



(d) Moment rotation diagram for square column with 4 layers of CFRP.



(e) Moment curvature diagram for square column with 2 layers of CFRP.



(f) Moment rotation diagram for square column with 2 layers of CFRP.

Fig. 5.14: *Comparison of Ductility parameters for Interior Square Columns.*

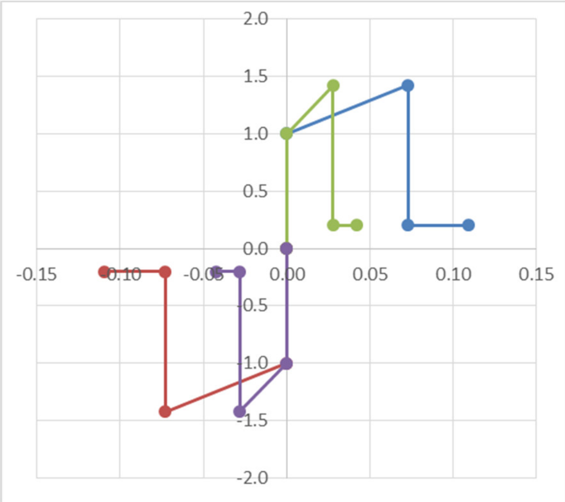
Increase in ductility parameters for rectangular and square columns:

Table 5.3: *Comparison of Ductility parameters of the columns.*

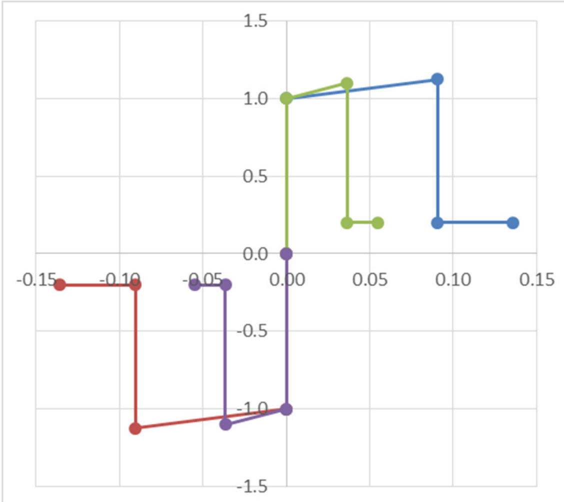
No. of Layers	Rectangular Column				Square Column		
	Ultimate Curvature		Plastic Rotation capacity		No. of Layers	Ultimate Curvature	Plastic rotation capacity
	Long Dir.	Short Dir.	Long Dir.	Short Dir.			
5 Layers	101.82%	82.47%	128.71%	94.63%	5 Layers	417.31%	551.99%
3 Layers	58.84%	50.09%	69.42%	60.09%	4 Layers	350.30%	475.43%
1 Layer	18.91%	16.73%	26.83%	24.51%	2 Layers	174.23%	232.98%

5.5.3 Hinge parameters

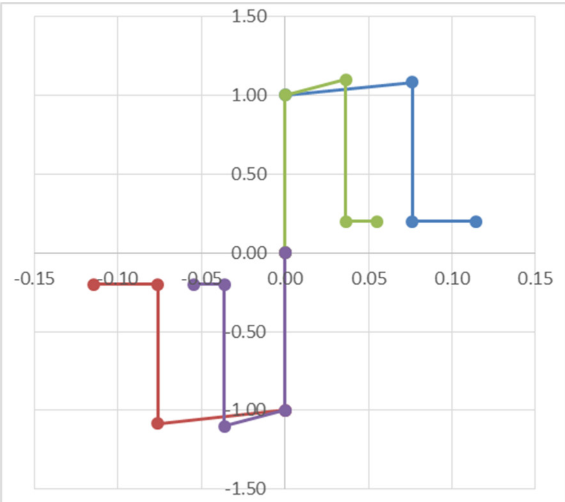
The nonlinear hinge parameters for the members has been generated as stated in section 4.5 of this dissertation report. The hinge parameter of the members represents the post yielding behaviour of the member. The enhancement in the hinge backbone curves of the members for various CFRP wrappings with no axial load are shown in the figure below:



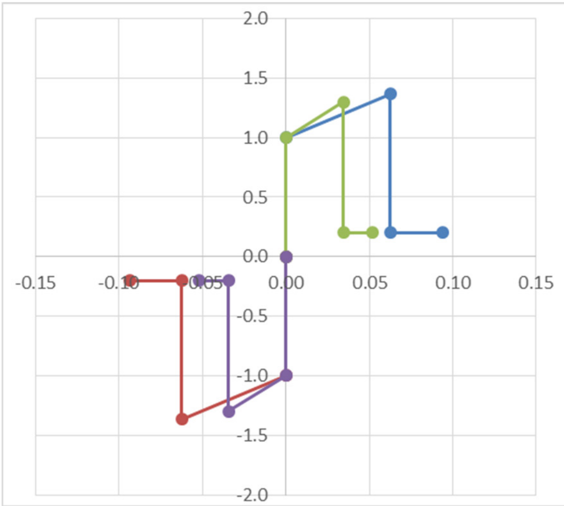
(a) Hinge backbone curve for existing and 3 layer wrapped exterior rectangular column.



(b) Hinge backbone curve for existing and 4 layer wrapped interior square column.



(c) Hinge backbone curve for existing and 2 layer wrapped interior square column.



(d) Hinge backbone curve for existing and 2 layer wrapped exterior rectangular column.

Fig. 5.15: *Nonlinear Hinge parameters for the Existing and Retrofitted columns.*

CHAPTER-6

CONCLUSIONS

In this dissertation, an existing building has been analyzed and suitable retrofitting techniques proposed to strengthen the same. Initially the strengthening was tried by means of local retrofitting only which was found to be inadequate for the structure. At the later stage, a combination of local and global retrofitting was investigated for the building which was found to be adequately strengthen the structure for the imposed seismic demand of the system. Nonlinear static analysis of the existing and retrofitted structure has been performed using manually developed Microsoft excel worksheets and feeding them into SAP2000.

As observed from the results obtained from the nonlinear analysis of the buildings, the FRP wrapping of columns only was found to be inefficient in enhancing the global behaviour of the structures due to the fact that the failure mechanism is predominantly controlled by the inelastic behaviour of the beams. However, when the beams along with the columns of the building were wrapped, the ductility of the structure was significantly increased without affecting the post yielding stiffness. From the nonlinear analysis performed of the retrofitted structure, it was found that the wrapping of beams and columns can greatly enhance the available ductility of the building. The enhancement in the ultimate displacement was found to be approximately 82% for building frame in the transverse direction whereas for frames in longitudinal direction, it was found to be 66% and 69% respectively for perimeter and interior frames in which both the beams and columns were wrapped with CFRP strips. In the frames where only columns were retrofitted by means of CFRP wrapping, so such significant enhancements were observed as the failure mechanism of the frames are primarily dominated by yielding of the beams.

When the combination of retrofitting technique was applied, it was seen that with the installation of additional steel bracings in the system, base shear capacity increases significantly but the ductility of the structure reduces considerably. For the building frame analyzed, the increase in the base shear capacity was recorded as 65% higher than the existing RC moment resisting frame. Wrapping of FRP sheets to the braced structure can provide a limited increase in the displacement capacity of the building. The increase in base shear capacity of the FRP wrapped braced building was found to be 75% higher than the existing RC frame. The increase in FRP wrapped braced frame was found to be 15% higher than the originally braced frame. The enhancement in base shear capacity of the frame due to wrapping of the columns was insignificant and found to be only 4%.

From the results of the analytical investigation done in this study, it can be concluded that, though the member ductility and strength parameters can be greatly enhanced from the FRP wrapping, their effect in the global performance of the structure is mostly insignificant; particularly in terms of enhancing the base shear carrying capacity. However, for installation of additional steel bracing system in the buildings can improve the overall performance of the structure by increasing its lateral load carrying capacity.

REFERENCES

1. Wan T. Tsai, Uniaxial Compressional Stress-Strain Relation of Concrete, *Journal of Structural Engineering*, Vol. 114, No. 9, September 1988.
2. Teng J.G., Chen J.F., Smith S.T. and Lam L., *FRP Strengthened RC Structures*, John Wiley & Sons, Ltd., 2002.
3. Nanni Antonio and Norris Michael S., FRP jacketed concrete under flexure and combined flexure-compression, *Construction & Building Materials*, Vol. 9,No. 5, pp 273-281, 1995.
4. Nanni Antonio and Bradford Nick M., FRP jacketed concrete under uniaxial compression, *Construction & Building Materials*, Vol. 9,No. 2, pp 115-124, 1995.
5. Mander J. B., Priestley M. J. N. and Park R., Theoretical Stress-Strain Model for Confined Concrete, *Journal of Structural Engineering*, Vol. 114, No. 8, August 1988.
6. Lam L. and Teng J. G., Design-oriented stress-strain model for FRP-confined concrete, *Construction & Building Materials* 17 (2003) 471-489.
7. H. Saadatmanesh, M. R. Ehsani, Limin Jin, Seismic Strengthening of Circular Bridge Pier Models with Fiber Composites, *ACI Structural Journal*, V. 93, No. 6, November-December 1996.
8. Computers and Structures. Inc. Berkeley, California, USA, *CSI Analysis Reference Manual for SAP2000, ETABS, SAFE and CSiBridge*, May 2015.
9. Bureau of Indian Standards, *Seismic Evaluation and Strengthening of Existing Reinforced Concrete Buildings – Guidelines*, IS 15988 (2013).
10. Bureau of Indian Standards, *General Construction in Steel — code of practice*, IS 800 (2007).
11. Bureau of Indian Standards, *Criteria for Earthquake Resistant Design of Structures, Part 1: General Provisions and Buildings*, IS 1893-1 (2002).
12. Bureau of Indian Standards, *Plain and Reinforced Concrete - Code of Practice*, IS 456 (2000).
13. Bureau of Indian Standards, *Dimensions for Hot Rolled Steel Beam, Column, Channel and Angle Sections*, IS 808 (1989).
14. Benzaid Riad and Mesbah Habib-Abdelhak (2013), Circular and Square Concrete Columns Externally Confined by CFRP Composite: *Experimental* Investigation and Effective Strength Models, *Fiber Reinforced Polymers - The Technology Applied for Concrete Repair*, Dr. Martin Masuelli (Ed.), ISBN: 978-953-51-0938-9, InTech, DOI: 10.5772/51589.
15. Bank Lawrence C., *Composites for Construction: Structural Design with FRP Materials*, John Wiley & Sons, Ltd., 2006.
16. A.Eslami and H. R. Ronagh, Effect of FRP wrapping in seismic performance of RC buildings with and without special detailing-A case study, *Composites: Part B*, Vol. 45, Issue 1, pp 1265-1274,2013.
17. American Concrete Institute, *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures*, ACI 440.2R-08 (2008).
18. Paulay, T., and Priestley, M. J. N., *Seismic Design of Reinforced Concrete and Masonry Buildings*, John Wiley and Sons, New York, 1992, 767 pp.