# Enhancing Seismic Resilience of Non-Engineered and Pre-Engineered RC Buildings: A Comparative Study of Retrofitting Techniques

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## Abstract

With the remarkable surge in reinforced concrete (RC) buildings in Nepal since their inception around the 1970s, the design of these structures has also evolved. Although the current building code (NBC 105:2020) incorporates the latest research in seismic design and is tailored to meet Nepal's unique requirements, a significant number of structures were erected prior to its enforcement, resulting in inherent structural vulnerabilities. Additionally, the quality of workmanship and construction practices have significantly impacted building performance, even when building code guidelines were followed. This paper assesses the vulnerability of non-engineered and pre-engineered frame typologies and applies various intervention measures to ensure compliance with the seismic performance requirements outlined in the current building code. The non-engineered and pre-engineered designs are modeled as representative 2D frames in OpenSees, followed by the application of suitable retrofit interventions. The retrofitting methods investigated include RC jacketing, steel jacketing, carbon fiber-reinforced polymer (CFRP), vegetal fiber-reinforced cementitious matrix (FRCM), and glass FRCM. The efficacy of each solution is assessed using non-linear static (pushover) analysis. The analysis reveals that after retrofitting, both non-engineered and pre-engineered frames have nearly the capacity of their engineered counterparts. All the aforementioned intervention measures are performed to achieve code compliance, and their performance is evaluated through pushover curves.

**Keywords**: Non-engineered, Pre-engineered, RC Buildings, Nepal Building Code (NBC), Seismic Retrofitting, OpenSees, Pushover Analysis, Structural Vulnerability

# 1 Introduction

Nepal, located in the seismically active Himalayan region where the Indian tectonic plate subducts under the Tibetan plate, has experienced numerous devastating earthquakes over the past centuries. Major seismic events have occurred in 1803, 1810, 1833, 1897, 1905, 1934, 1950, 1988, 2011, and 2015 [1]. The region's geological features drive this frequent seismic activity, posing several risks. The seismic history indicates that large earthquakes occur every 60-70 years in Nepal. The region's high seismic hazard zones exhibit a 2% probability of exceeding a Peak Ground Acceleration (PGA) of 0.38-1g and a 10% probability of exceeding a PGA of 0.21-0.62g over 50 years[2]. Globally, Nepal is ranked as the 11th most vulnerable country, with Kathmandu listed as the 21st most vulnerable megacity by the Bureau of Crisis Prevention and Recovery of the United Nations Development Programs [3]. Studies by the National Society for Earthquake Technology, Nepal [4], and the Japan International Cooperation Agency [5] predicted that a magnitude 8 earthquake occurring 100 km east of Kathmandu Valley could damage 50% of buildings and result in the loss of 1.3% of the population. This highlights the urgent need for comprehensive strategies to improve the seismic safety of Nepal's built environment. In the Kathmandu Valley, seismic vulnerability is exacerbated by factors such as outdated building codes, poor structural integrity, a dense population, and the prevalence of large buildings [6]. Addressing these deficiencies through effective retrofitting measures is essential to mitigating seismic risks and enhancing resilience.

In Nepal, the construction of reinforced concrete (RC) buildings began over four decades ago, following the 1988 earthquake in the eastern part of the country, which aimed to create stable and durable structures [7]. It is estimated that over 90% of these buildings are non-engineered RC frames, constructed by owners without professional guidance. [8] found that in the last two decades, about 50% of buildings in Nepal were constructed with RC, while others used materials like adobe and brick/stone with mud. The study indicated an exponential increase in RC buildings, highlighting a shift towards RC as the preferred construction method due to the structural weaknesses of older techniques. The 2011 census data shows that 10% of buildings in Nepal were made of RC, with 40% of these located in the Kathmandu Valley. The 2021 census indicates that this percentage has risen to 22.4%, with 30% in the valley. It was observed that these stocks will increase by 30% by 2030, according to a study by [3].

Among these buildings, non-engineered structures were constructed before codal provisions were introduced, designed solely to carry vertical loads without accounting for lateral seismic forces. These buildings were built to meet clients' specific needs, with contractors relying on their prior experience and sometimes guidance from mid-level technicians and masons, often without consulting design professionals. All buildings constructed without following the recommendations of the Mandatory Rules of Thumb (MRT) or any design code are considered non-engineered buildings [9]. The significant damage to Nepal's buildings from the 1988 Udayapur earthquake led to the introduction of the Nepal Building Code (NBC) in 1994 ([10]). NBC 205:1994, also known as MRT, provided seismic design guidelines under engineers' supervision. NBC 205:2012 was later introduced to address these issues by improving beam-column sizes and reinforcement. Since NBC 205:1994 lacked provisions for seismic design and ductile detailing, NBC 105:1994 was implemented to cover seismic design aspects. Buildings constructed under these codes are considered pre-engineered, ensuring minimum seismic safety. After the Gorkha earthquake, it became evident that these several buildings constructed according to the MRT guidelines were vulnerable and inadequate in resisting seismic loads. To address these deficiencies, NBC 105:2020 was introduced, aiming to rectify the seismic design of NBC 105:1994. Well-designed buildings now adhere to these updated codes, which apply to all types of structures, from low-rise to high-rise buildings.

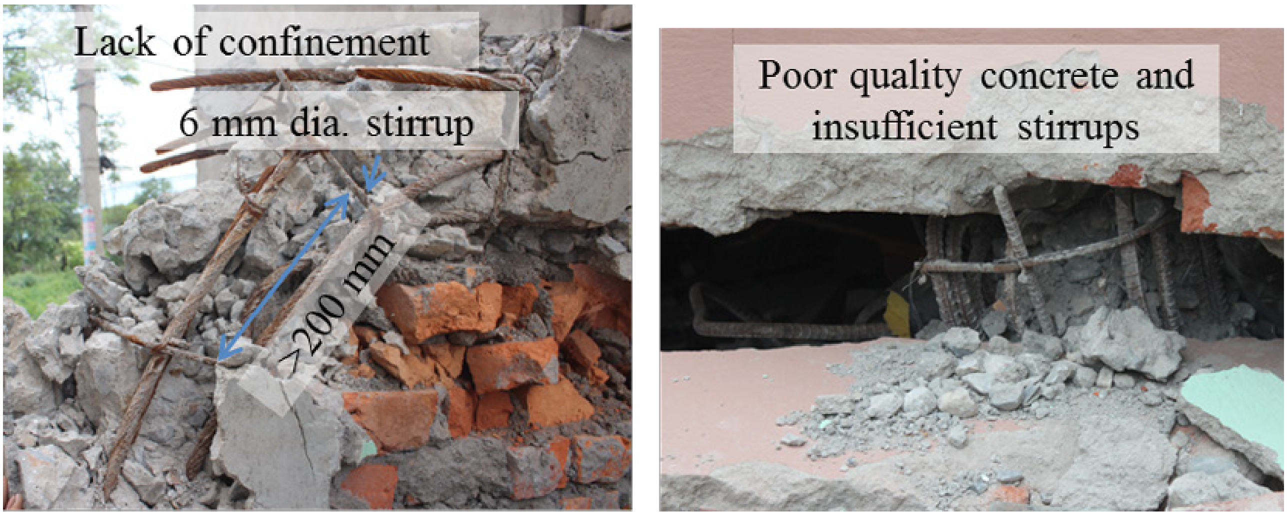



Figure 1: Structural Deficiencies during Gorkha earthquake[11]

The aftermath of the Gorkha earthquake highlighted significant vulnerabilities in Nepal's construction, particularly in RC buildings. Approximately 80% of these structures experienced various degrees of damage, with about 70% falling into the categories of non-engineered and pre-engineered buildings [12]. The minimum codal provisions of NBC 105: 2020 are not met by MRT 1994, making these buildings especially susceptible to seismic damage. [8] found that only well-designed RC buildings exhibited satisfactory seismic performance, while both pre-engineered and non-engineered buildings performed poorly. Owner-built houses were completely destroyed, while MRT-based buildings suffered partial damage. Common structural deficiencies in Nepalese RC buildings include the absence of ties in beam-column joints, inadequate confinement and lap lengths, low concrete strength, improper tie anchoring, irregular load distribution, and soft-story and short-column effects as shown in fig.1. Failures at beam-column connections, mainly due to improper ductile detailing, are a significant cause of building fragility [13]. Non-engineered buildings in Nepal are highly vulnerable to seismic events, and even many engineered buildings fail to meet seismic standards due to various issues like design flaws, outdated codes, poor site investigation, and subpar construction practices. Despite adhering to codes, poor workmanship affects building performance. After the Gorkha earthquake, a misconception arose that only RC buildings are earthquake-resistant, prompting a surge in RC construction, especially in the Kathmandu Valley, which now hosts nearly half of Nepal's RC buildings.

The literature on current scenarios and intervention strategies for strengthening residential buildings, particularly in seismically active regions, reveals diverse approaches and significant findings. [14] focused on developing empirical fragility functions for buildings in Nepal, highlighting the vulnerability of existing structures. [15]

demonstrated that RC jacketing, analyzed through static pushover and time history analysis using Seismostruct with lumped plasticity, significantly enhances the capacity and stiffness of buildings with good energy dissipation. [16] employed OpenSees for non-linear modeling of various building types, comparing pre- and post-2015 conditions to evaluate structural improvements. [17] further confirmed the efficacy of RC jacketing through similar analytical methods, noting reduced inter-story drift and fundamental period in retrofitted models. [18] explored the use of CFRP and PBO FRCM in four-point loading experiments, achieving substantial increases in flexural capacity. [9] emphasized the cost-effectiveness of RC jacketing and performance of retrofitting using dynamic time history analysis and distributed plasticity. [19] found that steel jacketing improved the overall behavior of strengthened columns, indicating enhanced structural performance. [20] investigated the seismic performance of MRT and NBC 105 designed beam-column joints using LS-DYNA for non-linear modeling, recommending optimal seismic strengthening strategies. [21] examined the stress-strain response and ultimate load capacity of concrete confined with FRP overlay, proposing a novel FRP confinement model. [22] demonstrates the improvement of base shear and pushover analysis up to 30% in non-engineered buildings with the application of vegetal FRCM, significant increase ductility. Collectively, these studies underline the critical role of various jacketing techniques and materials, such as CFRP, RC, and steel, in enhancing the seismic resilience of existing structures through both experimental and analytical methodologies.

# 2 Methodology

## 2.1 Modeling of frame

The study focuses on modeling a three-story, two-bay portal frame in OpenSees, consisting of beams and columns with fixed bases, without considering soil-structure interaction. The frame is modeled as a bare frame, excluding the effects of infill walls. Beams and columns are modeled as line element using force-based non-linear beamColumn element available in OpenSees. Each element has a pre-defined five integration points (IP), a standard number found in literature such as [23], as shown in Fig.2. These IPs help to discretize the element and determine displacements, with their number and location influencing the model's accuracy and computational efficiency. This approach of assigning non-linearity within various positions of elements, rather than just concentrating them at the ends, is known as distributed plasticity. This method divides the cross-section of each structural member into numerous uniaxial fibers of concrete and rebar, with each fiber assigned constitutive relations to capture material non-linearity. In the model, concrete fibers are discretized into 8x8 patches for the core concrete and 8x2 patches for the cover concrete, while rebar fibers are represented as point fibers, as shown in Fig.3. This configuration is intended to simulate the non-linear behavior of the frame accurately, with the number of patches of fiber and integration points balancing accuracy and computational efficiency.

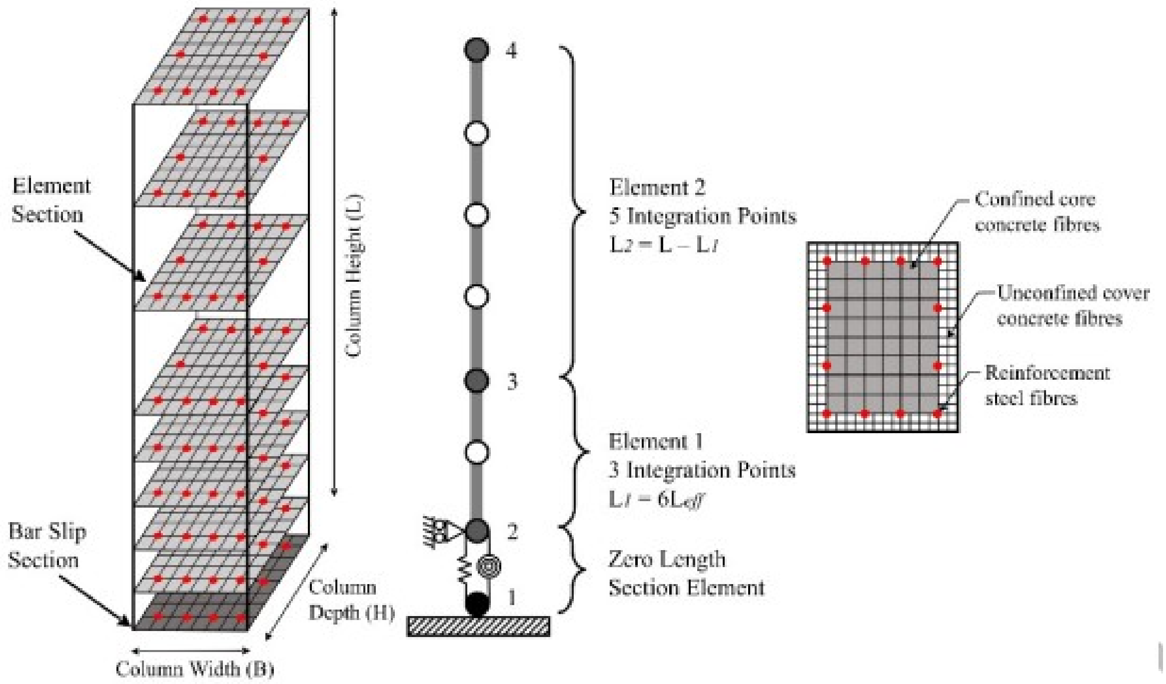



Figure 2: FEA with Integration points[23]

#### 2.1.1 Geometry

A 2D frame with an unsymmetrical bay of 3.2 m and 3.9 m along the x-direction and 3 stories with each story height of 3 m, as shown in Fig.4. The numbering with a green numeric denotes a node, and the red one is an element: horizontal as a beam and vertical as a column.

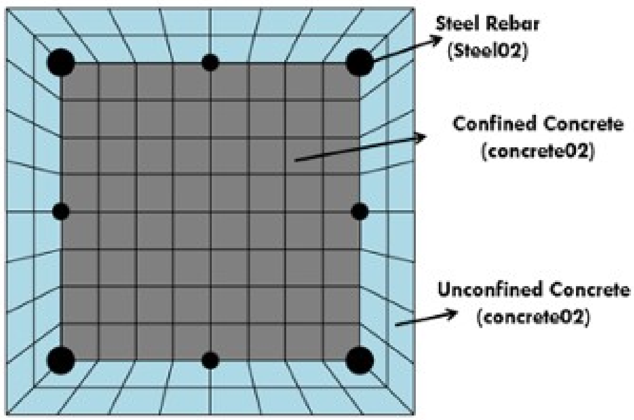



Figure 3: Column section with concrete and rebar fiber in OpenSees

#### 2.1.2 Material Modeling

In OpenSees [24], the 'Concrete02' and 'Steel02' material models were utilized to represent the non-linear behavior of concrete and steel reinforcement, respectively. The 'Concrete02' model is a uniaxial Kent-Scott-Park model with linear tension softening, appropriate for both confined and unconfined concrete scenarios [25]. Figure

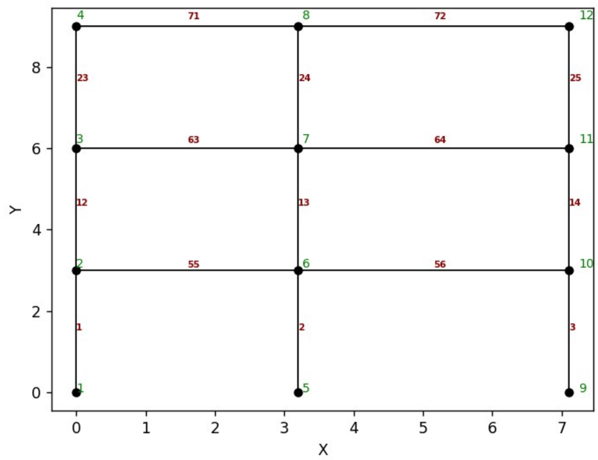
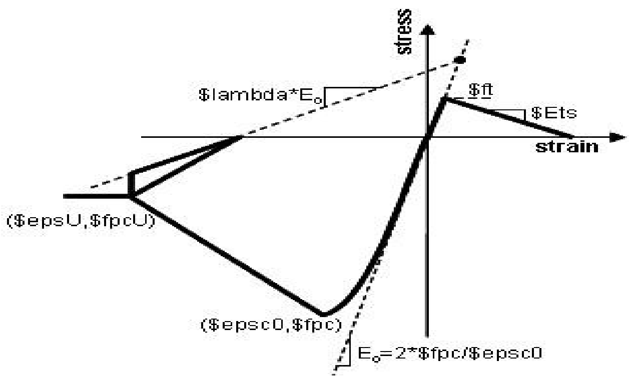
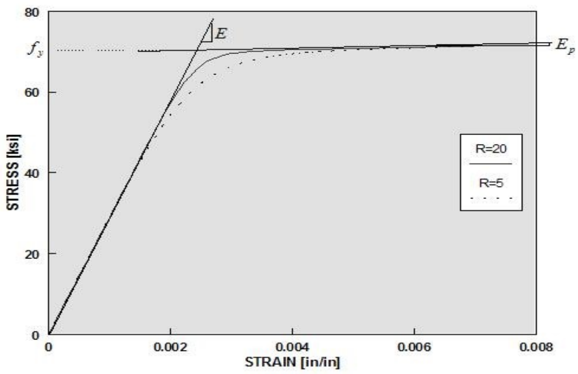


Figure 4: 2D model with three story

5a illustrates the stress-strain curve for the 'Concrete02' material model available in the OpenSees library. The confined core concrete employs Mander's Confinement model [26], which enhances properties through effective lateral confining stress. The confinement action is influenced by the configuration of transverse and longitudinal reinforcement. In contrast, the cover concrete is modeled without considering confinement . However, Mander's Confinement Model is typically used in engineered buildings, where lateral confinement of concrete is more effective due to appropriate transverse stirrups. Literature indicates that non-engineered and pre-engineered buildings exhibit less confinement action due to improperly spaced transverse stirrups. The 'Steel02' material model is employed to simulate steel reinforcement, based on the Giuffré-Menegotto-Pinto (GMP) model with isotropic strain hardening, effectively capturing the behavior of steel under various loading conditions [27]. The stress-strain curve of 'Steel02' is depicted in Figure 5b. Figure 3 shows the modeling of an RC element (column) using the 'Concrete02' and 'Steel02' material models.




(a) Concrete02 [25]




(b) Steel02[27]

Figure 5: Adopted Material Models

## 2.2 Model Validation

To utilize OpenSees for non-linear static analysis, rigorous verification must first be performed to establish baseline behavior and verify material models and their parameters. The results of the OpenSees model are then compared with the actual test of cyclic pushover performed by [28]. Details of the models are described in the table 6.

Table 1: Specimen Information and Properties

| Specimen Information | | Material Properties | |
|---|---|---|---|
| **Name:** | Saatcioglu and Ozcebe 1989, U4 | **Concrete Strength:** | 32 MPa |
| **Type:** | Rectangular | **Transverse Steel Yield Stress:** | 470 MPa |
| **Reference:** | Saatcioglu et al (1989) | **Longitudinal Steel Corner Yield Stress:** | 438 MPa |
| | | **Longitudinal Steel Intermediate Yield Stress:** | 438 MPa |
| **Geometry** | | **Loading** | |
| **X-Section Width:** | 350 mm | **Axial Load:** | 600 kN |
| **X-Section Depth:** | 350 mm | **P-D:** | Shear provided |
| **Length L-Inflection:** | 1,000 mm | **L-Top:** | 0 |
| **Length L-Measured:** | 1,000 mm | **L-Bottom:** | 0 |
| **Test Configuration:** | Cantilever | | |
| **Longitudinal Reinforcement** | | **Transverse Reinforcement** | |
| **Diameter:** | 25 mm corner bars, 25 mm intermediate bars | **Type:** | R: Rectangular ties (around perimeter) |
| **Number of Bars:** | 8 | **Number of Shear Legs:** | 2 |
| **Perpendicular to Load Clear Cover:** | 22.5 mm | **Region of Close Spacing Bar Diameter:** | 10 mm |
| **Number of Intermediate Bars:** | 1 | **Hoop Spacing:** | 50 mm |
| **Parallel to Load Clear Cover:** | 22.5 mm | **Region of Wide Spacing Bar Diameter:** | 10 mm |
| **Number of Intermediate Bars:** | 1 | **Reinforcement Ratio:** | 0.025 |
| **Reinforcement Ratio:** | 0.0321 | | |
| **Non-Dimensional Properties** | | | |
| **Span-to-Depth Ratio:** | 2.86 | **Axial Load Ratio:** | 0.153 |

In the process of using OpenSees, concrete and steel were chosen as the primary materials for the structural elements. The 'Concrete02' uniaxial material model was utilized to represent concrete, with input parameters including compressive strength, strain at peak stress, and ultimate strain. Longitudinal rebars were modeled using the 'Steel02' uniaxial material model, specifying parameters such as yield stress and elastic modulus. The Lobatto integration scheme, with five integration points (IPs), was used along the length of the column. Additionally, the validation process involved conducting a cyclic pushover analysis to assess the structure's response under this loading. Incremental displacement control, based on experimental data, was applied to the structure's end node during this analysis. A plot of the force-displacement response of test data and numerical simulation is presented in Figure 6.

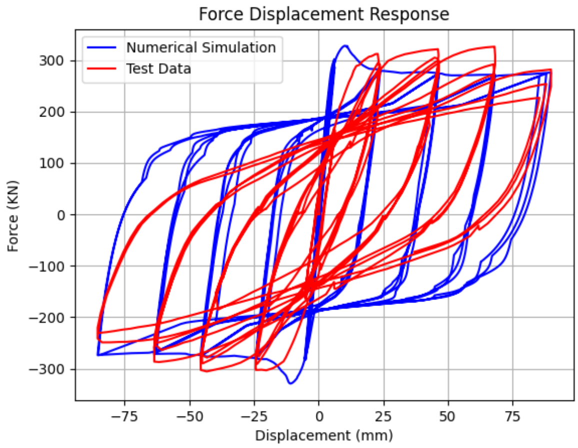



Figure 6: Validation of RC pier in OpenSees

## 2.3 Frame Typologies

Comprehensive data on column and beam sizes, as well as detailing information for buildings, were collected from various sources, including existing literature, contractors, field surveys, and documentation from the Lalitpur Metropolitan City (LMC). The Data gathered in this study includes structural detailing of RC beams and columns, building ages, and concrete strength. Frame models for each building typology (Non-Engineered, Pre-Engineered, and Engineered) were developed in OpenSees using representative parameters. The non-engineered models were based on literature and observations, while the Pre-engineered models followed MRT guidelines[29]. The Engineered models were designed according to NBC105:2020[30]. Considering Pre-Engineered and Engineered have enough privilege of ductile detailing, which gives confinement in the concrete core as per Mander's Concrete model[26]. The detailed mechanical properties given in table 2 for modeling in OpenSees and structural detailing of frame elements is given in table 3, table 4 and table 5.

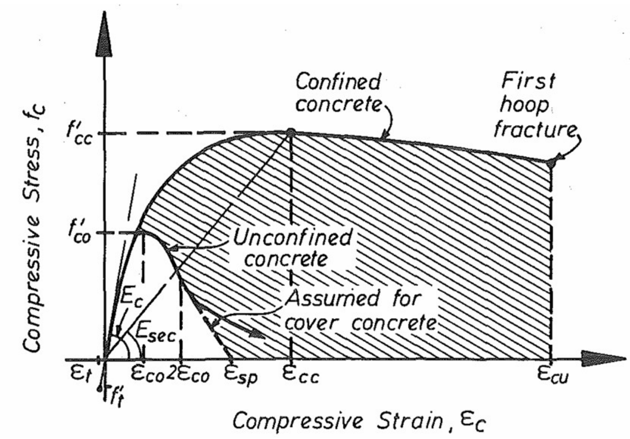



Figure 7: Stress-strain diagram for confined and unconfined concrete [26]

## 2.4 Non-Linear Analysis

Buildings may display inelastic behavior, such as steel yielding and cracking in concrete or masonry walls, during intense ground shaking. Consequently, for the purpose of precisely capturing this inelastic behavior, non-linear

Table 2: Mechanical Properties of Materials

| **Properties** | **Non-Eng.** | **Pre-Eng.** | **Eng.** | **Units** |
|---|---|---|---|---|
| Unconfined Concrete | | | | |
| Compressive strength (fpc) | 15 | 15 | 20 | MPa |
| Concrete Crushing Strength (fpu) | 3 | 3 | 4 | MPa |
| Strain at Maximum Strength (epscO) | 0.002 | 0.002 | 0.002 | |
| Strain at crushing strength (epscU) | 0.02 | 0.02 | 0.02 | |
| Elastic modulus in tension (Et) | 750 | 750 | 1000 | MPa |
| maximum stress in tension (ft) | 1.5 | 1.5 | 2 | MPa |
| Confined concrete | | | | |
| Compressive strength (fpc) | 15 | 21 | 28 | MPa |
| Concrete Crushing Strength (fpu) | 3 | 4.2 | 5.6 | MPa |
| Strain at Maximum Strength (epscO) | 0.002 | 0.002 | 0.002 | |
| Strain at crushing strength (epscU) | 0.02 | 0.02 | 0.02 | |
| Elastic modulus in tension (Et) | 750 | 1050 | 1400 | MPa |
| maximum stress in tension (ft) | 1.5 | 2.1 | 2.8 | MPa |
| Steel | | | | |
| Reinforcing steel Yield Strength (fy) | 415 | 415 | 500 | MPa |
| Modulus of Elasticity of steel (Es) | 210 | 210 | 210 | GPa |
| Strain hardening ratio (b) | 0.05 | 0.05 | 0.05 | |

Table 3: Column Detailing for Engineered and Non-engineered frame

| Model Typology | Column size mm x mm | Longitudinal Rebar | | |
|---|---|---|---|---|
| | | 1st Story | 2nd Story | 3rd Story |
| Non- Engineered | 230 x 230 | 4 – 16 mm $\phi$ | 4 – 12 mm $\phi$ | 4 – 12 mm $\phi$ |
| Engineered | 300 x 300 | 8 – 16 mm $\phi$ | 4 – 16 mm $\phi$ +4 – 12mm $\phi$ | 8 – 12 mm $\phi$ |

analysis approaches are favored [31]. Both material and geometric non-linearity are taken into account in the model employed in this investigation. When the stress-strain relationship is non-linear, material non-linearity occurs and is treated with a distributed plasticity strategy. Taking the stance that non-linearity can exist across the board, all factors are treated as nonlinear. A nonlinear beam-column element, an OpenSees implementation of a force-based element, is used to describe beams and columns. Nonlinearity is concentrated at five integration points per element [23]. Geometric nonlinearity addresses the deformations caused by significant displacements, rotations, and strains, leading to a non-linear relationship between these parameters. This type of nonlinearity is incorporated at the constitutive level using higher-order strain definitions or at the building level through joint transformations (P-$\Delta$ effects).

Table 4: Column Detailing of Pre-engineered frame [29]

| **Story** | **Corner Column** | **Face Column** | **Interior Column** |
|---|---|---|---|
| 3rd Story | 230 x 230<br>4 – 16 mm $\phi$ | 230 x 230<br>4 – 12 mm $\phi$ | 230 x 230<br>4 – 12 mm $\phi$ |
| 2nd Story | 230 x 230<br>4 – 16 mm $\phi$ | 230 x 230<br>4 – 12 mm $\phi$ | 230 x 230<br>8 – 12 mm $\phi$ |
| 1st Story | 270 x 270<br>4 – 16 mm $\phi$ | 270 x 270<br>4 – 16 mm $\phi$ | 270 x 270<br>8 – 12 mm $\phi$ |

Table 5: Structural detailing of Beam

| Typology | Beam size | Longitudinal rebar of all story |
|---|---|---|
| Non- Engineered | 230 x 325 | Top = 2 − 16 mm $\phi$ |
| | | Bot = 2 − 12 mm $\phi$ |
| Pre- Engineered | 230 x 325 | Top = 2 − 16 mm $\phi$ |
| | | Bot = 2 − 12 mm $\phi$ |
| Engineered | 230 x 350 | Top = 3 − 12 mm $\phi$ |
| | | Bot = 3 − 12 mm $\phi$ |

#### 2.4.1 Eigen Value Analysis

In the analysis of multi-storey frame structures, it is common to assume that structural masses are concentrated at each storey level, a method known as the lumped mass approach. This approach simplifies modeling by assigning masses and loads at each node, resulting in lumped mass matrices with non-zero values primarily along the leading diagonal. This makes the lumped mass method computationally efficient, requiring less storage space and processing time([32]).

Eigenvalue analysis was conducted for three typologies: non-engineered, pre-engineered, and engineered, to determine their fundamental time periods, as shown in table 6. The results indicate that non-engineered typology exhibits the highest time period, engineered structures the lowest, with pre-engineered structures falling in between. This trend suggests an increase in stiffness from non-engineered to pre-engineered to engineered structures, which is attributed to superior ductile detailing and material properties in engineered structures. Furthermore, the analysis highlights that the engineered frame, designed in accordance with NBC 105:2020, provides sufficient stiffness. In contrast, non-engineered and pre-engineered structures require strengthening to achieve comparable stiffness to that of engineered structures.

Table 6: Time Periods of 2D Frame

| **Time Period** | **Non-Engineered** | **Pre-Engineered** | **Engineered** |
|---|---|---|---|
| Fundamenetal Time Period | 0.811 s | 0.656 s | 0.559 s |

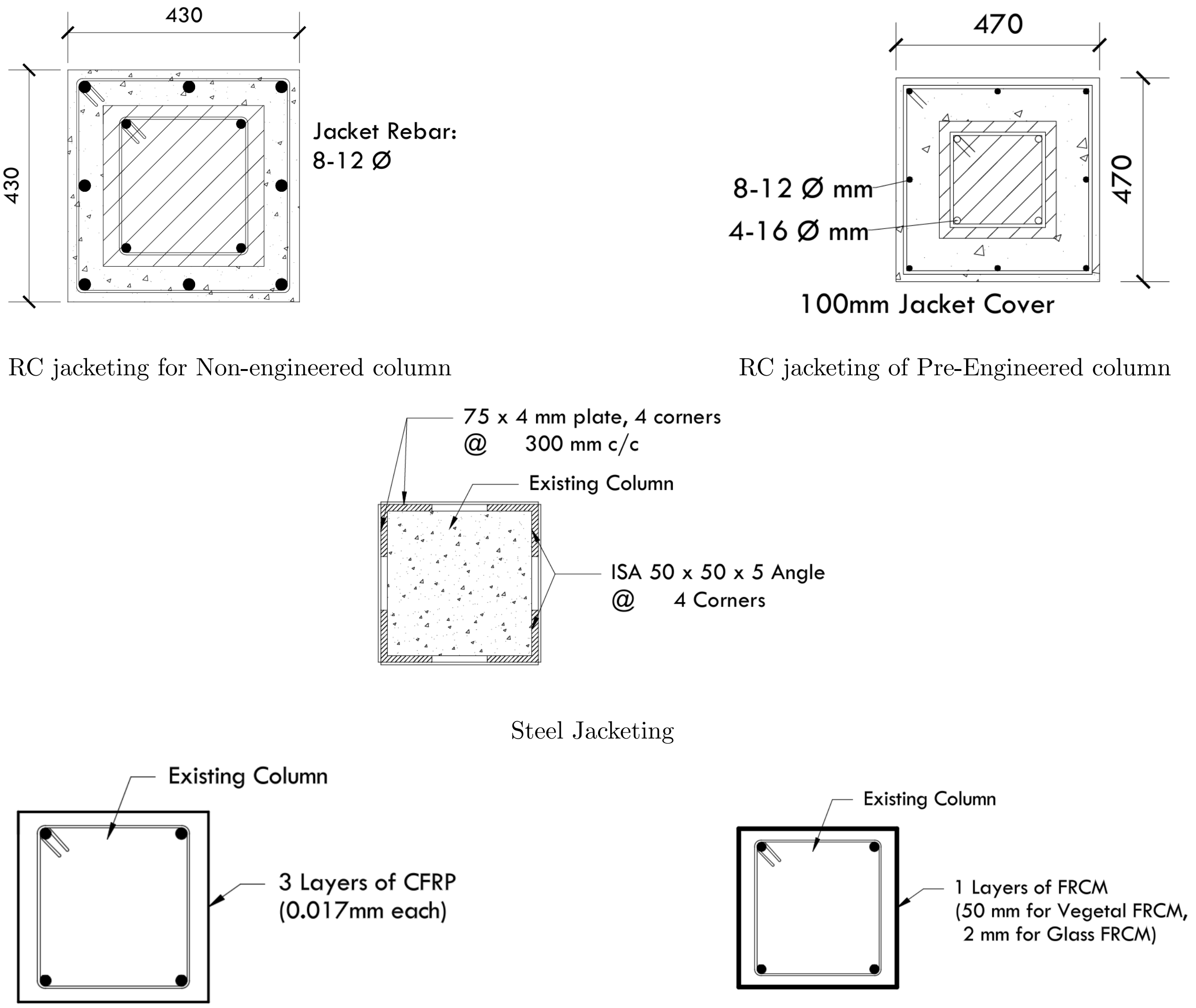
#### 2.4.2 Non-Linear Static Anlaysis

Nonlinear static analysis (pushover analysis) is a technique used to estimate the seismic performance of structures by evaluating their ultimate load and deflection capabilities. Pushover analysis helps identify weak points in a structure under lateral forces, such as those from earthquakes, and is most accurate for buildings with regular configurations that do not exhibit higher mode effects. There are two approaches to pushover analysis: force-controlled and displacement-controlled[33]. The force-controlled approach is used when the load is known and the structure can sustain it, typically for gravity loads. The displacement-controlled approach is used when drift limits are known from expected seismic activity. Based on displacement-controlled approach, structures is subjected to pushover load in a weaker direction, where load pattern is based on first mode as recommended by NBC 105:2020 [30] and monitoring the displacement of a control node, usually at the top level with target drift limit of 4% of total height of model. The relationship between the base shear and the control node displacement is plotted as a pushover curve.

## 2.5 Retrofitting Strategies

There are two main approaches to enhance the seismic capacity of existing structures: global level intervention and local level intervention. Global interventions involve modifying the entire structural system, while local interventions focus on enhancing the ductility of specific vulnerable components to meet limit states. This study concentrates solely on local level interventions with five retrofitting options: RC jacketing, steel jacketing, CFRP (Carbon Fiber Reinforced Polymer) jacketing, Glass FRCM (Fabric Reinforced Cementitious Matrix) jacketing, and vegetal hemp FRCM. These methods were applied to both non-engineered and pre-engineered frames to meet the target engineered frame. Reinforced concrete jackets were added based on the calculation of deficient concrete and rebar as per IS code 15988. Despite the required cover being less than 100mm, a 100mm jacket cover was provided to meet the minimum requirement specified by the code[34]. The confinement in the concrete was achieved using the Mander model [26]. For steel jacketing,Steel angle sections were added to all four corners of a column with supporting plates in between. The sections were determined using the document "Seismic Retrofitting Guidelines of Buildings in Nepal" through a trial and error analysis compared with engineered sections[35]. A minimum section of 50mm x 50mm x 5mm was sufficient to achieve the desired seismic performance.

FRP wrapping was applied with thickness 0.017mm of 3 layers and modeled based on the FRP confined concrete model and the stress-strain curve by Lam and Teng [36]. The confined stress and strain values, calculated from ACI guidelines [37], show concrete strength increases from 15 MPa to 21 MPa for non-engineered frames and from 21 MPa to 37 MPa for pre-engineered frames, with strain limited to 0.01. This method increased ductility, prevented brittle shear failure, enhanced shear strength, and prevented column buckling, thus minimizing damage in the compression zone. Fabric Reinforced Cementitious Matrix (FRCM) composites exhibit notable mechanical behavior under tensile stress, largely influenced by the properties of their constituents and the strength of their bonding. In this study, Glass FRCM of 2 mm thickness was employed as a retrofitting measure, wherein a tri-linear stress-strain curve was incorporated. The stress-strain values at each stage were derived from tensile test experiments by [38]. Vegetal FRCM studied by Mercedes [39] has demonstrated that among various vegetal FRCM composites, hemp exhibits the highest mechanical properties. Complementary research by [22] indicates that hemp-based retrofitting significantly enhances the structural strength of non-engineered building. In our study, vegetal hemp FRCM of 50mm thickness was applied by trial and error using a tri-linear stress-strain curve as defined by Mercedes[39]. The retrofitting strategies implemented in column of frame model is shown in Fig.8




RC jacketing for Non-engineered column

RC jacketing of Pre-Engineered column

Steel Jacketing

CFRP

FRCM

Figure 8: Proposed retrofittings in Column

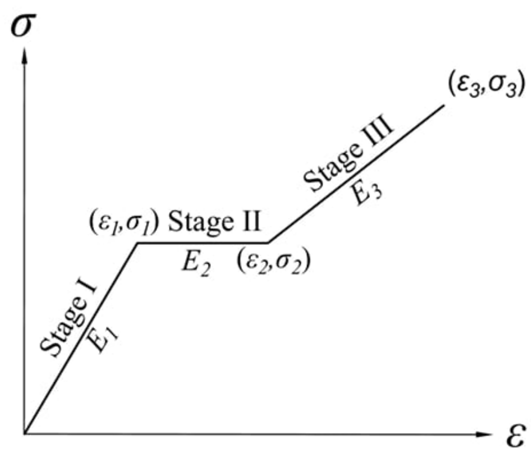



Figure 9: Tri-linear Stress-strain curve of FRCM [22]

# 3 Result and Discussion

In a 2D frame model, the performance of the non-engineered and pre-engineered typologies was compared to the performance of engineered (code compliant) typology. Static pushover analysis was performed to evaluate the global performance of model building. Various retrofitting schemes, including conventional methods, namely concrete and steel jacketing and innovative methods, particularly FRP jacketing, hemp FRCM jacketing and glass FRCM jacketing were introduced on the non-engineered and pre-engineered typologies to make them code compliant. It was observed that the applied retrofitting interventions increased the stiffness of the non-engineered and pre-engineered frames as the fundamental time period for the retrofitted frame decreased to nearly around time period of engineered(0.559) as shown in Table 7.

Table 7: Fundamental Time Period of Retrofitting Interventions in Seconds

| **Typologies** | **Retrofitting Technique** | | | | |
|---|---|---|---|---|---|
| | **RC Jacket** | **Steel Jacket** | **Glass FRCM** | **Hemp FRCM** | **CFRP** |
| Non-Engineered | 0.399s | 0.506s | 0.503s | 0.596s | 0.744s |
| Pre-Engineered | 0.373s | 0.449s | 0.431s | 0.504s | 0.497s |

Fig. 10 reveals the pushover curves of the building typologies before retrofitting. The maximum base shear coefficient of non-engineered, pre-engineered and engineered frames were found to be around 0.1, 0.15 and 0.25 respectively. This depicted that the strength capacity and initial tangential stiffness of engineered frame is relatively higher than those of the non-engineered and pre-engineered. The strength of pre engineered frame is 50% higher than that of non-engineered, but both are far less than capacity of engineered one. Engineered frame is almost 1.5 times and 2.5 times higher than non-engineered and pre-engineered frames, respectively. This shows the necessity of strengthening non-engineered and pre-engineered buildings to make them as resilient as engineered buildings. Considering the performance of engineered frame as a targeted frame, retrofitting was applied to achieve the performance of non-engineered and pre-engineered frame in the range of engineered. Fig.11- Fig.15 show the pushover curves after retrofitting interventions, clearly reflecting the increase in strength and stiffness of the buildings after these measures. The initial tangential stiffness of RC jacketing showed greater improvement compared to other methods, resulting in a more rigid RC jacketing frame. The maximum base shear was obtained in steel jacketing with significant increase of base shear by around 300%. Both non-engineered and pre-engineered frames exhibited a similar peak point path across all measures. Glass FRCM and Hemp FRCM have achieved slightly less than engineered ones with significant enhancement of ductility.Within these two FRCM approaches, Glass FRCM found to be more effective as per performance. All methods, except CFRP, achieved a peak base shear nearly equivalent to the engineered frame. This discrepancy is due to CFRP primarily enhancing the confinement of elements to prevent shear failure, without significantly improving flexural strength and stiffness. This FRP wrapping improves in other factors without significant change in strength or performance. A Similar result was obtained in the study observed by [40]. The maximum base shear coefficient after the application of retrofitting measures shows that base shear value is in the range of the targeted engineered frame and tabulated in table 8.

Table 8: Maximum Base Shear value

| Retrofitting scheme | Non-engineered | Pre-engineered |
|---|---|---|
| Original (kN) | 62 | 85 |
| RC jacketing (kN) | 172 | 180 |
| Steel jacketing (kN) | 190 | 225 |
| Hemp FRCM jacketing (kN) | 115 | 140 |
| Glass FRCM jacketing(kN) | 140 | 170 |
| FRP jacketing (kN) | 75 | 90 |

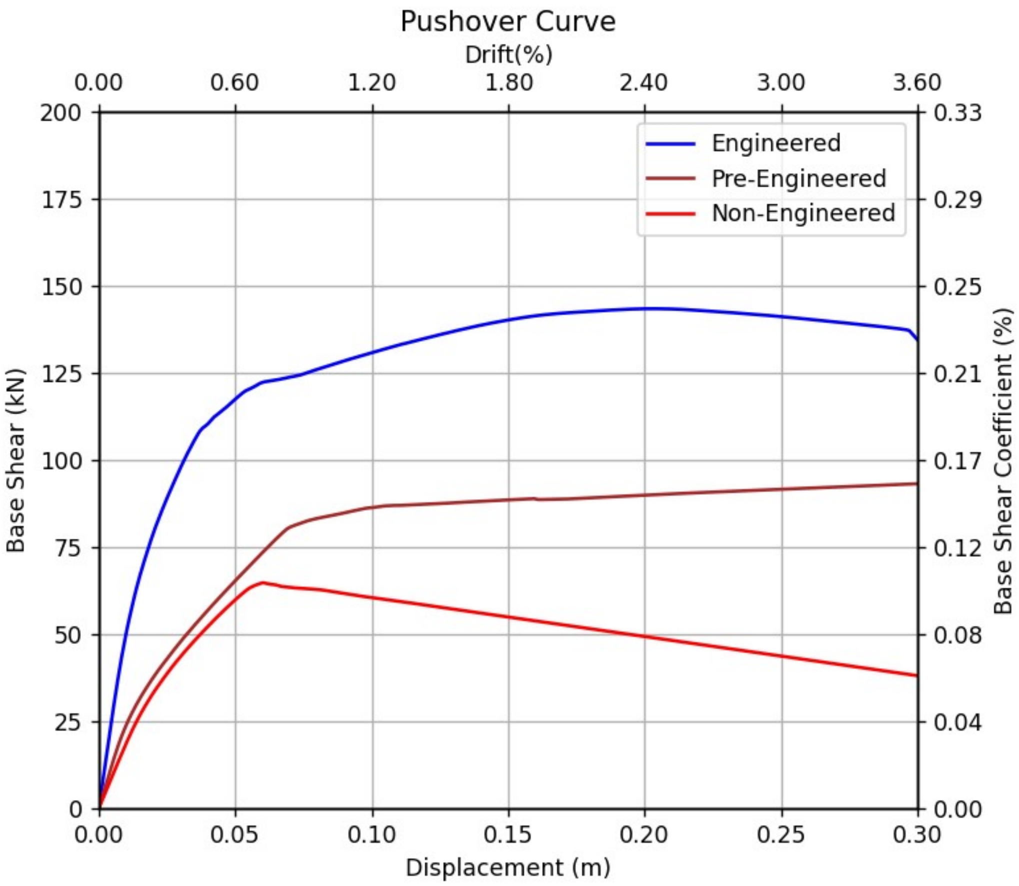
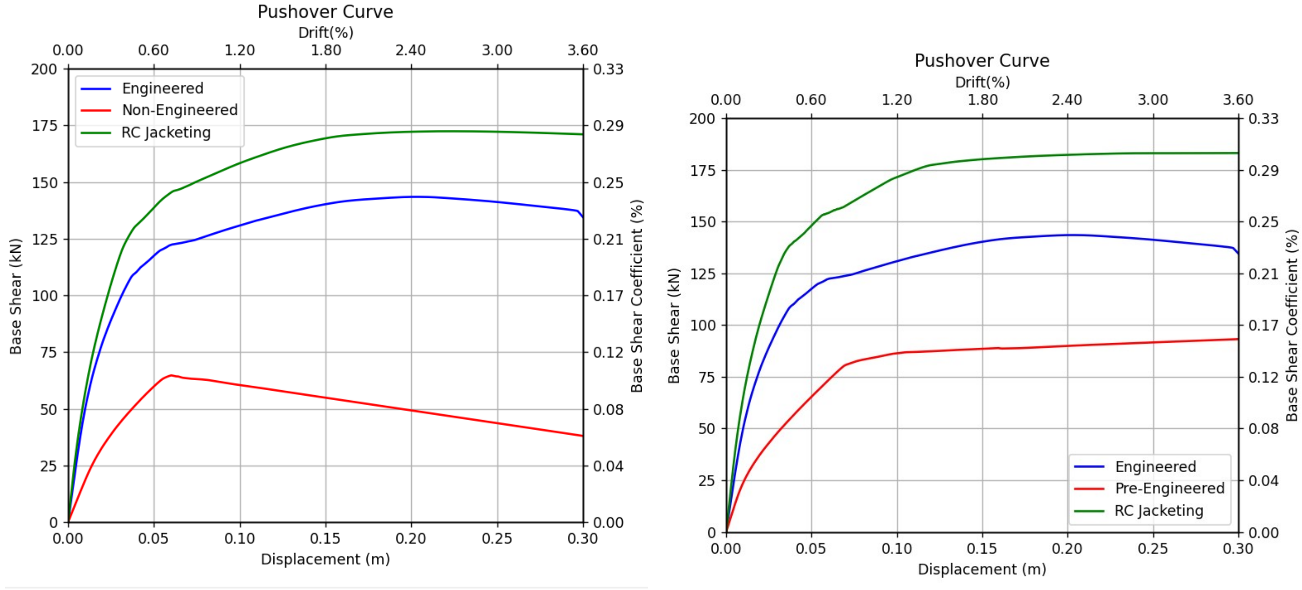



Figure 10: Pushover Curve of all typologies

# 4 Conclusion

Majority of the building stock in Nepal is not compliant with the recently revised building code, NBC:2020. This study mainly focused on the performance study of the frame model which mainly represented the existing building stock through non-linear static analysis. The major highlight of the study was the implementation of various retrofit interventions namely RC jacketing, steel jacketing, glass and hemp FRCM jacketing and CFRP jacketing to improve the seismic response of non-engineered buildings that were constructed using prevalent construction practices before the introduction of building codes and pre-engineered buildings constructed based on mandatory rules of thumb outlined in NBC. One of the bays of representative buildings was considered and analysis of 2D frame was done on OpenSees.

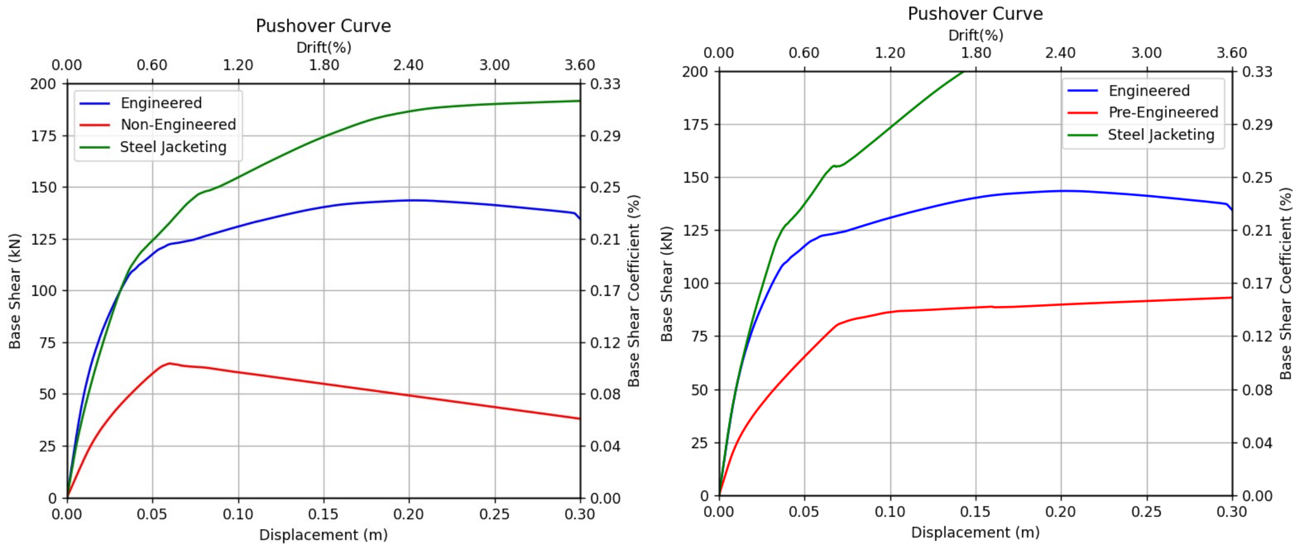
The use of 100mm thick RC jacket and 50mm×50mm×5mm steel angle sections on each of the four corners of the columns was found to be adequate for both non-engineered and pre-engineered buildings. This study




(a) Pushover of Non-Eng with RC jacketing

(b) Pushover of Pre-Eng with RC jacketing

Figure 11: RC jacketing



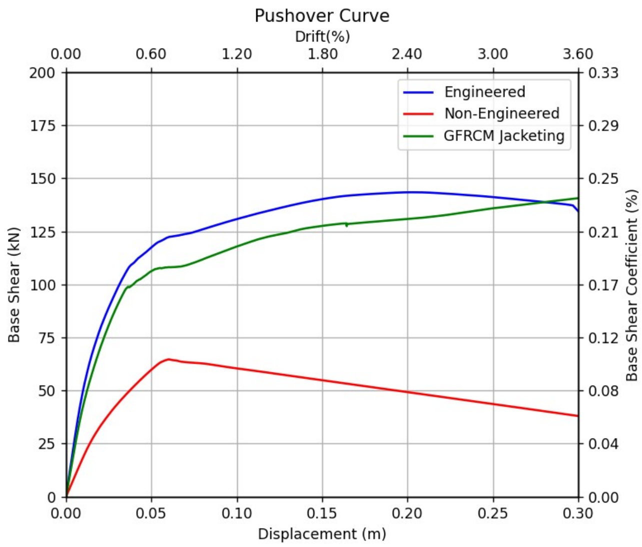

(a) Pushover of Non-Eng with Steel jacketing

(b) Pushover of Pre-Eng with Steel jacketing

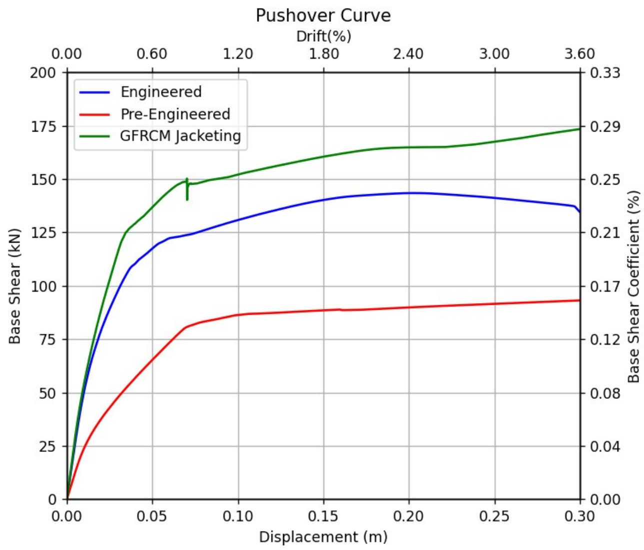
Figure 12: Steel jacketing

also includes the introduction of modern methods of interventions as FRP and FRCM retrofittings. The Glass FRCM was found more effective amonng them. 2mm thickness of glass FRCM on non-engineered and pre-engineered achieved the target performance whereas it required 50mm for vegetal hemp FRCM. The variation of these thickness is due to large value of modulus of rigidity of glass FRCM which is almost 10-100 times higher than Vegetal hemp FRCM. Meanwhile, FRP strengthning provide only confinement of the concrete section, prevent buckling and damage in compression zone but not succeed to achieve flexural strength and stiffness.

Other prevalent method of retrofitting such as base isolation, use of diagonal braces or shear walls was not examined in this study. Similarly, all of the analysis were done on 2D frame instead of more time consuming but accurate 3D model. Pushover analysis is applicable for low rise buildings considering load pattern based on first mode. The accuracy of pushover analysis might be affected when the target displacement is large and

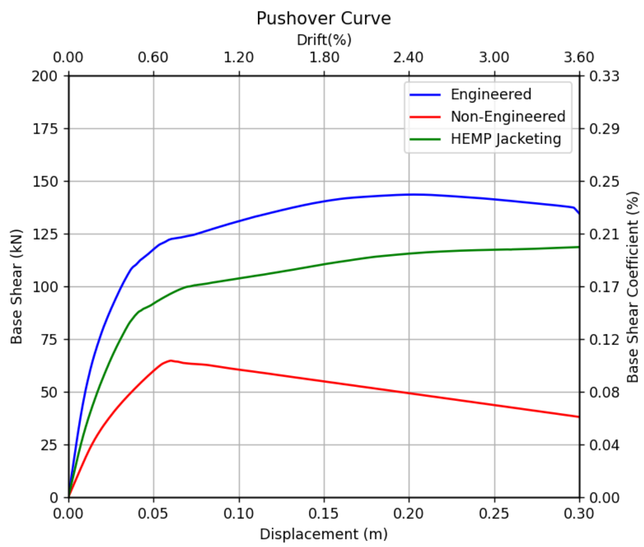



(a) Pushover of Non-Eng with Glass FRCM jacketing

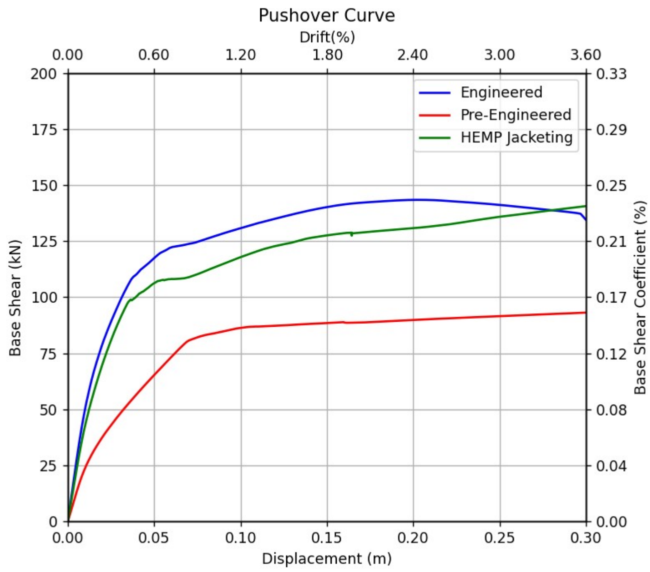



(b) Pushover of Pre-Eng with Glass FRCM jacketing

Figure 13: Glass FRCM jacketing



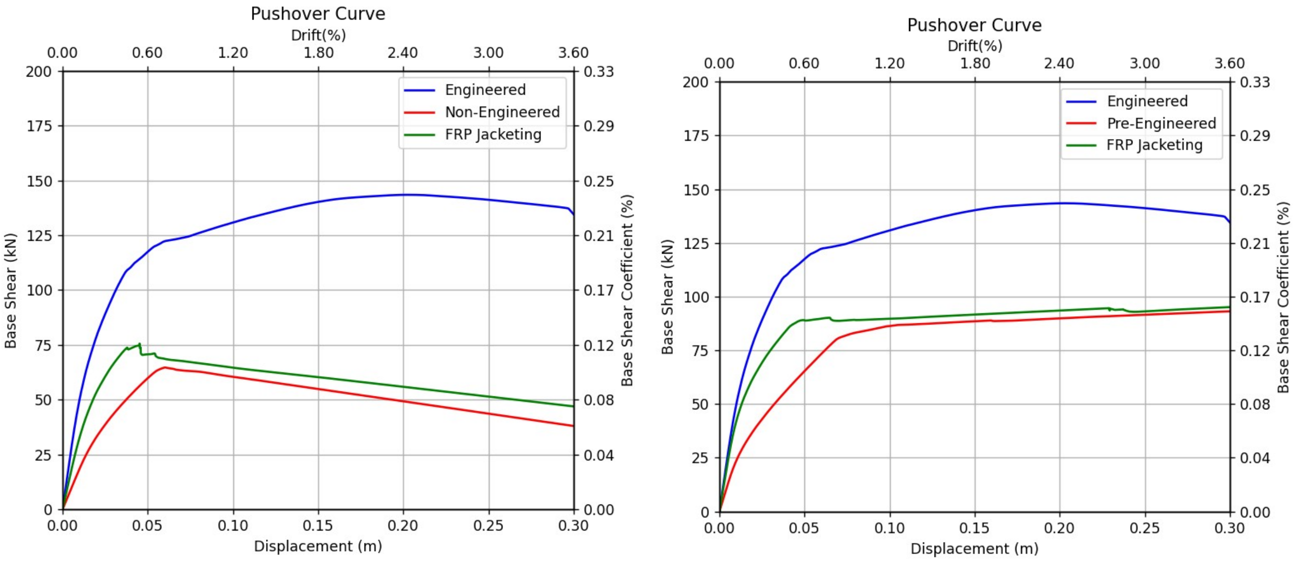

(a) Pushover of Non-Eng with Hemp FRCM jacketing




(b) Pushover of Pre-Eng with Hemp FRCM jacketing

Figure 14: Hemp FRCM jacketing

when selection of lateral load pattern and selection of failure mechanism due to higher modes are considered. In such case dynamic analysis are crucial for performance analysis. So, combined study of real 3D buildings with performance analysis by non-linear dynamic analysis and fragility analysis could be upcoming approaches for our study.




(a) Pushover of Non-Eng with CFRP jacketing (b) Pushover of Pre-Eng with CFRP jacketing

Figure 15: CFRP jacketing

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