

DAMAGE DETECTION OF MASONRY INFILLED RC FRAME STRUCTURE

A thesis

Submitted in partial fulfillment of the requirement

For the award of the degree of

DOCTOR OF PHILOSOPHY

IN

CIVIL ENGINEERING

Submitted by

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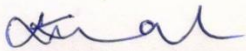
THAPAR INSTITUTE
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2020

CERTIFICATE

It is certified that the thesis report entitled “**Damage Detection of Masonry Infilled RC Frame Structure**”, which is being submitted herewith by **Balvir Singh**, in partial fulfillment for the award of degree in **Doctoral of Civil Engineering (Structures)** at **Thapar Institute of Engineering and Technology, Patiala** is an authentic record of student’s own work carried out under my supervision and guidance. The matter presented in the thesis has reached the standards fulfilling the requirements of the regulation for the award of said degree.



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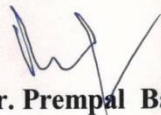
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I hereby certify that the work which is being presented in this thesis, entitled “**DAMAGE DETECTION OF MASONRY INFILLED RC FRAME STRUCTURE**”, in fulfillment of the requirements for the award of degree of ‘Doctor of Philosophy’ in Civil Engineering submitted to Civil Engineering Department of Thapar Institute of Engineering and Technology (Deemed to be University), Patiala is an authentic record of my own work carried out under the supervision of Dr. Naveen Kwatra and Dr. Shruti Sharma, and wherever other research work is referred it is indexed in the list of references.

The matter presented in this thesis has not been submitted in part or full to any other university or institute for the award of any degree in India or abroad.

(Balvir Singh)

ACKNOWLEDGEMENT

I bow my head with utmost gratitude before the Almighty ‘**WAHEGURU**’ whose eternal blessing has enabled me to accomplish this noble effort and achieve one more goal of my life.

A dissertation cannot be completed without the help of many peoples who contribute directly or indirectly through there constructive criticism in the evolution and preparation of this work. It would not be fair on my part, if I don’t say a word of thanks to all those whose sincere advice made this period a real educative, enlightening, pleasurable and memorable one.

I feel it my profound privilege to record my deep sense of gratitude and sincere thanks from the core of my heart to my thesis supervisor, Professor **Dr. Naveen Kwatra** for his invaluable guidance, constant encouragement, friendly attitude, immense patience, useful discussion and peerless criticisms during the course of investigation which helped me to learn something from him at each and every moment of my degree.

It is a great privilege for me to express my esteem and profound sense of gratitude to **Dr. Shruti Sharma**, Associate Professor, for her continuous guidance, constructive and valuable suggestions.

I would like to thank **Dr. Prempal Bansal**, Head, Department of Civil Engineering for providing me facilities required for completion of the research work. A big thanks to Doctoral committee members **Dr. Dwarkanath Ratha**, **Dr. A. B. Daine Roy** and **Dr. J. S. Saini** for their cheerful encouragement, commendable guidance and noteworthy remarks.

I owe my degree to my better half ‘**Sarabjot**’ who always encouraged me to complete this journey. I find no words to acknowledge the sacrifices, love, care, help, moral support rendered by her. Without her positive attitude it would never have happened.

I express my deep sense of affection to my family, my respected **mother** and for her blessings she shower on me. Thanks are due to my brothers **Jajbir** and **Baljinder** and sister in laws **Davidar**, **Komal** and my sister **Balwinder** for their love, care and support. Thanks to my nephews **Jagdev**, **Gurbaksh**, **Hardial**, **Jagbir**, **Gurdial** and niece **Simran** and **Lavleen** for their nice chats, love and care to boost my morale.

A big thanks to my in-laws family my **Mama**, **Papa** for their ever encouraging moral support and blessings. A big thanks to my sister in laws **Tajinder di**, **Deep di** and **Bindu** brother in laws **Bikram Virji**, **Harjinder Virji** for their love, care and support. Thanks to my dear most nephews **Akshdeep**, **Mankaranbir** and niece **Arshdeep**.

I would also like to offer my sincere thanks to all faculty, teaching and non-teaching of Civil Engineering Department (CED), and staff of the central library, Thapar Institute of Engineering and Technology, Patiala for their assistance.

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ABSTRACT

Masonry infills in a building frame are normally considered as architectural elements, and their presence is often ignored by design engineers. Existence of the infill walls basically provides higher stiffness and strength for the frames, but their detrimental effect on the structural performance is ignored due to a lack of adequate information about the frame. It has been observed through studies on 2D brick infilled frames that these non-structural elements had a significant effect on the seismic lateral capacity of the buildings but at the cost of ductility. However, very little work has been reported on 3D brick infilled RC framed structures. Secondly study on 3D frame will gave actual and accurate results of the behaviour of structure. It is therefore, very important to study the performance of brick infilled 3D RC framed structure under lateral load.

Most of the structures continually accumulate damage during their service life. The damages observed in buildings and other structure during an earthquake initiates the need of exploring the possible new techniques for retrofitting and strengthening the structures. Generally retrofitting of these types of buildings include addition of new members such as shear wall and bracing or strengthening of existing members by Fibre Reinforced Polymer (FRP) like Glass Fibre Reinforced Polymer (GFRP), Carbon Fibre Reinforced Polymer (CFRP), steel or RC jacketing etc. Strengthening of infill can also be done using Engineered Cementitious Composites (also known as ECC Concrete) has been contributing to safer, more durable, and sustainable concrete infra-structure which is cost-effective.

In this research effort the load deflection behavior and the change in dynamic characteristics of brick infilled RC frame with increasing lateral loads is studied. A brick infilled RC frame model (1/3 scaled) has been constructed in the laboratory and subjected to increasing lateral loads. The load deflection behavior has been studied till failure. The damaged frame has been retrofitted by using two prominent techniques –wrapping using GFRP/CFRP laminates at failure locations and by using ECC after replacing the damaged steel in the soft storey. The effects of GFRP/CFRP retrofitting and ECC strengthening have been reported.

The change in the dynamic characteristics using impact hammer excitation has been studied for increasing loading and subsequent damage. These include natural frequency, damping coefficient, and FRF magnitude for the three-storey RC frame model. Change in dynamic characteristics has been studied after subjecting control and retrofitted RC infilled frames to different levels of damage. Nonlinear analysis using Finite Elements (FE) has been performed to model the control and FRP retrofitted frames.

Results of retrofitted frame show that the use of GFRP/CFRP wrap for retrofitting provided significant lateral load capacity, as compared to the control frame. The ECC strengthened frame shows higher load carrying capacity than the control and FRP retrofitted frame and possess better post

yield behaviour. The damage indexes of FRP retrofitted and ECC strengthened frame have been reduced, indicating better performance as compared to the control frame. Damage indices based on deformation and change in stiffness showed a much more acceptable accuracy correlation with dynamic damage indices in general. The ECC strengthened frame showed significant reduction in FRF magnitude with progressive damage. On the other hand, the damping ratio showed a significant increase as compared to the control frame and FRP retrofitted frame. The ultimate loads of the nonlinear FE models showed good agreement with the data obtained from the experimental tests. Crack patterns at the final state in the FE model corresponded quite well with the failure modes of the experimental frames.

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CHAPTER 1

INTRODUCTION

1.1 GENERAL

Masonry infills can be frequently found as interior and exterior partitions in Reinforced Concrete (RC) and steel structures. Since they are generally considered as architectural elements, their presence is mostly ignored by engineers. However, even though they are considered as non-structural, they tend to interact with the bounding frame when the structure is subjected to strong earthquake motion. Such interaction may or may not be beneficial to the performance of the structure, and has been a subject of many debates. An ideal model of a bare frame structure is considered usually in order to analyze and design the structure, which undoubtedly has significant differences with its actual model. Existence of the infill walls basically provides higher stiffness and strength for the frames, but their detrimental effects on the structure performance is ignored due to lack of adequate information about the behaviour of frames and infill walls.

Masonry infill walls in frame structures have been long known to affect the strength and stiffness of the infilled frame structures. Many researchers and engineers expect that the inclusion of masonry infills will always increase the seismic resistance. Further, the composite action of the frame-infill system changes the distribution of forces in the frame members. In addition to that, it may also cause torsion in buildings, which leads to catastrophic failures of either a part of or the building as a whole. It is, therefore, necessary to evaluate building condition considering the effect of infill walls in the RC framed structures. Zhou et al. (2018) studied the effect of masonry infill walls in RC frames which lead to short column effect on the columns. Teguh (2017) studied the behaviour of 2D brick infilled RC frames under cyclic loading and reported that infill walls may reduce the ductility of the RC frames and change the crack development and failure mode of frame. Qian and Li, (2017) reported that masonry infilled panels could increase the first peak load and initial stiffness. It has been observed by different researchers on 2D brick in infilled frame that these nonstructural elements had beneficial effect on building's seismic lateral capacity but at the cost of ductility. However, very less work has been reported on 3D framed RC structure and it will give actual behaviour of 3D infilled frame. It is therefore, vital to study the performance of masonry infilled 3D framed RC structure under lateral load.

Structural health monitoring (SHM) using various existing techniques provides a means to assess the structural integrity and properties of existing structures in all aspects. Most of the structures continually accumulate damage during their service life. Damage of structures may also be caused by global instability. Alternating actions over a period of time/overloading/ageing may cause stiffness and strength deterioration, especially in RC members. The net effect is the erosion of the ductility and the energy dissipation capacity. At present, there are many methods used in measuring structural damage, some better than others, but none is without its limitations. Monitoring damages in structures using vibration diagnostics is an efficient and established non- destructive tool. The approach of vibration monitoring for damage detection defines the baseline vibration signature of the structure before damage occurs. Many researchers found that if the new vibration signature attained during a routine operation deviates from that of the baseline value, the building should be checked for possible local defects. Typical vibration identification assumes that the dynamic property of a structure is a sensitive indicator of its physical integrity. When any of the properties like mass, stiffness, or damping of the structure changes due to a structural defect, the vibration response of the structure will also change (Vimuttasoongviriyia et al., 2009; Goyal, 2007; Dincal, 2005). These dynamic characteristics can be estimated directly by vibrating a structure using digital signal processing equipment and Fast Fourier Transform (FFT) routines (Rao, 2000; Reddy et al.,1994; Meirovitch, 1986). Damage detection by computing the change in modal parameters, such as natural frequency, mode shape, and Frequency Response Function (FRF) gives an indication of the level of deterioration in the structure. It should be noted that even after repair it is more important to monitor the health of the building as cracks tend to be covered by layers of strengthening material.

The damages observed in buildings and other structures during an earthquake initiates the need of exploring the possible new techniques for retrofitting and strengthening the structures. In addition to this up gradation of existing structure with respect to current codal recommendation or to satisfy the current requirements also give a need to strengthening of existing structures. Generally retrofitting of these types of buildings include addition of new members such as shear wall and bracing or strengthening existing members by Fibre Reinforced Polymer (FRP) like Glass Fibre Reinforced Polymer (GFRP) and Carbon Fibre Reinforced Polymer (CFRP), steel or RC jacketing etc. Strengthening of infill may also be done using FRP or steel plate bonding.

FRP strengthening is one of the prime techniques in strengthening of an existing structure because of its various advantages such as lesser weight higher strength and flexibility to use in all kinds of structural elements easy workmanship etc. Glass Fiber Reinforced Polymers (GFRP) and Carbon Fiber Reinforced Polymers (CFRP) are rapidly becoming a superior alternative to traditional steel reinforcements for RC structures. Pohoryles et al. (2019) highlighted the important effect of realistic size, loading and geometry of test specimens on FRP retrofit effectiveness. Zhuang et al. (2018) tested RC beams strengthened with CFRP and reported that yield load growth ranges from 5% to 36% due to CFRP reinforcement. Chisari and Baden (2017) found that strength and ductility of columns is enhanced by the use of Fibre-Reinforced Polymer (FRP) in forms of wraps glued to the columns. Eslami et al. (2016) studied the seismic retrofitting of existing RC buildings under near-fault ground motions using carbon FRP (CFRP) sheets and reported that retrofitting schemes can substantially improve the seismic behavior of RC structures subjected to impulsive seismic motions. Ronagh and Eslami (2013) reported better performance of CFRP over GFRP in flexural strength and lateral resistance of the structure. Yuksel et al. (2010) studied the behavior of 2D bare and infilled carbon fiber reinforced polymer (CFRP)-retrofitted infilled RC frames with different bracing configurations At severely damaged locations GFRP has not performed well, whereas it performed well for other locations. CFRP performed very well but is very expensive. In all it becomes very imperative to study the repair of masonry infilled frame using a judicious combination of GFRP/CFRP wrapping at critical failed locations.

Recently Engineered Cementitious Composites (ECC, also known as ECC Concrete) have contributed to safer, more durable, and sustainable concrete infra-structure that is cost-effective and constructed with conventional construction equipment. With two percent by volume of short fibers. ECC has been prepared in ready-mix plants and transported to construction sites using conventional ready-mix trucks. ECC is ductile in nature. Under flexure, normal concrete fractures in a brittle manner. In contrast, very high curvature can be achieved for ECC at increasingly higher loads, much like a ductile metal plate yielding. Extensive inelastic deformation in ECC is achieved via multiple micro-cracks with widths limited below 60 microns. Use of high ductility concrete in seismic zones is recommended due to its high seismic response. In retrofitting of large structures, cost-minimization of materials with maximization of seismic resistance is desirable. Advanced strengthening

techniques require lesser labor and its immediate response to the need leads one to choose them instead of conventional techniques.

ECC has been shown to exhibit excellent behavior under tensile and shear loading (Li et al., 1994) and possesses high fracture energy and notch insensitivity (Maalej and Li 1995), and can therefore be viewed as ideal materials for various structural applications. The material also exhibits pseudo strain hardening and therefore energy dissipation (Van Mier 2004). ECC has been used very effectively for retrofitting of various components of RC structures. Qudah and Maalej (2014) concluded that the use of ECC material in the connection plastic zone as replacement of concrete and partial replacement of transverse (confinement) reinforcement can significantly enhance the joint shear resistance, energy absorption capacity, and cracking response, thereby, enhancing the joint seismic resistance and reducing reinforcement congestion and construction complexity. Dehghani et al., (2015) proposed that ECC-strengthening technique can effectively increase the lateral strength and energy absorption capacity of the infilled frame, prevent brittle failure modes in the infill wall, and provide a reasonable strength. Lin et al., (2016) reported that when 30 mm of ECC overlay was applied onto the compression surface of a unreinforced masonry (URM) wall. The strength of the wall increased by 170% when compared to the average strength of the two as-built URM walls. Hence the use of ECC as concrete overlay has been suggested by various researchers for repair in RC element as well as RC frames. In this work it is intended to investigate the performance of masonry infilled RC frame retrofitted with ECC at critical concrete damage locations by replacing damaged/ replacing yielded steel with new steel in the plastic hinge region of the frame.

Finite Element (FE) analysis is used to examine the effects of internal tensile reinforcement on the magnitude of principal tensile stresses in the critical region. Many researchers have worked on FE analysis of beams and column repaired with FRP laminates (Kumari&Kwatra (2013),: Skafida et al., (2014),: Shanmugavelu et al., (2016),: (Shangari&Elangovan (2016),:Zadeh&Eshghi (2018),) Most of these studies have reported only load-deflection behavior vis-a-vis the form of composite overlay bond failure on strengthened structural elements. In this research effort the non-linear behavior and the change in dynamic characteristics of RC frame models with and without damage under lateral loads were studied. The dynamic characteristics studied included in the study were natural frequency, damping coefficient, and FRF magnitude of the three-storey RC frame model. Change in dynamic characteristics have been studied after subjecting RC infilled frames to

different levels of damage. Some of the damage indices have been selected to indicate the damage for both non-linear static and dynamic approaches. The masonry infilled RC frame repaired with GFRP and CFRP and the load deflection behavior and vibration monitoring were investigated in the same manner as the control frame. A three-dimensional FE model has been developed to assess the performance of brick infilled RC frame before and after FRP retrofitting subjected to lateral loads. A comparison of FE and experimental frame has been done.

1.2 OBJECTIVES OF INVESTIGATION

Considering the above gaps, the following specific objectives have been laid down for the research work:

- To study the load deflection behaviour and dynamic characteristics of three-storey reinforced concrete masonry infilled frame models detailed with reinforcement as per IS-456 subjected to different damage levels (lateral loads).
- To study the load deflection behaviour and dynamic characteristics of the retrofitted model of masonry infilled RC frames which are subjected to different damage levels after strengthening using FRP.
- To study the load deflection behaviour and dynamic characteristics of the masonry infilled RC frames which are subjected to different damage levels after strengthening using ECC.
- To carry out mathematical nonlinear modelling of RC frames (with and without damage) and of retrofitted frames using FRP.

1.3 ORGANIZATION OF THESIS

The thesis is presented in seven chapters as detailed below:

Chapter-1 Introduction

Chapter-2 In this chapter, relevant review of literature presents the work done by various researchers in the field of retrofitting, describing structures with GFRP, CFRP laminates, ECC strengthening of RC structure, damage detection and vibration monitoring and finite element modeling of retrofitted structures.

Chapter-3 Details the scheme of experimentation, materials used model of structures, methodology of testing and strengthening scheme. The mechanical properties of

concrete, steel reinforcements and GFRP, CFRP laminate, ECC strengthening of RC structure generated from the experimental program have also been presented in this chapter.

Chapter-4 Presents definitions of the fundamental response parameters of a structure under lateral load. This chapter also presents load-displacement behavior results, failure mechanism of experimental tests and a discussion.

Chapter-5 Presents vibration measurement technique and structural dynamic characteristic results of experimental tests with discussion and correlation of damage indices based on the change in dynamic characteristics.

Chapter-6 Deals with the finite element model developed for analysis of the non-linear and dynamic behaviors of RC structures for both control and retrofitted frames using GFRP and CFRP sheets. The comparative analysis between FE model and experimental results is presented in this chapter.

Chapter-7 This chapter presents the significant conclusions drawn from this study and recommendations for future work.

References and Appendices follow in succession.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

Reinforced concrete is a complex composite material prepared from a combination of two materials, concrete and steel, with completely different mechanical properties. Although many successful RC structures are built around the world each year, there are a huge number of structures that depreciate or become dangerous due to changes in load, use, configuration or natural hazards. A large number of present buildings were built in northern India and around the world when seismic areas were often unrecognized, resulting in structural elements designed only for gravity loads (Kanwar et al., 2006). Concrete is the world's most important building material. Under real field conditions, however, the concrete is also subjected to stress and environmental influences, chemical attack and thermal effects. The tensile strength of concrete is only 10% of its compressive strength. The main drawback of concrete is its fragile nature and, due to its fragile nature, cracks, destruction and deterioration occur and repeated preservation of the structural elements is required. (Sahmaran et al., 2009). Existing RC buildings, although characterized by high flexural strength and stiffness, have uncertain resistance. These RC buildings are made of beams and columns, and each of these elements in a frame contributes to its resistance. In earthquakes or fast winds, these structures are subject to increasing base shear (Paz, 1997, Mo, 1994, Newmark, 1991, Dowrick, 1988). These lateral loads can damage the structural elements by forming various plastic hinge mechanisms.

The concept of structural health monitoring for the maintenance of existing buildings is making steady progress. The trend towards the development of new technologies has accelerated following recent natural hazards that pose the problem of rapid damage assessment of existing structures (Dincall, 2005, Fang et al., 1998, Stubbs et al., 1992, Park and Ang 1985). Most building structures continually accumulate damage during their lifetime. Damage to building structures can be due to a permanent aging process. To ensure seismic safety, monitoring is required. Unrecognized damage can lead to more seismic damage from future earthquakes and eventually catastrophic failures (Bracci et al., 1997). Therefore, monitoring and rapid detection of building damage is essential.

The structural analysis by measuring and investigating vibration signals is a recognized technique. This technique first defines the signature of the fundamental of a feature before

damage occurs. If the new vibration signature obtained during a routine operation deviates from that of the baseline value, the structure should be checked for possible local defects (Rao, 2000; Thomson, 1993). The typical vibration diagnosis assumes that the dynamic property of a structure is a sensitive indicator of its physical integrity. When any one of the mass, stiffness, or damping properties of the structure changes due to a structural defect, the vibration response of the structure will also change. In modal structural testing, accelerometers have been widely used as sensors to obtain vibration signals because of their convenience of application and accuracy (Reddy et al., 1994). These dynamic characteristics may be determined from frequency response data, which can be measured directly from the vibrating structure using digital signal processing equipment and FFT routines. When compromise to the integrity of the structure is found, strengthening through retrofitting is necessary. Generally, brittle RC structures break easily. If the columns of brittle structures can be strengthened, this will inhibit the formation of storey mechanisms and will also inhibit the formation of overall structural drifts.

There are several ways to strengthen the elements of a structural system. Traditionally, the affected parts of the structures were repaired with steel plates or reinforced concrete. Although this method can be cost effective, it is slow, dust-prone, unreliable and difficult to implement. It also adds a considerable mass to the structure. Fiber reinforced polymer alleviates all these difficulties. In addition, they have a very high specific strength (strength-to-weight ratio), specific stiffness (stiffness-to-weight ratio) and increased resistance to corrosion (Balaguru, 2003). GFRP materials are rapidly becoming a superior alternative to traditional steel reinforcements for RC and masonry structures in North America, Western Europe and Japan. In this cultural and economic context, these materials have been proposed for a variety of construction and repair applications, including bridges, buildings, power plants, parking lots and marine structures (Kachlakev and McCury, 2000, Triantafillou, 1998, Al - Sulaimani, 1994, Bhutta, 1993). Important literature on vibration monitoring as well as on the adaptation or amplification of stressed elements is available. A short literature review on this topic will be discussed and presented in this chapter with the following subtitles:

2.2 Retrofitting of structural elements with GFRP and CFRP

2.3 Behaviour of the RC structure strengthened with ECC

2.4 Damage detection and vibration monitoring of RC structures

2.5 Finite element modelling of retrofitted structures

2.2 RETROFITTING OF STRUCTURAL ELEMENTS WITH GFRP AND CFRP

Today, reinforcement of RC elements is mostly achieved with GFRP materials, since this type of reinforcement has excellent properties such as light weight and high corrosion resistance and is therefore often used for repairs and retrofits. Researchers conducted various studies on strengthening of frame, beams, column, beam-column joint or buildings RC structures by using GFRP/CFRP or any other FRP material. Studies of different researchers are discussed below under subheadings:

Jia et al. (2005) studied the effects of different environmental conditions in conjunction with sustained load and the application of external FRP reinforcement including to degradation due to environmental conditioning. A procedure to quantify the degradation of FRP and concrete bond due to environmental conditions in the framework of nonlinear mechanics was formulated. The effect of application of load in one of four environments was studied: freeze-thaw, air-conditioned laboratory environment, real time outdoor, and elevated dry temperature. It was observed that the ultimate strength of the specimens did not change appreciably as a result of sustained loading; localized slip at the concrete-FRP interface underwent substantial changes, including a dramatic reduction in the amount of energy required to effect debonding of the sheet at any given location. Also it was expected that over a long term, the debonded length can increase and that the long term deflections can also increase substantially.

Yuksel et al (2010) conducted an experimental study on the behaviour of bare and carbon fiber reinforced polymer (CFRP)-retrofitted infilled RC frames with different bracing configurations. Quasi-static experimental results are presented and discussed on six 1/3-scaled infilled RC frames that were retrofitted using CFRP material in various schemes. The test results showed a significant increase in the yield and ultimate strength capacities of the frames with a decrease in relative storey drifts, especially in the cross-braced and the cross diamond-braced type of retrofitting schemes. The energy dissipation capacities of the retrofitted frames turned out to be more than those of the bare infilled frame, thus reducing the seismic demand imposed on the frames. The cross diamond-braced type of retrofitting scheme, which was positioned on the infill wall and outside the beam-column connection regions of RC frame, showed the best behaviour among the other schemes. This scheme not only prevented brittle shear failures of the infill wall, but also prevented the transfer of additional forces to the weak and brittle beam-column connections.

Mulgund et al. (2011) observed that calculation of earthquake forces by treating RC frames as ordinary frames without regards to infill leads to underestimation of base shear. The configuration of infill in the parking frame changes the behaviour of the frame therefore it is essential for the structural systems selected, to be thoroughly investigated and well understood for catering to soft ground floor. The performance of fully masonry infill panels was significantly superior to that of bare frame and soft storey frames. The present study also demonstrated use of nonlinear displacement based analysis methods for predicting performance based seismic evaluation.

Akin et al. (2014) analysed the effect of CFRP in the RC framework and found that the applied CFRP modification scheme did not increase the final resistance of the RC frames compared to the filling walls of hollow clay tiles (HCT) increased by a factor of 2 to 3. The parametric study found that the greatest increase in resistance was achieved in the reconditioning of HCT walls with an aspect ratio between 0.8 and 1.2. A significant rise in ultimate strength can be attained by reinforcing the HCT filler walls with CFRP having an aspect ratio between 0.8 and 1.2. With CFRP modifications however no drastic increases in stiffness were expected. The improved drift properties provided by the CFRP modification applied to existing filler walls tend to decrease as the aspect ratio increases.

Ozkaynak et al. (2014) conducted a study and examined the equivalent damping ratio of carbon fiber reinforced polymer that was adjusted in full RC systems using a series of 1/3 frames a bay and a floor. Quasi static tests were performed on eight samples with two different load patterns: one-cycle and three-cycle shift histories and pseudo dynamic tests on eight samples for selected acceleration records scaled to three different PGA levels with two conditions of inertia mass. The equivalent damping for the bare frame varies between 8%, and 11% depending on the damage level. The equivalent damping ratio for the entire frame was 13%. The influence of the filling walls on the damping was therefore evidently visible in the outcomes.

Cao and Ronagh (2014) studied the in-elastic time history and damage analyses of an 8-storey frame designed for 3 different conditions as 1) poorly-confined, 2) seismically detailed to the “intermediate” and 3) retrofitted by FRP confinement were performed at different seismic intensities. The confinement effect of FRP on the damage of the poorly-confined frame was investigated with the reference to the “intermediate” frame. Although the poor confinement of the transverse reinforcement was neglected, the effect of FRP confinement was confirmed to significantly reduce the damage index of the retrofitted frame by 0.33, 0.51 and 0.42 in comparison to that of the original if subjected to the seismic

intensities of 0.30g, 0.45g and 0.60g, respectively. Consequently, the damage of the poorly-confined frame was brought down one or two damage levels. The retrofitted frame suffered less damage than the “intermediate” frame also confirmed the significant effect of FRP confinement as presented in Figure 2.2. These demonstrated the FRP external confinement as an appropriate retrofitting solution for RC structures poorly confined by the internal transverse reinforcement. With this retrofitting solution, poorly-confined RC frames can be upgraded to seismically designed frames. This significant effect of FRP confinement was worth taking into account when retrofitting RC frames with the deficiency of transverse reinforcement.

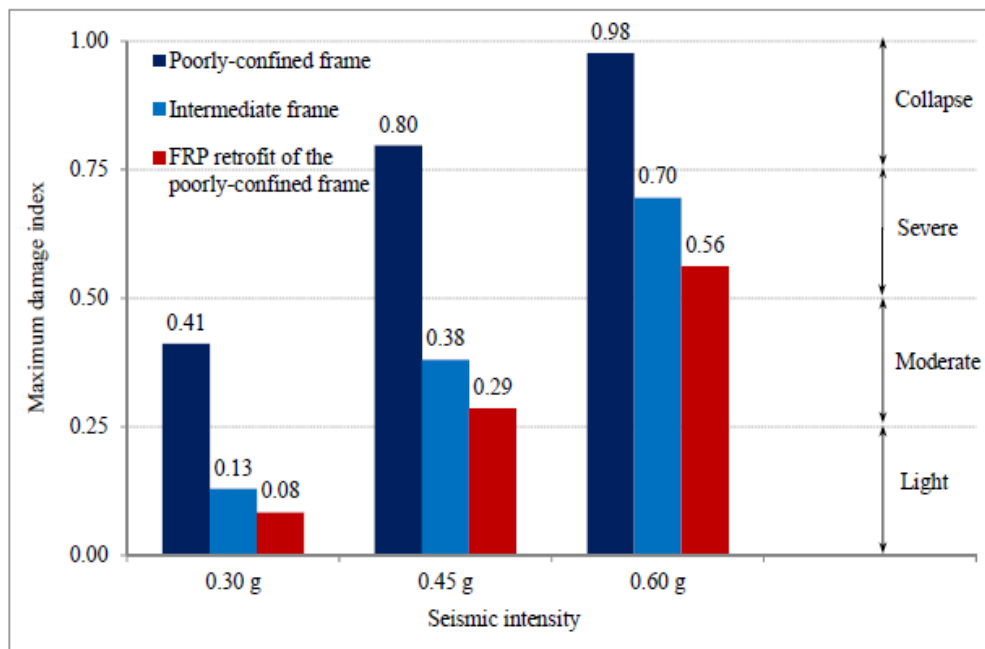


Figure 2.1 Damage index under different types of frames (Cao and Ronagh 2014)

Sathiaselvan and Arulselvan (2015) concluded that earthquake-damaged RC frames need to be effectively remediated by applying ferro-cement with cement mortar. In addition, they concluded that the retrofitted frame (RF) could carry an additional load of 30 per cent over the originally designed frame (F) load as shown in Figure 2.1. It could be noted that due to the proper interface between the frame and the filling the adverse effect of the filling was removed. The application of the ferro-cement technique is cost effective and can be done with fewer skilled workers.

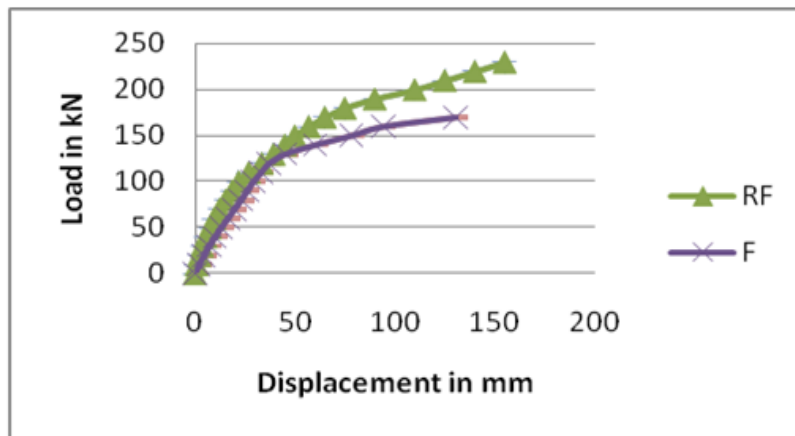


Figure 2.2 Load vs. Displacement (*Sathiaseelan and Arulselvan, 2015*)

Singh et al. (2017) revealed that fiber reinforced polymer (FRP) sheets are the most efficient structural materials in terms of strength to weight ratio and its application in strengthening and retrofitting of a structure or structural elements are inevitable. The performance enhancement of structural elements without increasing the cross sectional area and flexible nature are the major advantages of FRP in retrofitting/strengthening work. Study was on the inelastic response of conventional and retrofitted reinforced concrete (RC) frames using Carbon Fibre Reinforced Polymers (CFRP) and Glass Fiber Reinforced Polymers (GFRP) was subjected to quasi-static loading. The hysteretic behaviour, stiffness degradation, energy dissipation and damage index are the parameters employed to analysis the efficiency of FRP strengthening of brick in-filled RC frames. Repair and retrofitting of brick infilled RC frame illustrated an improved load carrying and damage tolerance capacity than control frame.

Bagheri et al. (2019) conducted an experiment and calculated maximum reliability index with different FRP lengths of columns of a six-storey RC frame to find the optimal length for FRP-retrofitted columns. The seismic behaviour of structure after verification was assessed by nonlinear time history analysis with Open Sees finite element software. Three scaled far-field earthquake records were selected for nonlinear time history analysis. The outcomes of the study showed that increased FRP material quantities do not affect the retrofitted reliability index. Moreover, in all cases of reinforcement and analyses, the compressive strength of the concrete is extremely important.

Nandhini et al. (2020) concluded that FRP strengthening improves the flexural, shear and compressive strength of RC elements. It was preferred for its advantages like easy application, lightweight, stiffness, high tensile strength, etc. FRP exhibits good bonds with concrete by the use of epoxy resin. The bond gets failed when only the temperature exceeds

the glass transition temperature. Nowadays bio-based resins were also developed to bond FRP on the surface of the damaged structures. FRP increases load carrying capacity of RC structures.

2.2.2 RETROFITTING WITH GFRP AND CFRP IN RC BEAMS

Li et al. (2001) conducted an experiment by strengthening the RC beams with externally bonded CFRP sheets. One beam was strengthened in flexure only, and four others were strengthened in flexure and in shear. The authors found that for the strengthened beam, the ultimate strength also significantly increased in comparison with the normal beam. The experimental results indicated that the beam strengthened in both flexure and shear had a slight advantage, in ultimate strength, in comparison with the beam strengthened in flexure.

Ceroni (2010) investigated RC beams with externally bonded CFRP laminates and Near Surface Mounted (NSM) bars under monotonic and cyclic loads. The load carrying capacity increased about 26% to 50% for reinforced concrete with lower steel reinforcement percentage and about 17% to 33% for higher steel reinforcement percentage.

Lakshmikandhan et al. (2013) observed the behaviour of CFRP strengthened beam. Undamaged control beam was strengthened with CFRP laminates and the repaired/strengthened beams were tested under monotonic load for comparison. The study had confirmed the applicability of cyclic loading method to evolve the stiffness degradation and damage assessment. The bonding strength of CFRP governed the strength of the repaired beams in most cases. The bonding of CFRP was better in the cracked beams than in the un-cracked beam. The results had also shown that the repaired damaged beams outperformed than the undamaged control beam strengthened with CFRP.

Shariatmadar (2016) conducted some tests on the FRP amplification method of damaged beam-column connections. CFRP plates were used to wrap around the critical areas of the beam, column, and junction zone of the subsamples studied to enclose these areas. The outcomes indicated that the method is effective and can restore or even improve the resilience and original stiffness of damaged joints. The performance level of damaged joints in cyclic tests has also been improved. The degree of repair capacity damage was also evaluated, for the tested B-C joints with a deviation of up to 1.5%. However, the plastic rotation and ductility ratio of the critical beam portion and the bonding surface decreased due to the elastic nature of the FRP materials.

Chen et al. (2018) conducted an experiment to explore the effectiveness of application of Basalt FRP (BFRP) to strengthen RC beam under three-point bending test. The

damage modes and structural response of un-strengthened and BFRP strengthened RC beams were recorded and identified. They reported that FRP strengthening was effective to enhance the structural load-carrying capacity. Basalt FRP (BFRP) was a promising material for the application to structure strengthening with its advantages of low cost, corrosion resistant and sound mechanical property.

Siddika et al. (2019) revealed that FRP possesses high tensile strength and can increase the ultimate strength of RC beams up to twice that of the original RC beam when strengthened, though FRP had low elastic modulus. Additionally, RC beams demonstrated superior shear and torsion resistance capacities when wrapped in FRP layers at an angle and when additional anchorages are applied. The type of anchorage used in FRP strengthening was dependent on the strengthening scheme, loading conditions, and practical considerations. FRP possesses high stiffness, resistance to spalling and fragmentation, good energy absorption capacity, and ductility. Therefore, FRP-strengthened concrete elements can be used to develop structures with high impact and blast resistance along with seismic resistance.

2.2.3 RETROFITTING WITH GFRP AND CFRP IN COLUMNS

Sheikh (2002) reported that retrofitting with fibre reinforced polymers (FRP) to strengthen and repair damaged structures is a relatively new technique. In an extensive research program underway at the University of Toronto, application of FRP in concrete structures was being investigated for its effectiveness in enhancing structural performance both in terms of strength and ductility. The structural components tested so far include slabs, beams, columns and bridge culverts. Research on columns has particularly focussed on improving their seismic resistance by confining them with FRP. All the specimens tested can be considered as full-scale to two-third scale models of the structural components generally used in practice. Results so far indicate that retrofitting with FRP offers an attractive alternative to the traditional techniques. In many circumstances, it can provide the most economical (and superior) solution for a structural rehabilitation problem.

Chang et al. (2004) investigated pseudo-dynamic testing of two bridge columns with a reduced scale of two is to five. The seismic performance of two damaged bridge columns was recovered after repairing with CFRP sheets. Three identical RC bridge columns were fabricated. One column was used to perform the reversed cyclic loading test in order to obtain the basic properties of these specimens, and the other two were tested pseudo-dynamically so that accurate seismic responses and reliable hysteretic loops of the RC bridge columns under

near fault ground motions could be achieved. In addition to pseudo dynamic testing of the as built bridge columns, the damaged bridge columns were repaired with carbon fiber reinforced polymer and retested pseudo-dynamically. The repair of these specimens was focused in the plastic hinge zone. Three layers of the CFRP composite sheets were applied in the potential plastic hinge zone, which was about one-fourth of the column height. They found that the strength and ductility of the repaired bridge column may be largely restored after the CFRP composite sheets are engaged and external confinement is developed at larger lateral displacements. The displacement response of the same column before and after repair is very similar except that a slightly larger displacement response occurs during testing of the repaired specimen. Also there were no observable flexural cracks on the outside CFRP layer along the fiber direction and no tension failure of the CFRP composite sheet was seen.

Ghobarah and Elmandoohegalal (2004) tested three rectangular short columns under cyclic lateral loads and constant axial load. One specimen was tested as control specimen, while two specimens were strengthened using three layers of CFRP. They found that several short RC columns failed in brittle shear, when subjected to lateral cyclic displacements. The strengthening of a short RC column that contains high percentage of transverse reinforcement using anchored CFRP sheets prevented its brittle shear failure and significantly improved the displacement ductility and energy dissipation capacity. Anchoring techniques used were effective in improving the column confinement and in reducing the concrete bulging at the column sides. Regarding the crack pattern several 45° cracks were observed at first yield. The number of cracks increased and cracks from the push and pull loading crossed forming a diamond shaped pattern at the middle of the column and zigzag shapes near the edges.

Mukherjee et al. (2004) presented the mechanical response of concrete columns confined with FRP composites. Practical columns often deviate from axi-symmetric conditions due to noncircular cross section, geometric imperfections, and loading eccentricities. This study discusses these complicating effects on the mechanical behaviour of columns confined with FRP. Experiments had been carried out to examine the effects of geometric and loading imperfections on columns of various shapes. A model originally developed for axisymmetric situations had been extended to include the complicating effects. An analytical study for the corner radius that avoids concentration of stress was carried out. The theoretical models had been verified with the present and published experimental results.

Memon and Sheikh (2005) studied the effectiveness of strengthening deficiently built columns as well as repairing damaged square columns with GFRP for seismic resistance. Eight specimen structures consisting of square columns cast integrally with a stub were

studied. Cyclic loading was applied by pushing downward on the stub near the columns. It was found that at a low axial load level, the structures showed an almost elastic behaviour for the first three cycles in all the tests. In the later cycles, the first sign of the deformation was the formation of the ridges or bumps in GFRP on the top and the bottom surfaces of the specimens in the test region. The ridges mostly initiated near the column-stub surface. In all, the high axial load tests, deterioration in the hinging zone was quite rapid. Also in most of the specimens, the formation of ridges started in the third cycle downward. The overall ductile performance depends on the extent of damage sustained by the specimens prior to being wrapped. It was also seen that more GFRP layers were needed for highly damaged columns to achieve a performance similar to that of undamaged retrofitted columns. A larger amount of GFRP is therefore required for columns subjected to higher axial load levels to realize similar performance enhancements as those demonstrated by retrofitted columns under lower axial loads.

Lakshmikandhan et al. (2012) presented the behaviour of retrofitted beams from the experimental and analytical studies. The results had showed that the width and stiffness of CFRP affected the failure modes of retrofitted beams. The increase of CFRP stiffness had increased the maximum load to a certain value, thereafter the stiffness increase resulted in decreases of load capacity. The use of composite materials such as fibre reinforced polymers in strengthening and repairing of structural elements, particularly those made of reinforced concrete, is widely spreading. However, for successful and cost-effective applications, engineers must improve their knowledge with respect to the actual behaviour of strengthened structures. Therefore, glass fibre reinforced polymers, because they are more ductile and cheaper than carbon fibres, can be considered as an alternative solution to repair and strengthen concrete elements. The present study was conducted to examine the efficiency of external strengthening systems for reinforced concrete beams using FRP fabric (Glass–Carbon). In order to address this problem, different strengthening configurations are considered (use of separate unidirectional glass and carbon fibres with some U-anchorage or of bidirectional glass–carbon fibre hybrid fabric). A total of seven flexural strengthened reinforced concrete beams are instrumented and tested under repeated loading sequences using a 4-point bending device to complete a failure analysis. The results for strength, stiffness, ductility and failure modes are discussed for the various strengthening solutions considered. An analytical model to predict the flexural failure of strengthened concrete elements was also developed. The results demonstrated that the model predicted strengthened concrete beam behaviour under applied loads accurately. The present tests also revealed the

cost-effectiveness of twin layer glass–carbon FRP fabric as a strengthening configuration for reinforced concrete structures.

Omar and Stefan (2014) revealed that significant increase in strength and ductility of concrete can be achieved by CFRP composite jackets, the confinement modulus and the confinement strength of the composite jacket had been identified as the two critical parameters in describing the system confinement effectiveness. Also, the ultimate confined concrete which was determined by the rupture of the composite jacket (rupture strain), was much lower than the rupture strain obtained from flat tensile coupon samples which refined constitutive model of concrete confined by FRP.

Ghatte et al. (2015) had attempted to improve the seismic performance of lower quality RC columns, which have been adjusted by external FRP constraints. They found that FRP inclusion of lower quality columns with a cross-sectional aspect ratio of two can significantly improve the behaviour under lateral loads acting in the direction of the column's weak axis.

2.2.4 RETROFITTING WITH GFRP AND CFRP IN BEAM-COLUMN JOINT

El-Amoury and Ghobarah (2002) presented experimental results of tests carried out on plain RC and GFRP reinforced RC beam-column joints, the possibility to avoid the shear failure of the joint through the externally bonded composite reinforcement and forcing the plastic hinge to form by flexure at the beam end. In addition, they strengthened the bottom part of the beam end in such a way to integrate the existing non-adequately anchored steel reinforcement. The latter GFRP sheets reduced fixed-end rotation effects improving the sub-assembly stiffness.

Said and Nehdi (2004) studied the effect of beam-column joint rehabilitation techniques using FRP. They presented forces and stresses in interior beam-column joints. A full-scale code-conforming beam-column joint was made and tested under reversed cyclic load to serve as a benchmark for this comparison. Enhancements imparted to substandard beam-column joints by FRP rehabilitation techniques in terms of strength, ductility and energy dissipation gains are assessed. They found that FRP joints repair schemes generally enhanced the performance of substandard joints, but they often came short of satisfying current standard level performance, and that different rehabilitation strategies can be adopted depending on the type of joint deficiency and the purpose of the rehabilitation scheme.

Mukherjee and Joshi (2005) investigated the performance of RC beam-column joints under cyclic loading. Joints had been casted with adequate and deficient bond of

reinforcements at the beam-column joint. FRP sheets and strips have been applied on the joints in different configurations. The columns were subjected to an axial force while the beams are subjected to a cyclic load with controlled displacement. The amplitude of displacement is increased monotonically using a dynamic actuator. The hysteretic curves of the specimens have been plotted. The energy dissipation capacity of various FRP configurations has been compared. In addition, the control specimens had been reused after testing as damaged specimens that are candidates for rehabilitation. The rehabilitation had been carried out using FRP and their performance has been compared with that of the undamaged specimens.

Shrestha and Smith (2007) revealed that the shear capacity of reinforced concrete (RC) beams-column connections can be enhanced by the addition of externally bonded fibre reinforced polymer (FRP) composites. The FRP can also increase the deformation capacity and energy absorption capacity of the connection. Tests to date had typically involved the application of load in a cyclic push-pull manner and, depending on the design, the connection can fail in various modes ranging from FRP debonding and FRP rupture to failure in the beam, column or joint region. The cyclic nature of the loading however makes it quite difficult to gain a detailed understanding of the behaviour of the FRP alone. The connections were subjected to monotonic loading which allowed the behaviour of the FRP to be closely monitored. The experimental results can serve as a good set of data for future numerical and analytical models to be calibrated against. Finally, experimental results were compared with analytical models.

Wang and Hsu (2009) investigated the performance of RC beam-column joints under cyclic loading. FRP sheets and strips have been applied on the joints in different configurations. The columns were subjected to an axial force while the beams are subjected to a cyclic load with controlled displacement. The amplitude of displacement was increased monotonically using a dynamic actuator. The hysteretic curves of the specimens have been plotted. The energy dissipation capacity of various FRP configurations has been compared. In addition, the control specimens had been reused after testing as damaged specimens that were candidates for rehabilitation. The rehabilitation had been carried out using FRP and their performance had been compared with that of the undamaged specimens.

2.2.5 Closure

On the basis of literature reviewed it has been recognised that fibre reinforced polymers (FRP) like GFRP and CFRP can give promising results in strengthening of RC

structures. Since GFRP and CFRP have very high tensile strength, shear resistance capacity, strength to weight ratio, and increased resistance to corrosion. So keeping in view, the importance of FRP strengthening this research has been planned with retrofitting the RC structures using GFRP and CFRP material. It should be noted that the use of advanced materials like GFRP and CFRP for RC structural repair and strengthening applications can be very useful in changing seismic behaviour of RC structures.

2.3 BEHAVIOUR OF THE RC STRUCTURE STRENGTHENED WITH ECC

Structural applications for plain concrete are limited due to its low tensile strength, low ductility and fragile behaviour. In addition ECC can limit the width of cracks to failure to an average of less than 60 microns. Details about the properties and behaviour of ECC have been reported by several researchers on beams, column, frame etc. as discussed follows:

2.3.1 BEHAVIOUR OF RC COULMN STRENGTHENED WITH ECC

Fischer and Li (2003) were the first to investigate the effect of the ductile deformation behaviour of ECC on the response of both steel reinforced and FRP reinforced columns under reversed cyclic loading conditions. The advantage of applying ECC mortar to concrete flexural members can significantly improve the seismic behaviour of concrete building structures, which includes greater deformation and load-carrying capacity, smaller member size, reduced reinforcement and longer span. Engineered Cementitious Composite (ECC) is one such type of FRCC which offers more durable and sustainable concrete infrastructure which is not only cost-effective but also offers the advantage of ease of construction with conventional equipments. The presence of micro fibres and the absence of coarse aggregates acts as secondary reinforcement and bridges the cracks more effectively than conventional FRC and offers strain hardening behaviour as shown in Figure. 2.5.

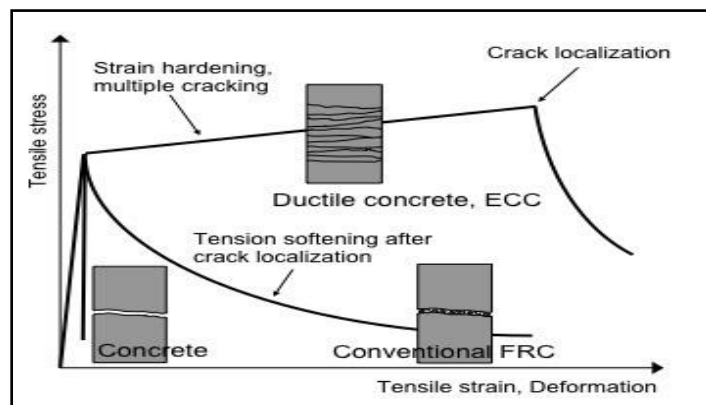


Figure 2.3: Tensile behaviour of ECC composite (Fisher and Li 2003)

Kesener and Balington (2005) investigated an infill system for frame structures as a retrofit strategy for critical facilities. The system used precast panels made with an engineered cementitious composite (ECC) material in lieu of a traditional reinforced concrete or masonry. The ECC material is advantageous due to the increased tensile strain capacity and compressive toughness of the material relative to traditional materials. Finite element simulations were performed to identify promising infill geometries. A series of structural scale tests was developed from the simulation results. Testing results indicated that different levels of panel strength and stiffness can be achieved by varying the mix design of the ECC material and the amount of reinforcement in the panels. The panel test results combined with the simulation results demonstrate the potential of the infill system as a retrofit.

Cho et al. (2012) conducted an experiment on new method of seismic strengthened reinforced concrete composite columns to improve the seismic strength and performance of reinforced concrete columns by applying high performance fiber reinforced cementitious composites (HPFRCs) instead of concrete locally in the plastic hinge region of the column. HPFRC has high ductile tensile strains about 2–5% with sustaining the tensile stress after cracking and develops multiple micro-cracking behaviours. Three specimens of reinforced concrete composite cantilever columns by applying the HPFRC instead of concrete locally in the column plastic hinge zone and one of a conventional reinforced concrete column were designed and manufactured. From the experiments, it was known that the developed HPFRC applied reinforced concrete columns not only improved cyclic lateral load and deformation capacities but also minimized bending and shear cracks in the flexural critical region of the reinforced concrete columns. The crack and failure patterns of each column from the frontal and side view are shown respectively in Figure 2.4.

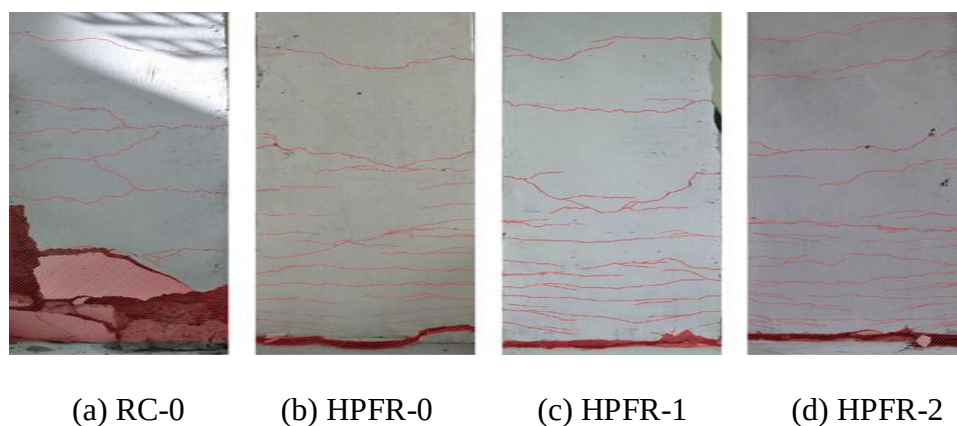


Figure 2.4 Crack pattern in different ECC strengthened columns (*Cho et al.*, 2012)

Yuan et al. (2019) proposed a novel approach for improving the seismic behaviour of members through the use of high-ductility engineered cementitious composites (ECCs) in the plastic hinge zone. Numerous hybrid FRP-steel-reinforced ECC-concrete columns were tested under reversed cyclic loading. The effects of the matrix type, reinforcement type, axial force ratio, and reinforcement ratio on the seismic behaviours, including the failure mode, crack pattern, load-carrying capacity, residual deformation, ductility, and energy-dissipation capacity of the columns were systematically investigated. The replacement of concrete with ECC in the plastic hinge zone efficiently eliminated the local buckling of the FRP bars and significantly improved the seismic performance of the column. Compared with the steel-reinforced ECC-concrete composite column, the hybrid-reinforced composite column exhibited significantly less residual deformation and higher post-yielding stiffness. With an increase in the axial force ratio the ultimate strength increased, while the deformation capacity decreased.

2.3.2 BEHAVIOUR OF RC BEAM-COLUMN JOINT STRENGTHENED WITH ECC

Yuan et al (2013) conducted an experiment to study the performance of ECC. In this experiment, number of RC/ECC composite beam-column joints had been tested under reversed cyclic loading to study the effect of substitution of concrete with ECC in the joint zone on the seismic behaviours of composite members. The experimental parameters included shear reinforcement ratio in the joint zone, axial load level on the column and substitution of concrete with ECC or not. According to the test results, for the specimens without shear reinforcement in the joint zone, substitution of concrete with ECC in the joint zone cannot change the brittle shear failure in the joint zone, but can significantly increase the load capacity and ductility of the beam-column joint specimens, as well as the energy dissipation ability due to high ductility and shear strength of ECC material. For the specimens with insufficient or proper shear reinforcement ratio, substitution of concrete with ECC in the joint zone can lead to failure mode change from brittle shear failure in the joint zone to a more ductile failure mode, i.e. flexural failure at the base of the beam, with increased load capacity, ductility and energy dissipation ability. Increase of axial load on column and shear reinforcement in the joint zone had little effect on seismic behaviours of the members when they failed by flexural failure at the base of beam. In a word, the substitution of concrete with ECC in the joint zone was experimentally proved to be an effective method to increase the seismic resistance of beam-column joint specimens.

Quadah and Maalej (2014) studied the performances of a series of ECC-enhanced beam–column interior connections were compared to that of a control concrete counterpart to evaluate the benefits of using ECC in this type of application. The column load–displacement response (including the ultimate load and ultimate displacement), the (hysteresis) energy absorption capacity, and the cracking response were used as criteria in the comparison. The test results indicated that the use of ECC material in the connection plastic zone as a replacement of concrete and partial replacement of transverse (confinement) reinforcement can significantly enhance the joint shear resistance, energy absorption capacity, and cracking response, thereby, enhancing the joint seismic resistance and reducing reinforcement congestion and construction complexity.

Muthuswamy and Thirugnanam (2014) conducted an experiment with objective to identify the potential of hybrid fibre reinforced concrete (HFRC) as a ductile material which can be used for the construction of beam-column joints. One fifth scale model of an exterior beam-column joint made of conventional concrete and fibre reinforced concrete has been selected as test specimen for this experimental study. The specimens were subjected to cyclic loading. Such various parameters that load carrying capacity, load-deflection behaviour, ductility, energy absorption, stiffness and failure patterns of the joints have been studied. The experiment results of beam column joint with fibre reinforced concrete are compared with the beam-column joints which are made of conventional concrete. From the test results, it is found that the beam-column joint with HFRC results to 50% increase in ductility and 80% increase in energy absorption capacity in comparison with conventional concrete. HFRC in beam-column joint may be used to resist the earthquake in any seismic prone area.

Ganesh and Prabhavati (2015) carried out an investigation to study the effect of hybrid fibres on the strength and behaviour of high performance concrete beam column joints subjected to cyclic load. A total of 12 reinforced concrete beams column joints were casted and tested. Design mix of M25 was chosen which is widely used in residential. Crimped steel fibres and Glass fibres were used in hybrid form. From cube tests, the volume fraction of (i) crimped steel fibres 0.75% (ii) glass fibres viz. 0.33% gives best results. Addition of fibres in hybrid form improved many of the engineering properties such as the crack pattern, ultimate load and ductility factor of the composite. The combination of 0.75% volume fraction of steel fibres and 0.33% volume fraction of Glass fibres gave better performance with respect to energy dissipation capacity and stiffness degradation than the other combinations.

2.3.3 BEHAVIOUR OF RC BEAMS STRENGTHENED WITH ECC

Montesinos and Wight (2000) investigated the seismic behaviour of beams after increasing hoops spacing and evaluate the potential of High Performance Fibre-Reinforced Cement Composite (HPFRCC) materials as a replacement of joint transverse reinforcement. They also tested a 3/4-scale exterior beam column subassembly under large displacement reversals. The result indicated that the specimen with ECC material exhibited a large number of hairline diagonal cracks with little damage at the end of the test (5.0% drift). In terms of shear distortion response, it is clear that the ECC connection exhibited excellent performance during the test, even though no transverse steel reinforcement was used in the connection region.

Maalej et al. (2005) investigated the performance of FRP-strengthened RC beams incorporating engineered cementitious composites (ECC) as a ductile layer around the main flexural reinforcement (ECC layered beams) characteristics of engineered cementitious composites (ECC) subjected to dynamic tensile loading and high-velocity projectile impact and reported in this experiment. Hybrid-fiber ECC containing a combination of high-modulus steel fibres and relatively low modulus polyethylene fibres was adopted to achieve a desired balance between the ultimate strength and the strain capacity of the material required for impact- and blast-resistant structures. Dynamic uniaxial tensile tests at varying strain rates of $2310\text{--}6$ to 0.2 s^{-1} were carried out, and ECC was found to be able to provide much higher enhancement in tensile strength than plain concrete and still be able to maintain pronounced tensile strain-hardening behaviour. At higher rates of strain, ECC showed multiple cracking behaviour, similar to that observed from quasi-static tests, with tight crack width of about 0.1 mm. The results from high-velocity (300–750 m/s) impact tests demonstrated the potential of ECC in providing improved functionality (compared with concrete) as protective material in aspects such as increased shatter resistance with reduction in damage arising from scabbing, spalling, and energy absorption associated with distributed micro cracking.

Canbolat et al. (2005) on very short coupling beams ($l_n/h = 1.0$) demonstrated the potential for HPFRC to provide improved ductility over traditional concrete coupling beams despite simplified reinforcement detailing. These results are consistent with findings from tests of HPFRC beam-column connections and walls, which have indicated that HPFRC increases shear strength and confinement in elements subjected to load reversals. The coupling beam tests showed that HPFRC provides confinement to the concrete and diagonal reinforcement and actively resists shear stresses through large displacement reversals.

Yung Chih Wang (2007) studied reinforced concrete beam column junctions strengthened with Ultra high steel Fiber reinforced Concrete (UFC). It was concluded that UFC displayed excellent performance in terms of mechanical and durability behaviour. The test results showed that UFC replaced joint frame behaves very well in seismic resistance. The performance was found to be much better than the frame strengthened with RC jacketing as normally seen in the traditional retrofit schemes.

Kumar et al. (2010) studied the use of Slurry Infiltrated Fibrous CONcrete (SIFCON) as a substantial material in RC beams. It was reported that SIFCON can be used in places where structures need to be designed to resist impact loads. It was also concluded that with proper design the cross-section can be optimized by replacing certain portion by SIFCON.

Ooijevaar et al. (2010) applied vibration based damage identification (VBDI) method for experimentally investigating the dynamic response of an intact and a locally delaminated 16-layer unidirectional carbon fibre reinforced T-beam. A force–vibration set-up including a laser vibrometer system was employed to measure the dynamic behaviour of the T-beam. The Modal Strain Energy Damage Index algorithm was applied using the bending and torsion modes.

Akbarzadeh and Maghsoudi (2010) conducted an experiment to study the flexural behaviour and redistribution in moment of reinforced high strength concrete (RHSC) continuous beams strengthened with CFRP and GFRP sheets. Test results showed that with increasing the number of CFRP sheet layers, the ultimate strength increases, while the ductility, moment redistribution, and ultimate strain of CFRP sheet decrease. Also, by using the GFRP sheet in strengthening the continuous beam reduced loss in ductility and moment redistribution but it did not significantly increase ultimate strength of beam. The moment enhancement ratio of the strengthened continuous beams was significantly higher than the ultimate load enhancement ratio in the same beam. An analytical model for moment-curvature and load capacity are developed and used for the tested continuous beams in current and other similar studies. The stress-strain curves of concrete, steel and FRP were considered as integrity model. Good agreement between experiment and prediction values was achieved.

Lequesne et al (2013) studied the seismic behaviour of coupling beams and walls constructed with tensile strain-hardening, high-performance fiber-reinforced concrete (HPFRC) through tests of large-scale precast coupling beams and coupled walls. A precast coupling beam design was developed to speed up construction and minimize interference with wall reinforcement. Three isolated precast coupling beam specimens with a span-to-

depth ratio of 1.75 were tested under large displacement reversals. Test results indicated the use of HPFRC allowed a reduction of the reinforcement required to achieve a stable coupling beam response by providing confinement and contributing to beam shear strength. A concrete design shear stress capacity of $0.41\sqrt{f_c'}$, [MPa] ($5\sqrt{f_c'}$, [psi]), where f_c' was the compressive strength of the concrete, was found to be appropriate. In addition to the coupling beam tests, two four-storey coupled wall specimens with precast HPFRC and regular concrete coupling beams were tested under lateral displacement reversals. Besides allowing the evaluation of seismic behaviour of coupled walls with HPFRC coupling beams, the use of HPFRC in the plastic hinge regions of the walls as a means of relaxing transverse wall reinforcement was evaluated. The two coupled wall specimens exhibited drift capacities of at least 2.5%. The HPFRC coupling beams were more ductile and damage tolerant than the regular concrete beams as shown in Figure 2.3. The incorporation of an HPFRC material in the wall allowed the use of a concrete design shear stress capacity of $0.33\sqrt{f_c'}$, [MPa] ($4\sqrt{f_c'}$, [psi]) and a wider spacing of transverse reinforcement confining the wall boundary regions.



Figure 2.5. Reinforced concrete (a) and HPFRC (b) coupling of beams at completion of testing (Lequesne et al., 2013)

Mohammed et al (2016) conducted a study and drew conclusions like the flexural and shear failure modes for R-SC-ECC beams were comparable to failure modes of normal reinforced concrete beams. Predicting the ultimate momentum R-SC-ECC beams using EC2 models yielded very conservative results due to neglected tensile strength of SC-ECC in the computing of ultimate strengths. RILEM models were suitable for predicting ultimate strength of R-SC-ECC beams with small reinforcement ratios. However, as reinforcement ratio increased RILEM models yielded results that were more conservative. Finite element models using VecTor2 package had successfully predicted the structural behaviour of R-SC-ECC beams including failure modes, ultimate moments, failure loads and deflections. R-SC-

ECC beams exhibit ductility class medium to high. Therefore, concluded that ECC can be very suitable for regions with seismic activities as ECC has the ability to absorb energy.

Singh et al (2017) conducted an experiment to develop and demonstrate the effectiveness of precast engineered cementitious composite (ECC) plates for strengthening of masonry beams by bonding them on tension face as well as both on tension and compression faces like sandwich beam. Two types of bonding materials were used i.e., epoxy and cement mortar for bonding the ECC plates with masonry beam. All beams were tested under four-point loading systems. The results revealed that the application of precast ECC increased the strength and deformability of masonry beams and hence demonstrated its effectiveness as strengthening element for masonry structures.

Shang et al. (2019) reviewed that ECC characterizes tensile strain hardening and multiple cracking properties as well as strong interfacial bonding performance with substrate concrete which makes it a promising retrofitting material. A lot of researches have been conducted on reinforced concrete (RC) structures including beams, columns, beam–column joints and fire-damaged slabs strengthened with ECC material and an extensive collection of valuable conclusions were obtained. These strengthening systems usually combine ECC with FRP textiles or steel bars to form a composite strengthening layer. The review demonstrated that ECC strengthening can greatly improve the performance of RC structures.

2.3.4 BEHAVIOUR OF INFILL WALL STRENGTHENED WITH ECC

Singh and Kaushik (2001) studied behaviour of fiber reinforced concrete corners under opening bending moments. It was indicated that there was a noticeable gain in efficiency with increase in fiber volume fraction up to a certain limit beyond which there was a drop in mix workability and joint efficiencies.

Bischoff (2003) conducted an experiment on fiber-reinforced concrete used in structural applications requires characteristic material properties that can be easily incorporated into existing design procedures. This study investigated the post cracking response of reinforced concrete tension members made with both plain and steel fiber-reinforced concrete (SFRC). Loading was either monotonic or cyclic, and shrinkage effects were included in analysis of the member response. Tension-stiffening results were used to determine the average tensile response of concrete after cracking, and an expression was developed to predict this smeared behaviour as a material property for cracked SFRC, as well as to estimate crack spacing. Specimens with steel fibres exhibited increased tension stiffening and smaller crack spacing, which both contributed to a reduction in crack widths.

The post cracking tensile strength of fiber concrete at the cracks is the determining factor affecting behaviour and is a fundamental material property used to predict tension stiffening and crack behaviour for conventionally reinforced SFRC. The uniaxial strength of SFRC immediately after cracking governs serviceability behaviour, while the post cracking strength at larger deformations governs strength design and is responsible for tension stiffening after yielding of the reinforcement. Cyclic loading did not have a significant effect on either tension stiffening or crack width control for the specimens tested.

Li et al. (2019) conducted an experiment with two types of ECC materials troweled on masonry infill walls strengthened with expansion bolt and interfacial agent. The seismic performance of the unreinforced and the ECC reinforced masonry structures was evaluated by the reversed cyclic loading test. The results showed that the application of the ECC on the masonry infill walls can improve the ultimate bearing capacity (in plane), ductility, stiffness and accumulative energy dissipation of the structure. Compared with the masonry structure with one side reinforced by the ECC, the structure with both sides reinforced by the ECC could better improve the ultimate bearing capacity, stiffness and cumulative energy dissipation with little difference on the ductility improvement. Moreover, the expansion bolt and the interfacial agent used to improve the connection between the ECC and the masonry structure have the ability to greatly increase the mechanical properties of the structure under cyclic loading.

2.3.5 Closure

It has been established from the reviewed literature that Engineered cementitious composites (ECC, also known as - ECC Concrete), developed in the last few years, may contribute to safer, more durable, and sustainable concrete infra-structure that is cost-effective and constructed with conventional construction equipment. The ECC material is advantageous due to the increased tensile strain capacity and compressive toughness of the material relative to traditional materials. It can be concluded that ECC strengthening can be very helpful under different seismic zones for strengthening of RC structures. So there is a need to conduct experiment on strengthening of RC structures while keeping in view the structural health so this experiment has been planned and conducted.

2.4 DAMAGE DETECTION AND VIBRATION MONITORING OF RC STRUCTURES

Most damage indices available have been developed from research in the earthquake engineering community. During an earthquake, a structure is expected to experience inelastic response. Consequently, it is possible to measure factors such as ductility and dissipated

energy. A measure of damage or a damage index is required to make a rehabilitation decision. This information helps to interpret the stiffness changes the parameter estimation procedures identified. Various researchers worked on damage detection and vibration monitoring of RC structures and their work is discussed below under different sub headings:

2.4.1 DAMAGE DETECTION AND VIBRATION MONITORING OF RC FRAME

Jian et al. (2005) An experiment on a three-storey frame structure was conducted by to obtain vibration test data in 36 damage cases. Using four steel columns and three slabs, a shear type frame structure was fabricated with bolted connections. The structural model had acceleration sensors installed for each slab and a strain gauge on the bottom of column near ground level. They simulated column damage on different levels. An impact force was applied on the slab of the third layer to excite structural vibration. Thirty-seven sets of dynamic vibration response data were acquired by a dynamic test system for undamaged structure and three damaged structures. The structural natural frequencies were obtained by Fourier Transform from the first three natural frequencies. It was found that the natural frequencies changed little because of structural damage. A neural network based multi-sensor information fusion was proposed to achieve identification of damage occurrence, damage localization and damage quantification, respectively.

Dincal S. (2005) investigated the performance of an existing structural damage detection method (SDIM) where only experimentally-obtained measurement information can be used to calculate the frequency response function used to detect damage as shown in Figure 2.6. The development of a SDIM that accurately identified damage while processing measurements containing realistic noise levels and overcoming experimental modelling errors provided a robust method for identifying damage in the larger, more complex structures found in practice. The existing SDIM program used an advanced genetic algorithm, along with a 2-D finite element model of the structure, to identify the location and the severity of damage using the linear vibration information contained in FRF as response signatures. Datagen is a Matlab program that simulates the 3-D dynamic response of the four-storey, two-bay UBC test structure. The dynamic response of the structure can be obtained for a range of preset damage cases or for any user-defined damage case. Therefore, using the FRF measurements obtained from the UBC test structure allows for a more realistic evaluation of the performance of the SDIM provided by GaDamDet as the impact on performance of more realistic noise and model errors can be investigated. Previous studies evaluated the performance of the SDIM using only simulated FRF measurements obtained from a 2-D structural model. In addition, the disparity between the 2-D model used by the

SDIM used to identify damage and the measurements obtained from the 3-D test structure is analyzed. The results indicated that the SDIM is able to accurately detect structural damage to individually damage members or to within a damaged floor, with little false damage identified. The SDIM provided an easy to use, visual, and accurate algorithm and its performance compares favourably to performance of the various damage detection algorithms.

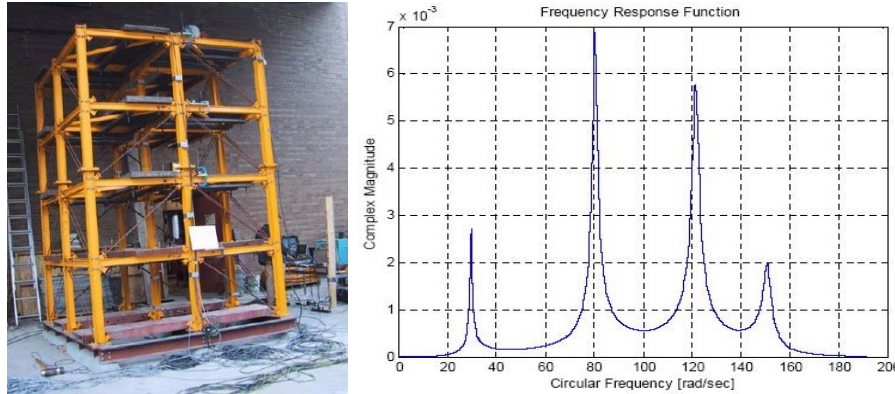


Figure 2.6 a) Experimental setup and b) frequency response function between force input and acceleration output (Dincal S., 2005)

Vimuttasoongviriya et al (2011) used VBDI to study the effect of lateral load on the damage indexes of RC frame strengthened using GFRP sheets. They reported that the use of GFRP for structural retrofitting increased the lateral load capacity and ductility of the RC frame significantly. Further, the damage indexes of retrofitted frame reduced indicating better performance as compared to control frame.

2.4.2 DAMAGE DETECTION AND VIBRATION MONITORING OF RC BEAMS

Ndambi et al. (2000) observed from RC beam tests that damping increased with excitation amplitude, and that modal damping ratios were highly influenced by non- linear effects and are therefore, highly subjective and difficult to estimate.

Formenti D. (2000) presented the sources of variations in FRF measurements on a beam. An FFT spectrum analyzer was used to simultaneously measure two FRF measurements, the excitation point and direction was in different directions. The results indicated that the test setup allowed for fairly repeatable measurements with the possible exception being the result for Mode 3 of the low force excitation. The peak amplitude of this mode had the largest variation of FRF. However, the rest of the results were very repeatable in terms of the two levels of excitation and the resulting peak FRF amplitude.

Maia et al. (2002) presented the subject of damage detection in structures. A series of numerical simulations on a simple beam were made in order to compare various damage detection methods based on mode shape changes. A generalization of these methods to the whole frequency ranges of measurement was proposed, i.e. methods based on mode shape changes become based on operational mode shapes. The objective of such a study was to ascertain the possibility of using various damage detection methods without the need for modal identification. They had developed some simple methods and tools based upon the use of FRF, which seem promising and have given good results in some practical applications. They also presented a new approach of FRF-based methods.

Wang et al. (2005) investigated the coupled bending and torsional vibration of a fiber-reinforced composite cantilever beam with an edge surface crack. They observed that the natural frequency shifts, along with observations on the mode shape changes due to surface crack and hence, can be used to detect both the crack location and its depth for on-line structural health monitoring.

Kwon and Elnashai (2006) found that values of coefficients of variations for material properties were generally lower than 15-20% and were often negligible compared to the randomness of both seismic input and quality control. In the past it had been extremely difficult to develop equations to predict the level of damping in real-life structures, and so designers were generally only able to conduct a dynamic analysis once the structure had been built using non-destructive testing. Despite the wealth of damping research undertaken in a huge variety of research areas, there remains to be found: (a) a consensus regarding basic damping levels in reinforced and pre-stressed concrete beams; (b) clarification of the mechanisms that affect in-service damping; and (c) an accurate prediction methodology to estimate real-life damping levels in concrete beams at any stage of their service life.

Zhu and Law (2007) developed a finite element based model for the damage assessment in reinforced concrete beams. The damage in the reinforced concrete beam occurred due to the formation of micro cracks and macro cracks. The macro crack caused debonding with large slip and the reduction in load transferring capacity of beam.

Baghiee et al. (2009) studied damage and CFRP strengthening of RC beams by vibration monitoring. They focused on the use of mode shapes and their derivatives. The Modal Assurance Criterion and Coordinate Modal Assurance Criterion factors were used to detect damage. These factors were derived from mode shapes and modal curvatures. They found that the modal assurance criterion was subjected to very small change by damage or

strengthening. The coordinate modal assurance criterion factors might detect the changes in beam stiffness at degree of freedom.

Xu et al. (2012) investigated different framed structures with different joint damage scenarios introduced by loosening the bolts connecting the beams and columns by using direct displacement measurement under base excitations. It was reported that the proposed time domain methodology could easily identify the variation of inter storey stiffness due to the joint damage with acceptable accuracy without any modal shapes and frequencies extracted from a dynamic test. The proposed approach provides an alternative way for damage detection of engineering structures by the direct use of structural dynamic displacement measurements.

Zhang and Xiong (2018) proposed a new sensing system combining BOFDA (Brillouin optical frequency-domain analysis) and FBG (Fiber Bragg Grating) technology, which were used to detect internal and surface cracks and their development in reinforced concrete structures, and an attempt was made to estimate the width of surface cracks. In these experiments, a special reinforced concrete beam structure was designed for crack detection under load. Four continuous distributed optical fibres were fixed on the steel skeleton, which was located within the reinforced concrete beam. Three FBG sensors were fixed on the lower surface of the beam, near its centre. By analysing the sensor data, it can be found that the BOFDA-distributed fiber can be used to detect internal cracking before surface cracking, and the difference between scans can be used to judge the time of onset of internal cracking, but the relative error in position is about 5%, while the FBG sensor can detect the cracking time of micro cracks on the lower surface in near-real-time and can be used to calculate the crack width. Through the experiment, it was found that if the combination of BOFDA and FBG technology is adopted, it can initially use the strain data obtained by multiple groups of BOFDA monitoring to predict the general location of the internal cracks, then to monitor the exact location of the surface cracks by FBG in the medium term, and to estimate the width of the final expansion of the cracks finally.

2.4.3 DAMAGE DETECTION AND VIBRATION MONITORING OF RC BUILDINGS

Boroschek and Yanez (2000) obtained strong earthquake motion and ambient vibration records of 22-storey high Chilean buildings. The data was used to determine damping ratios for the whole structure and to add to the data base used for the dynamic modelling of a Chilean building's response to seismic actions. A neural network based-approach for structural health monitoring was presented.

Kao (2003). The proposed approach involved two steps. The first step, system identification, uses neural network to identify the undamaged and damaged states of a structural system. The second step, structural damage detection, used neural networks to generate free vibration responses with the same initial condition or impulsive force. The inputs of the neural network were usually structural responses in time or frequency domain, or structural modal parameter (frequency, damping ratio, and mode shape), and the outputs were usually the damaged levels of members in the structure. He presented that change in structural properties (stiffness and damping) caused changes in periods and amplitudes of the free vibration of the structure system. Therefore, periods and amplitudes of the free vibration were useful indices to reflect changes of structural properties. The proposed approach made it easy to accurately generate a free vibration response of an unknown structure system using neural networks.

Ahmed et al. (2009) revealed that Vibration Based Damage Identification (VBDI) applied to structural health monitoring can be very useful in interpreting the global vibration response of a structure to identify local changes (e.g. damages) in it. But due to complicated features of real life structures there are some uncertainties involved in its key input parameters (e.g. measured frequencies and mode shape data) whereas output was highly sensitive to errors in modal parameters. They concluded that if vibration based methods are to be used for identification of damages they should be incorporated with semi analytical methods such as neural networks and statistical pattern recognition techniques for better accuracy which can result in structural health assessment.

Brehm et al. (2009) presented a mode selection strategy that enhanced the pure mathematical modal assurance criterion by physical information in terms of modal strain energies of the numerically obtained mode shapes. They found that the energy based modal assurance criterion led to a more reliable mode assignment. The modal assurance criterion approach indicated a sensitivity of bearings, which disappeared when the energy based criterion was applied. Failure mode control was significantly affected by material randomness, presence of non-structural components and quality control. Variations of mechanical properties depend on the construction material utilized.

Kanwar et al. (2016) studied dynamic characteristics of different RC buildings under different levels of damage using VBDI approach and it was used to develop a correlation between damage in the RC buildings with its known dynamic characteristics using artificial neural network models. However, this use of damage index had not been preferred to monitor damage in retrofitted structures in proper way because the cracks are covered by

strengthening material layers. Also the VBDI approach had not been used to monitoring response due to damages in brick in-filled RC framed structures repaired with composites.

2.4.4 Closure

However, most of the studies on reinforced structural elements have found load-drop behaviour and compound defects in the composite coating. In modernized structures, the location of the damage and the propagation of cracks under operating loads or natural hazards in general are not known because the cracks are covered with layers of reinforcing material. Therefore, to ensure safety it is necessary to monitor the structural state using dynamic features to define changes in material damage or geometric properties of matched structures. The aim of this research is to investigate this type of surveillance, including the nonlinear behaviour of the structures involved.

2.5 FINITE ELEMENT MODELLING OF RETROFITTED STRUCTURES

The finite element (FE) analysis is used to examine the effects of internal tensile reinforcement on the magnitude of principal tensile stresses in the critical region. The non-linear behaviour of FRP-strengthened beams is also examined in the FE analysis. FE related relevant literature is reviewed and discussed below:

2.5.1 FINITE ELEMENT MODELLING OF RETROFITTED RC FRAME

Kumari & Kwatra (2013) studied the non-linear response of RC frame using FE modelling under the incremental loading. Study was conducted under two phases. In the first phase of analysis experiment was conducted to study the response and load-carrying capacity of a four storied RC frame using non-linear finite element analysis. In the second phase of study an analysis model for retrofitted RC frame using finite element method using software ATENA was presented. The results obtained from FE analysis in both cases were compared with the experimental data for four storied RC frame tested by BARC at CPRI. The results from finite element analysis were calculated at the same location as used in experimental test. The accuracy of the finite element model was assessed by the comparison with experimental results which were to be in good agreement. The base shear/ displacement curves from finite element analysis agree well with the experimental results in linear range. The maximum difference in base shear was observed to be 7.9%.

Skafida et al. (2014) proposed a model which used a set of elements/springs to account separately for the compressive and shear behaviour of masonry. The comparison between experimental data and numerical results was conducted on the basis of two quantities: (i) the dimensionless index that is basically the difference in the base shear (at the

same displacement levels) divided by the maximum experimentally obtained base shear and (ii) the average error in the cumulative strain energy dissipated by the numerical model and the actual specimen. It was concluded that the model can adequately reproduce the load-displacement response exhibited by infilled RC frames. The medians of the positive and negative f-values, indicating overestimation and underestimation conditions, respectively, vary from 2.0% to 7.4%, proving a satisfying fit to the experimental data. The error in the dissipated strain energy was also acceptable, ranging from 5.2% to 14.9%.

Lin and Chen (2020) developed a structural health monitoring (SHM) system based on refined composite multi scale cross-sample entropy (RCMCSE) and an artificial neural network for monitoring structures under ambient vibrations. RCMCSE was applied to enhance the reliability of entropy estimations. First, RCMCSE was implemented to extract damage features, and finite element analysis software was then used to generate training samples, which included stiffness reductions to achieve various damage patterns. A neural network model was constructed and trained using entropy values for these damage patterns. An experiment was conducted on a seven-storey steel benchmark structure to validate the performance of the proposed system. Additionally, a confusion matrix was established to evaluate the performance of the proposed system. The results obtained for a scaled-down benchmark structure indicated that 89.8% of the floors were accurately classified, and 90% of the practical damaged floors were correctly diagnosed. The performance evaluation demonstrated that the proposed SHM system exhibited increased damage location accuracy (Figure.2.7).

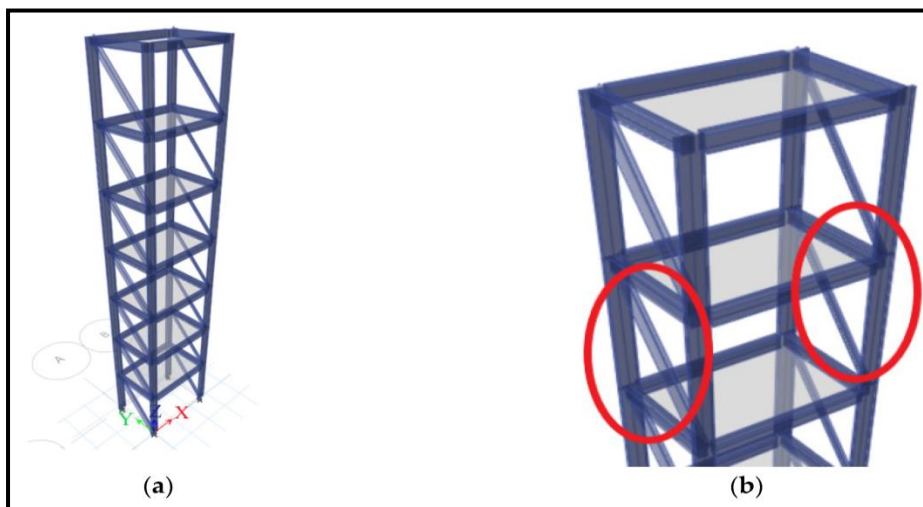


Fig. 2.7 (a) Finite Element Model (b) Damaged scenario, bracing was removed from red circle area (Lin and Chen, 2020)

2.5.2 FINITE ELEMENT MODELLING OF RETROFITTED RC BEAM

Kachlakev et al. (2001) investigated the structural behaviour of Horsetail Creek Bridge, with and without FRP laminates and established a methodology for applying computer modelling to RC beams and bridges strengthened with FRP laminates. They found that the general behaviours of the FE models represented by the load-deflection plots at mid-span show good agreement with the test data from the full-scale beam tests. However, the FE models showed slightly more stiffness than the test data in both the linear and nonlinear ranges. The effects of bond slip and micro-cracks occurring in the actual beams were excluded in the FE models, contributing to the higher stiffness of the FE models. The final loads from the FE analyses were lower than the ultimate loads from the experimental results by 5% - 24%. This was probably due in part to neglecting the inclined portions of the steel reinforcement; ignoring the effects of concrete toughening mechanisms and using assumed materials properties values instead of measured values. The crack patterns at the final loads from the finite element models correspond well with the observed failure modes of the experimental beams, and the crack pattern predicted by the FE analysis for the flexural strengthened beam agrees with hand calculations showing that the beam fails in flexure.

Francois et al (2002) revealed that non-linear FE analysis can predict correctly the flexural behaviour of CFRP bonded RC beams. This analysis was validated by comparison with test results. The model allowed simulating the global behaviour of pre-cracked and repaired beams. Results of the experimental study indicated that externally bonded CFRP plates can be efficiently used to strengthen or to repair RC beams. An increase in flexural strength was achieved for every beam and especially for the pre-cracked ones. Regarding the effect of the number of layers, it seems that the beam behaves as if the plate was thicker and no delamination was observed between both layers. Nevertheless, the possible brittle failure of the beams still needs to be considered.

Yang et al. (2003) conducted an experiment on FE modelling of concrete cover separation failure in FRP plated RC beams on the tension side of the RC beam. This study investigated the behaviours of an FRP plated RC beam using a discrete crack model based on FEA. Linear elastic fracture mechanics was used in this study and on-going research was being undertaken to extend this to use non-linear fracture mechanism. In this research, only half of the beam was modelled accounting for its symmetry. Four node quadrilateral isoparametric elements and three node constant strain elements were used to model the concrete, adhesive and CFRP plate. The internal steel reinforcements were modelled using two-node truss elements. The concrete cover on the tension face was modelled using four layers of

elements. Both the adhesive and the CFRP plate were modelled using one layer of elements. The results obtained from this study, in a preliminary study was successfully simulated the concrete cover separation failure mode in FRP strengthened RC beams. Initial numerical results confirmed that the bonding of a plate leads to smaller and more closely spaced cracks spaced cracks than the un-strengthened beam. For plated beam, the cracking had a significant effect on the stress distribution in the FRP plates. The stress distribution was uniform in the constant bending moment span only before major cracks are developed or close to the ultimate state. The length of the plate had a significant effect on the failure mode. The numerical example showed that if all other parameters remain unchanged, a beam strengthened with a short plate is more likely to fail due to concrete cover separation and in a more brittle manner.

Perera and Recuero (2004) studied the adherence analysis of FRP strengthened RC beams and discussed the effect of bonding between RC and CFRP when epoxy adhesive is used. They compared the analytical and experimental results which came out from the works can be done in this experiment. A test had been designed to characterize the behaviour of the adhesive connection between FRP and concrete; the test was based on the beam test, similar to the adherence test for steel reinforcement of concrete. In the same way, a numerical model based on finite elements has been developed to simulate the behaviour of RC members strengthened with FRP plates. The nonlinear response of the strengthened members was determined through the development of numerical material nonlinear constitutive models capable of simulating what happens experimentally. The results came out from this study was in the experimental tests, a local failure occurred mainly determined by high shear bond stresses transmitted to the concrete from the plates via adhesive. Debonding started at the mid span at the concrete blocks ends and propagated from there to the intermediate areas of the blocks. Therefore, it can be concluded that the shear bond stresses play a fundamental role in strengthening of RC beams with FRP plates. While in the analytical work, the model considered the position and increase of concrete cracks which has a very important influence on the overall response of the strengthened beam; it also affected the distribution of the stresses in the various locations of the member and the failure mechanism. In general, the model performed reasonably well in predicting the behaviour of the FRP strengthened beam.

Supaviriyakit et al. (2004) performed analytically the nonlinear FE analysis of RC beam strengthened with externally bonded FRP plates. The FE modelling of FRP strengthened beams was demonstrated in this experiment. The key for success of the analysis is the correct material models of concrete, steel and FRP. The concrete and reinforcing steel

are modelled together by 8-nodes 2-D iso-parametric plane stress RC element. The RC element considers the effect of cracks and reinforcing steel as being smeared over the entire element as shown in Figure 2.9. Concrete cracks and steel bars were treated in a smeared manner. The FRP plate was modelled by 2-D elastic element. The epoxy layer can also be modelled by 2-D elastic elements. Stress-strain properties of cracked concrete consist of tensile stress model normal to crack, compressive stress model parallel to crack and shear stress model tangential to crack. Stress-strain property of reinforcement is assumed to be elastic-hardening to account for the bond between concrete and steel bars. FRP is modelled as elastic-brittle material. The objective of the test was to investigate the effect of the bonded length on peeling mode of FRP. The results obtained from his analytical work that the finite element analysis can accurately predict the load deformation, load capacity and failure mode of the beam.

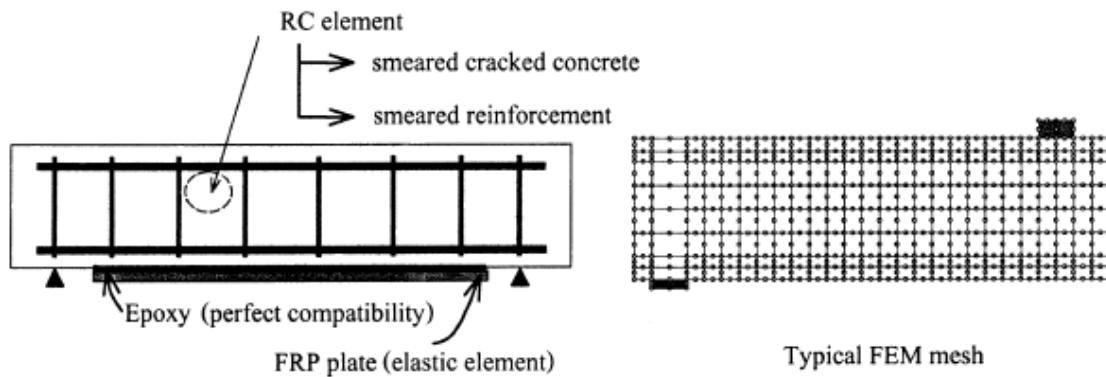


Figure 2.8 FE modelling and typical FE mesh (Supaviriyakit et al. 2004)

Pham and Al-Mahaidi(2005)conducted an experiment on non-linear finite element modelling of debonding failure of rectangular reinforced-concrete beams strengthened with externally-bonded Carbon Fiber Reinforced Polymer (CFRP) fabrics under bending. The FE model was able to simulate the beams behaviour. It predicted the ultimate capacity, the crack patterns as well as the CFRP strain distributions relatively well for all 16 cases. Three failure modes were observed as end debond, intermediate span debond and combination of both end and intermediate span debond. The parametric study was able to clarify the trends as the CFRP thickness, tensile reinforcement amount and CFRP bond length were varied. *Bindhu et al. (2010)* investigated the effect of cross inclined bars at the joint as confining reinforcement on the behaviour of exterior beam-column joint subjected to earthquake loading. Four specimens were casted. Group A comprises of 2 joint assemblages with joint detailing as per IS: 13920-1993 with 2 axial load cases and Group B comprises of 2 joint assemblages with cross inclined bars as confining reinforcements as per IS:13920-1993 with 2 axial load cases

same as that of Group A. Analytical study was carried out by ANSYS. The experimental results and analytical study indicate the additional cross bracing reinforcements improves the seismic performance.

Shangari & Elangovan (2016) analysed finite element reinforced concrete model by using ANSYS finite element program for both unstrengthen and RC beams. In analytical investigation six beams were analyzed using theoretical formulae. In experimental investigation six beams were casted and analyzed under two point loading system in a loading frame. The retrofiting is attained with the adhesion of GFRP sheet to the total volume of the beams. The ultimate failure load, deflection, crack pattern, percentage of increase in strength of GFRP strengthened RC beams were compared with those of their counterpart specimen of control group to evaluate the effectiveness of strengthening. From the numerical analytical and experimental investigation it was concluded that the ultimate load carrying capacity of RC beam wrapped with GFRP sheet was higher when compared to the unwrapped beams.

Shanmugavelu et al. (2016) presented the working of ANN model for predicting the performance characteristics of reinforced concrete beams strengthened with glass fibre reinforced polymers (GFRP) laminates. A total of seven beams of dimension 150mm x 250mm x 3000mm were casted and tested to study the effectiveness of GFRP on reinforced concrete beams. Beams were strengthened with CSMGFRP, WRGFRP and UDCGFRP laminates of 3mm and 5mm thickness. All the beams were subjected to four-point bending and tested upto failure. Deflections were measured through appropriate instrumentation at all stages of loading. The GFRP strengthened RC beams exhibit increased strength and composite action until failure. The proposed model provided satisfactory prediction of the performance parameters.

2.5.3 FINITE ELEMENT MODELLING OF RETROFITTED RC COLUMN

Mahini & Ronagh (2009) investigated the use of FRP web bonded strengthening system and the results showed that the method was effective and capable of restoring or even upgrading strength as shown in Figure 2.8. In addition, using the basic principles of equilibrium and compatibility, an analytical model was presented that simplifies the analysis and design of this strengthening scheme. Based on the model, a range of possible design graphs were presented in order to select the type and the amount of FRP required for the upgrading of an existing joint to a specified moment capacity and curvature ductility. The closeness of the graphs and the model proves that the graphs can be used by design engineers with confidence.

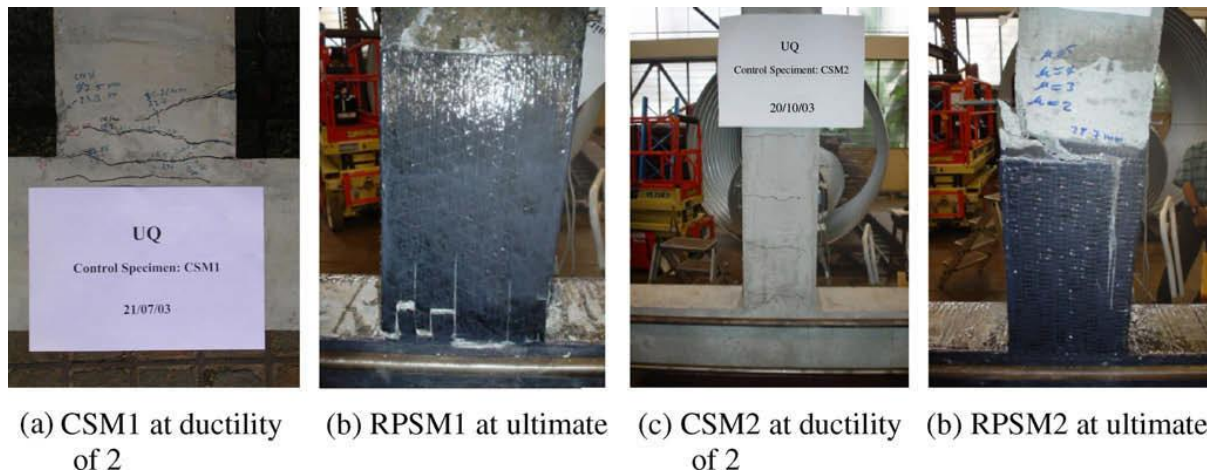


Figure 2.9 Retrofitting of column with FRP (Mahini and Ronagh, 2009)

Zadeh & Eshghi (2018) made an attempt to introduce a rational and comprehensive procedure for nonlinear FEA of FRP-strengthened slender RC columns with structural deficiency. Appropriate elements and constitutive material modelling from the software were chosen to account for the realistic behaviour of the columns. Modelling and analysis procedures were verified using experimental results. The columns were investigated in the conditions with and without FRP wrap and under different axial load levels. The developed FE models were capable of predicting the lateral load–deflection behaviour of both RC columns and FRP-strengthened RC columns. The failure modes predicted were the same as those experimentally observed. FEA performed in this study clearly demonstrated that the externally bonded GFRP sheet is a practical solution towards enhancing shear and flexural strengths, as well as ductility of slender RC columns whose plastic hinge zones are reinforced with insufficient stirrups. By increasing axial load, stress and strain in FRP sheets were increased. There was a stress concentration on the sheets at the column corners, but no failure or plastic deformation was detected, which is consistent with experimental observations. The wrapping GFRP sheet around the plastic hinge zone of the defective RC column provides not only enough shear strength, which results in a ductile flexural failure mode with the concept of strong shear and weak flexure. With the confinement of GFRP, a desirable ductile flexural failure mode rather than a brittle shear failure mode can be achieved. Similar to the experimental study, FEA revealed that wrapping only the plastic hinge can move the plastic hinge away from the bottom of the column. This was a critical consequence of this retrofitting technique for columns in the first floor of the building, as it did not lead to inhibiting the formation of plastic hinge, which is necessary for the mechanism in buildings during an earthquake.

2.5.4 FINITE ELEMENT MODELLING OF RETROFITTED RC BUILDING

Lord and Ventura (2002) obtained the modal parameters of a high rise building of a 48 storey building in Vancouver using ambient vibration. The experimental results and the analytical results from FE model were studied. ETABS was used to develop a detailed FE model of the building for modal updating purpose. The modal assurance criterion was used to correlate the results obtained from experimental results and FE results, and showed with confidence that the mode shapes found were the actual structural mode shapes of the structure. A good correlation was achieved between the ambient vibration data and the FE model.

Yuan et al (2004) investigated about external bonding of fiber reinforced polymer (FRP) composites as this technique has become very popular for strengthening concrete structures all over the world. The performance of the interface between FRP and concrete is one of the key factors affecting the behaviour of the strengthened structure, and has been widely studied using simple shear tests on FRP plate/sheet-to-concrete bonded joints. While a great deal of research is now available on the behaviour of these bonded joints, no closed-form analytical solution has been presented which is capable of predicting the entire debonding propagation process. This study presented such an analytical solution, in which the realistic bi-linear local bond-slip law was employed. Expressions for the interfacial shear stress distribution and load-displacement response are derived for different loading stages. It was also shown how experimental load-displacement responses of these joints can be used to quantify interfacial properties, including the interfacial fracture energy and parameters of the local bond-slip relationship. The debonding process is discussed in detail and the analytical results are compared with experimental data. Finally, results from the analytical solution are presented to illustrate how the bond length and the plate stiffness affect the behaviour of such bonded joints. While the emphasis of the study was on FRP-to-concrete joints, the analytical solution was equally applicable to similar joints between thin plates of other materials (e.g. steel and aluminium) and concrete.

Ferracuti and Savoia (2006) studied the numerical modelling for FRP concrete delamination. A nonlinear bond-slip model is presented for the study of delamination phenomenon. This research showed that FRP retrofit could be very useful to improve the behaviour of the RC structure under both short term and long term service loadings. A nonlinear interface law was adopted; the bonding between FRP plate and concrete is modelled by a nonlinear interface law. A nonlinear system of equations is then obtained via finite different method. Newton-Raphson algorithm, different control parameters can be

adopted in the various phases of delamination process (alternatively, force or displacements variables). Some numerical simulations are presented, concerning different delamination test setups and bond lengths. The numerical results of this study agree well with experimental data reported in the literature. A nonlinear shear stress-slip law is adopted for the method is used to solve the nonlinear system of governing equations. It was also shown that, adopting classical delamination test setup, snap-back behaviour in the load-displacement curve occurs for bonded plate lengths greater than the minimum anchorage length.

2.5.5 CLOSURE

Finite element analysis (FEA) is a computerized method for predicting how a product reacts to real-world forces, vibration, heat, fluid flow, and other physical effects. Finite element analysis shows whether a product will break, wear out, or work the way it was designed. Simplified, FEA is a numerical method used for the prediction of how a part or assembly will behave under given conditions. It is used as the basis for modern simulation software and helps engineers to find weak spots, areas of tension, etc. in their designs. So, in this research finite element modelling for analysis of the non-linear and dynamic behaviours of RC structures for both control and retrofitted frames using GFRP and CFRP sheet has been planned to do comparative analysis between FE model and experimental results.

2.6 GAPS IN THE RESEARCH AREA

Abundant experimental numerical FE and analytical studies have been reported on RC frame with and without masonry infill using analytical approaches. Most of the studies have focused on 2-D frames and performance of 3-D masonry infilled RC frames have not been studied in detail. Many strategies are available for reducing the seismic risk inherent in an existing building. But retrofitting of an infilled frame using a combination of GFRP and CFRP judiciously at critical points is not well studied. New approach to repair and strengthening of RC structures using Engineered Cementitious Composites (ECC) has also been reported on structural elements as well as frames. But repairing RC brick infilled frame with ECC along with replacing the yielded reinforcement has not been studied.

CHAPTER 3

EXPERIMENTAL PROGRAM

3.1 GENERAL

Structural Health Monitoring (SHM) and damage detection methods provide an opportunity to assess the structural integrity and safety of a structure by measuring its dynamic properties. The need for a structural health assessment is growing steadily to preserve existing civil structures.

Masonry infills are often used as internal partitions and exterior walls in buildings. They are generally treated as non-load bearing elements, and their interaction with the bounding frame is often ignored in the design. The performance of such structures during an earthquake has attracted much attention. Although the frame-fill interaction has sometimes led to undesirable structural performance, the filled frame in recent studies showed better performance in terms of stiffness, strength, and energy dissipation. Without bulkheads, the loss of a support column can lead to a partial or even complete progressive collapse of reinforced concrete frames. The presence of masonry walls can slow down the process and even prevent the development of a progressive collapse mechanism. In northern India, existing brick-infilled reinforced concrete (RC) buildings were built at a time when seismic areas were not recognized. These brick RC buildings have inadequate reinforcement details that result in poor lateral resilience and must increase their load capacity. These brick RC buildings are damaged by natural hazards such as earthquakes.

Retrofitting of these damaged structures is a better option than the option of reconstruction, which has not gained much importance in India. The use of FRP is becoming increasingly popular to remedy structural defects in RC buildings. The development of polymeric materials such as Carbon Fiber Reinforced Polymer (CFRP) and Glass Fiber Reinforced Polymer (GFRP) has resulted in more flexibility in material selection to increase strength. Reinforcement of the RC columns, which contains a high percentage of transverse reinforcement, uses FRP sheets to prevent its brittle shear failure and to significantly improve the ductility of the displacement and the ability to dissipate energy (Kim, 1995). The use of GFRP with epoxy resin and CFRP has increased in recent decades due to its excellent material properties such as light weight, high strength, corrosion resistance, low labor costs, and ease of field handling (Goyal, 2007, Memon and Sheikh, 2005, Mukherjee and Joshi, 2005). When repairing and modernizing the damaged structure, it is imperative in the

severely affected region to replace concrete and reinforcement with new materials. The replacement can be done, but the type of concrete remains the same and unchanged. The use of fibers in concrete provides better resistance to cracking and growth. The presence of randomly distributed discontinuous fibers in the concrete acts as secondary reinforcement and limits the formation of early stage cracks and the growth of cracks at the beginning. The fiber acts as a bridge between the cracks in the transfer forces and provides better resistance to the applied load and improves ductility.

Engineered Cementitious Composite (ECC) is one of the types that provide a more durable and sustainable concrete infrastructure that is cost effective and can be built using traditional construction equipment. The presence of microfibers and the absence of coarse aggregates act as a secondary reinforcement and close the cracks more effectively than conventional FRC and provide strain hardening performance. The ECC is inherently ductile and provides a very high bending curvature rather than a conventional brittle failure. The use of high ductility concrete in the potential hinge area of the RC structure improves the seismic response of the structure. Damage detection by measuring vibrations of dynamic properties such as natural frequency, mode shape and frequency response is a non destructive technique and is often used to control structural damage. The focus is mainly on structural elements or compounds that withstand maximum deflection.

Damage detection using vibration measurement of dynamic properties such as natural frequency, mode shape and frequency response function is non destructive technique and it is widely used to monitor the damage of structure. Attention is mainly focused on the structural elements or connections which resist high stresses (Estekanchi and Arjomandi, 2007; Mo, 1994). The vibrations induced may be monitored by comparing the frequency of the structure at different time intervals. The vibration data is generated in time as well as frequency domain. Vibration energy is distributed over a range of frequencies consisting of the fundamental frequency and combinations of the harmonics. With this technique, the defective component is identified by resolving the vibration signal into its constituent frequencies and relating these to the known discrete frequency of the component. In this way the component's signature or thumbprint is established (Rao, 2000; Reddy et al., 1994).

Taking into account the above reasons, RC 1/3 model has been constructed in the laboratory and tested under controlled conditions to assess their damage status. The experimental program has been developed to investigate the change in vibration behaviour

and study of load displacement behaviour for control, FRP retrofitted and ECC strengthened frames. As damage increased, vibration responses due to impact excitation has been measured. In the laboratory, a non-ductile RC frame model has been constructed. The material system, the details of the samples and the test procedure are listed below.

3.2 Material Properties

RC frame considered in present study have been casted in the laboratory. The material consists of concrete, steel reinforcements, Bricks, GFRP, CFRP sheets, polypropylene fibres, silica fumes, fly ash. Relevant tests in accordance with the Indian Standard (IS) codes of practice were conducted to determine the physical properties of the materials used in the study (IS-8112, 1989; IS-1489,(Part 1): 1976; IS-383, 1970). The details of the materials along with their properties have been presented in the subsequent sections.

3.2.1 Cement

Cement is very often the most important because it is usually the most delicate link in the chain. The function of cement is to combine all materials such as sand and stone and also fill the gap between sand and stone particles to form a compact mass. The cement used in this study is ordinary Portland cement (OPC) according to IS: 8112-1989, tested with consistency 31. 5% and cement specific gravity was 3.14. The initial and final setting time of the cement was 72 minutes and 640 minutes, respectively. The corresponding compressive strength of the cement paste was 33.5 MPa (7 days) and 44 MPa (28 days).

3.2.2 Aggregates

Aggregates are inert granular materials such as sand or gravel which together with water and Portland cement are an integral part of concrete. The importance of using the right type and the right quality of aggregates cannot be overemphasized. Fine and coarse aggregates typically account for 60% to 75% of the concrete volume and strongly affect the properties, proportions and economy of the freshly cured concrete mix. The coarse aggregate is mainly used to add volume to the stable and uniform concrete mix. The fine aggregate also helps the cement paste to keep the coarse aggregate particles in suspension (IS-383, 1970). This effect promotes the plasticity of the concrete mix and prevents the separation of paste and coarse aggregate during transport. The properties of these particles together greatly affect the performance of the concrete.

a) Fine Aggregate

IS: 383-1970 defines the fine aggregate as the one that passes through the 4.75 mm IS sieve. The fine Grade II aggregate has been used in the present study and free from silt with a specific gravity of 2.61 (dry) and a water absorption of 1.45.

b) Coarse Aggregate

IS: 383-1970 defines that the aggregates retained in a 4.75 mm IS sieve are referred to as coarse aggregates. To increase the density of the resulting concrete mix, the coarse aggregate often consists of two or more sizes. Two types of aggregates of different sizes has been used in the present study. The coarse aggregates used were dried and free from any silt with a specific gravity of 2.59 (dry) and a water absorption of 2.98.

3.2.3 Concrete

The concrete mix M20 was designed according to the recommendations of the Indian standard for the design of concrete mixtures (IS 1489, 1982). A 1: 1.6: 3.2 mixture with 0.52% water cement ratio was used. The 28-days compressive strength of concrete was 24.1 MPa for first floor, 23.4 MPa for second floor and 22.4 MPa for upper floor. The average compressive strength of the frame has been found to be 22.5 MPa.

3.2.4 Reinforcement

The longitudinal main reinforcements consisted of 8 mm and 10 mm diameter bars and 6 mm diameter round bars. These rebars met the minimum amount of steel required by the Indian standard (IS: 456-2000) for the RC structure. The steel bars showed a minimum elastic limit of 415 MPa and a modulus of elasticity of 200 GPa (Fe 415). Table 3.1 shows the test results for the rebar used.

Table 3.1: Tensile strength of steel bars.

Steel Reinforcement Bars		
Steel size	Yield strength (MPa)	Ultimate strength (MPa)
Longitudinal bar (10 mm-diameter)	490.2	586.6
Longitudinal bar (8 mm-diameter)	523.4	628.9
Transverse bar (6 mm-diameter)	517.9	636.4

3.2.5 Bricks

Bricks are most commonly used to fill in the frame field in the RCC partition frame. First-class bricks are used for filling the frame panel with cement mortar in the ratio 1: 4. The bricks used in the test were tested in the laboratory and meet all specifications according to IS: 1077-1992 with a water absorption of 17% and a compressive strength of 14.3 MPa.

3.2.6 Casting of Specimen

One specimen of RC frame has been casted with a steel reinforcement. During casting of the frame sample, three cubes (150 mm in size) were casted for each storey. The cubes were tested 28 days after the curing. On the specific test day, these cubes were tested under compression after curing to determine the average compressive strength of the concrete.

3.2.7 Fiber reinforced polymer sheet

Carbon and glass fiber is the most commonly used additive in reinforced polymer sheets, commonly called CFRP and GFRP. These sheets lower reduces creep, particularly at higher temperatures. Glass fibers can vary in length and have a general diameter of 10 to 15 microns with white colour. GFRP used have Fiber G-Sheet EU 900 model. This material is wrapped with epoxy resin to provide a range of high performance features. Fiber reinforcement systems contain bidirectional glass fibers as shown in Figure 3.1. Similarly, the CFRP used is a bidirectional sheet as shown in Figure 3.2. The mechanical strength of the resin was ignored. Table 3.2 contains the details of GFRP and CFRP.

Table 3.2 Properties of GFRP and CFRP laminates.

Property	GFRP	CFRP
Fiber	Glass fiber sheet	Carbon fiber sheet
Thickness	0.34 mm	1.2 mm
Density	2.6 g/cm ³	1.8 g/cm ³
Tensile strength	1.59 GPa	3.00GPa
Elastic modulus	63 GPa	228GPa



Figure 3.1 Glass Fiber sheet (GFRP)



Figure 3.2 Carbon Fiber sheet (CFRP)

3.2.8 Composite (FRP) strengthening system and application

The composite strengthening resin system consists of the following components:

a) Crack injector 1315

Crack Injector 1315 (M Brace) is a low viscosity epoxy resin consisting of a crack injection resin system based on trimethylhexane-1,6-diamine. It has a long mixed life designed to seal cracks in the concrete and restore its structural integrity. Due to its low viscosity, it can easily be injected into fine cracks to bond between damaged joints. It can restore the structural integrity of concrete due to the high strength of the concrete joint. This reinforcement system consists of two parts: resin and hardener. Both parts are shown in Figure 3.3. The properties of the crack injector are given in Table 3.3



Figure 3.3 Crack Injector 1315

b) Epoxy Aggregate

Epoxy aggregates (M Brace) is the concrete smoothing material used in the composite reinforcement system available in bags. It is 100% solid epoxy resin material. The composition, which contains the content of free silica (quartz), is available in natural mineral

deposits of rock or sand and gravel. Recommended use includes sealing surfaces prior. to epoxy injection, bonding rigid materials, and leveling irregular surfaces. This composite system consists of three parts: resin, hardener Figure 3.4 and aggregate Figure 3.5. Mix only as much putty as can be used within its lifetime. Apply the trowel adhesive to the substrate with a steel trowel or other suitable device to remove surface defects. Application thickness and coverage rate are highly dependent on the nature and profile of the concrete substrate. The prepared and smoothed surfaces must be coated within two days to make sure sufficient adhesion of the fiber strengthening system to the substrate.



Figure 3.4 Putty and hardener

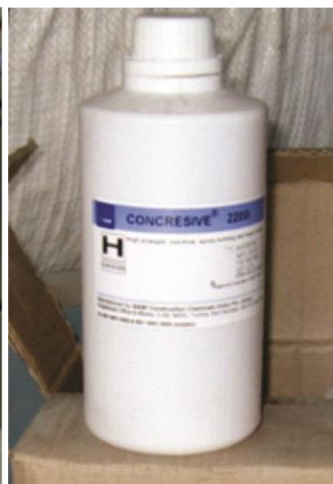


Figure 3.5 aggregate

Table 3.3:Physical Properties of Epoxies

	Crack injector 1315	Epoxy 2200
Viscosity	Low	High
Compressive Strength:		
1 day	65MPa	40MPa
7 day	70MPa	65MPa
Flexural strength (7 day)	55MPa	20MPa
Tensile strength (7 day)	18MPa	10MPa
Bond strength	>2.5MPa (Concrete failure)	>2.0MPa (Concrete failure)
Pot life	50 Min.(25°C) 30 Min.(40°C)	50 Min.(25°C) 30 Min.(40°C)

c) Epoxy 2200

Epoxy 2200 (M Brace) is the adhesive used to bond concrete with bisphenol-A epoxy (epi-chloro-hydrin) resin. It is highly viscous. Epoxy resin is capable of repair in the presence of moisture with temperature up to 2°C When applied to sound concrete. The Primer gives high tensile bond strength to the M Brace composite strengthening system. To be effective, the workplace must be well ventilated. The primer should not be used if the ambient temperature is below 5°C. Only the amount of primer that can be used within its useful life should be mixed. The mixed primer should be pasted with a roller brush. If necessary, a second layer can be applied if the surface is very permeable. Figure 3.6 and Figure 3.7 shows both parts (saturant and hardener).



Figure 3.6 Saturant part of epoxy



Figure 3.7 Hardener part of epoxy

d) Saturant

Saturation resin (M Brace) semi solid material that allows a lightweight sheet to adhere to the composite strengthening system is shown in Figure 3.8. When the saturant resin is cured with the top sheet, a high performance composite system is created for use in external structural repairs to improve applications. The fiber reinforcement should be cut to the prescribed size before applying. The number of cut sheets is limited to those that can be used in one day.

Apply the first layer of fully mixed saturated resin to the fiber reinforcement surface with a roller brush. The saturated resin reinforcement should found with the fiber side down on the concrete surface to which the primer was applied. The fiber reinforcement must be squeezed two or three times in the fiber longitudinal direction with a deformation roller and a

rubber spatula to impregnate the resin in the fibers and to deform in the layer. For joining fiber strips, an overlap length of 15 cm in the horizontal direction and an overlap length of 2 cm in the vertical direction is required. Additional resin should be applied at the overlap location on top of the outer layer of the fibrous layer which overlaps.

Table 3.4: ECC Mix Proportion and Properties of material

ECC Mix Ratio	
Cement : Sand : Fly Ash : Silica Fumes	1: 0.5: 0.2 : 0.1
Water Cement Ratio	0.45
Plasticizer (SP-1) (percentage of cement content by weight)	0.75%
Polypropylene Fibres	2.5%
Properties of Fly ash	
Colour	Whitish Grey
Bulk Density	1490 Kg/m ³
Specific gravity	2.3
Properties of Silica Fumes	
Colour	Dark Grey
Specific gravity	2.25
Mean grain size (µm)	0.15
Bulk Density	2.25 mg/m ³
Properties of poly-propylene fibres	
Admixture SP-1	
Colour	Dark brown liquid
Specific gravity	1.160 at 25°C
Chloride content	chloride-free
Freezing point	0° C

The composite fiber reinforcement system should rest for at least 30 minutes. Any lifting or shifting that occurs during this time should be corrected with the roller and the spatula. The second layer or multiple mixed saturant layers should be applied to the surface

of the fibrous layer. The coated surface must be firmly pressed in the fiber longitudinal direction two or three times with the roller and the spatula in order to impregnate the fiber layer in the same manner as before.

Concrete blasting or shot blasting for removing of grout and surface contaminants is recommended . Surface defects in the concrete are repaired with putty. Depressions on the concrete surface or a concrete joint, are filled with putty. Sharp corners should be smoothed with a radius of at least 10 mm. Interior angle is made of rounded concrete with putty.

3.2.9 Strengthening of RC frame with ECC

The control frame model has been tested until failure under cyclic load, than the damaged frame was repaired, retrofitted with GFRP / CFRP and tested again under cyclic load until collapse. In the column of the ground floor and in the area of the beam- connection of the first floor, considerable damage was found due to a failure of the soft storey. The damaged building was kept in the vertical position with sufficient support to ensure perfect vertical and is considered as zero position. Damaged concrete in the ground floor column and beam-column joint of the first-floor has been completely removed and the reinforcement in the hinge area has been replaced by new bars of reinforcement. In a small cracked area, the epoxy was injected instead of replacing the concrete to have a better bond with the cracked surface. In severely damaged areas, ECC was used and epoxy 2200 applied to the existing concrete surface for better adhesion to the old concrete surface. The SP-1 plasticizer is shown in Figure 3.8 and the polypropylene fibers shown in Figure 3.9 were used in ECC.

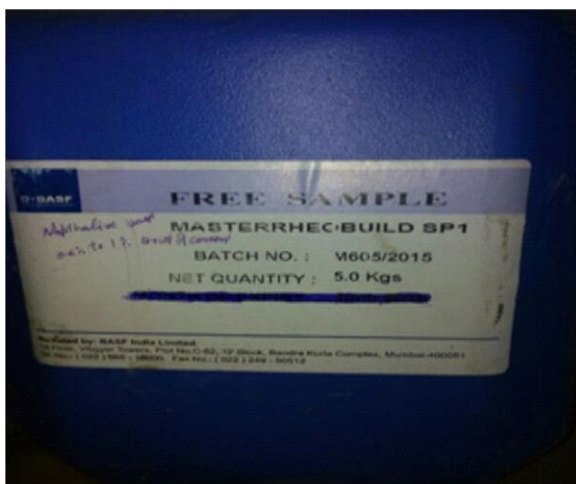


Figure 3.8 Plasticizer SP-1



Figure 3.9 Polypropylene fibre

3.3 DETAILS OF THE TEST STRUCTURES

The frame was designed and casted. The frame consisted of a third scale of a three-story RC frame. Each column is the same size with a height of 950 mm floor to floor.

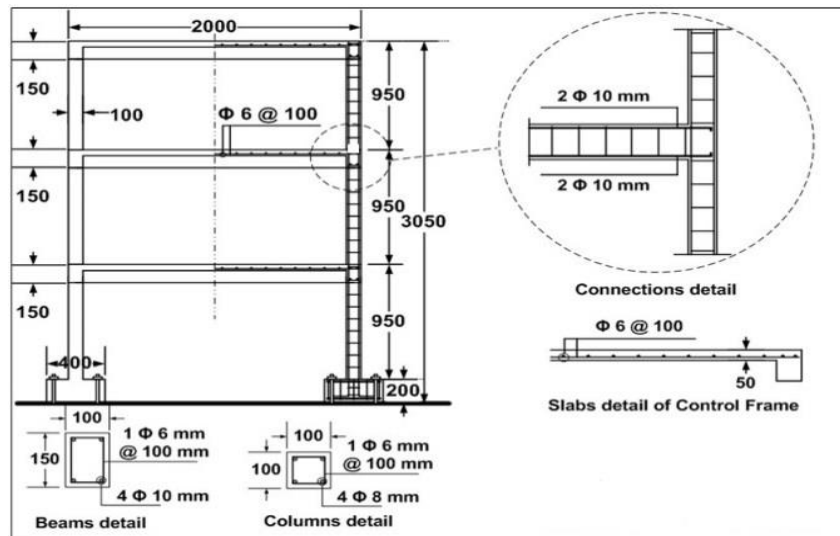


Figure 3.10. Reinforcement details of RC frame

All the beams have been equally sized with rectangular cross-sections of 100 mm x 150 mm and a length of 2000 mm. The overall dimension of the frame was 2000 mm x 2000 mm x 3050 mm (Kanwar et al., 2006). Specimen was designed to withstand vertical loading in accordance of Indian Standard IS 456: 2000. Each column was the same size with a square cross section of 100 mm x 100 mm (with four 8 mm diameter longitudinal bars with 6 mm diameter transverse ties spaced at 100 mm centre to centre). All the beams were rectangular with a cross-section of 100 mm x 150 mm (with two 10 mm longitudinal bars on the tensile and compressive surface with 6 mm diameter transverse ties spaced at 100 mm centre to centre). The schematic drawing of the model structure and the steel reinforcement of the frame is shown in Figure 3.10. The ground floor of the frame remained a soft storey. The first and second floors of the frame were filled with bricks as shown in Figure 3.11.

3.3.1 Supports

Each column was casted integrally with a 150 mm x 200 mm x 400 mm stub base. The column and stub base reinforcement shown in Figure 3.12 and the beam reinforcement shown in Figure 3.13. To avoid support movement, the stub was in turn bolted firmly using a total of four bolts of high tensile strength plate which rest floor and all the stub were connected to a beam size of 150 x 100 mm as shown in Figure 3.14 (Memon and Sheikh, 2005; Ye et al., 2002).

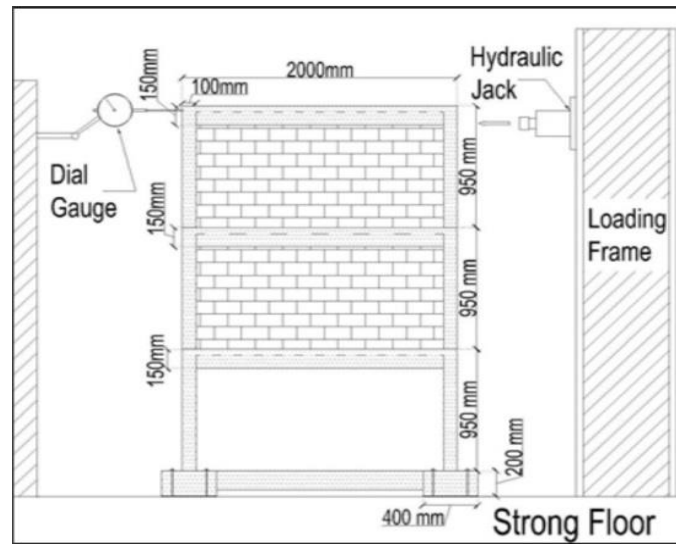


Figure 3.11 Lateral load resistant test of frame

3.3.2 FRP retrofitting scheme

It can be seen that existing concrete structures have unsatisfactory performance for a variety of reasons. This can be manifested in a low performance under operating load in the form of excessive deformations and cracks or in an insufficient end resistance. In addition, revision in the design of structure and load codes may result in many structures, previously considered satisfactory failing to comply with applicable regulations. In the current economic climate, rehabilitation of damaged concrete structures in compliance with the strictest operating limits and ultimate strength of existing regulations, as well as reinforcement of existing concrete structures for transporting higher allowable loads, appears to be a most attractive alternative to demolition and reconstruction. In this study, the damaged frame was retrofitted with CFRP and GRP plates. CFRP was used to adjust the column near the foundations, and GRP was used to adjust the joints of the support column.

To investigate the behaviour of CFRP and GFRP under the loads and the effectiveness of the different binders with which the properties of the FRP are bonded to the concrete, standard prisms of $100 \times 100 \times 500$ mm are used Figure 3.15 with the same configuration frame. First, two control prisms and two samples repaired with GFRP and CFRP in the stress zone were tested under two points loading as shown in Figure 3.16 and Figure 3.17.



Figure 3.12

Column detail with stub



Figure 3.13

Beam detail of frame



Figure 3.14

Stub foundation



Figure 3.15 Control Prism



Figure 3.16 GFRP Retrofitted Prisms



Figure 3.17 CFRP Retrofitted Prisms



Figure 3.18. Reconnected prism using epoxy

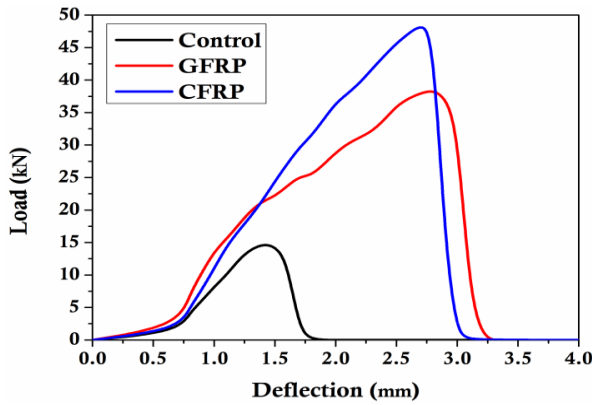


Figure 3.19 Load -deflection behaviour of Control and FRP repaired prisms

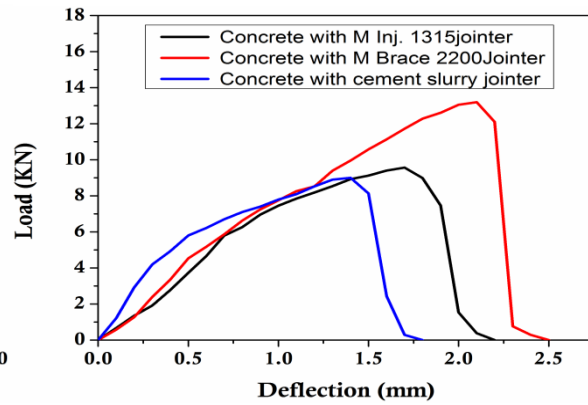


Figure 3.20 Flexural strength of reconnected prisms

To investigate the ability of epoxy bonding to fill cracks, another six prisms made of control concrete were tested under two point loads. Broken prisms were reconnected using cement mortar, epoxy 2200 and crack Injector 1315 epoxy (M Inj1315) (2 samples each). The reconnected samples were allowed to cure for a week and tested at two point loading until failure, as shown in Figure 3.18.

Flexural behaviour of prisms shown in Figure 3.19, specimens tested under two load points that the FRP enhanced samples a drastic improvement in the deformation properties of the strengthened prisms in comparison to control prisms. In particular, the sample repaired with higher CFRP at the bottom has better load capacity and the occurrence of cracks in the un-strengthened part shows the effectiveness of the bonding. The GFRP-repaired sample is split in half in the middle of the section, and the experiment shows how important a higher-modulus FRP is for resilience.

The load deflection behaviour of the reconnected samples is shown in Figure 3.20. The cement suspension sample has the lowest performance compared to other reconnected samples. It shows that the epoxy injection acts as a better binder to make contact between cracked surfaces. The Assembler M Brace 2200 shows the best results and is used to retrofit RC frames with FRP.

The FRP applications in the damaged frame were performed the steps (Mukherjee and Joshi, 2005):

- 1) The damaged frame was returned to the original position.
- 2) Loose or damaged concrete was removed and surfaces were cleaned.

- 3) All corners of the damaged members were beveled and rounded, and M Brace 2200 putty along with silica aggregate filled in all cracks so that the crack injector does not flow out from the cracks as shown in the figure 3.21.
- 4) In the frame, the cracks of the damaged frame were filled with the composite resin of the injector M Brace 1315 as shown in Figure 3.22, after which the surface was finished again with M Brace Putty as shown in Figure 3.23.
- 5) After flattening the surface, the GFRP sheets were wrapped with M Brace Saturant in the beam-column joints shown in Figure 3.24 and the CFRP sheet on column hinge as shown in Figure 3.25.

The application of GFRP to the beam- column joint and the CFRP sheathing to the column hinge was performed on the damaged element as shown in Figure 3.26. FRP panels were mounted in several steps Figure 3.27.

- 1) One layer of FRP sheet has been used on the tension and compression direction zones of the columns and beams to increase the flexural strength capacity.
- 2) Columns and beams were confined at each edge zone by being wrapped with a layer of transverse directional FRP sheet.
- 3) The connection joints were wrapped with two layers of FRP. The first layer was along the direction of the column and another layer was perpendicular to the column direction.



Figure 3.21 Void filling with M Brace putty



Figure 3.22 Injecting Master injector

1315



Figure 3.23 Finished surface



Figure 3.24 GFRP with M Brace epoxy



Figure 3.25 CFRP with M Brace epoxy



Figure 3.26 Final Retrofitted Frame

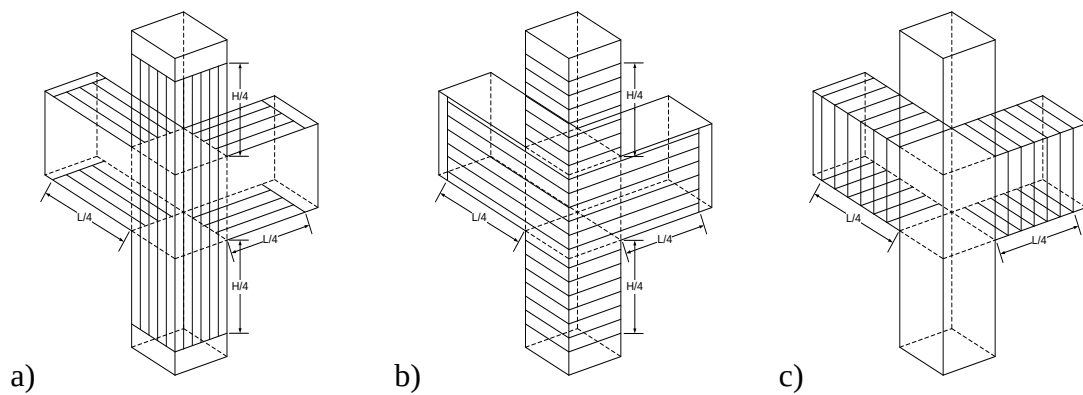


Figure 3.27 Application of FRP sheets: a) first step b) second step and c) final step.

3.3.3 Selection and performance of epoxy for repair

When repairing the RC frame after cyclic loading, the ECC mix is used to retrofit severely damaged locations, whereas minor cracks are injected with epoxy resin to repair it.

While selecting the epoxies for retrofitting, the effectiveness of the bonding properties of different types of epoxies with concrete and ECC should be studied in detail. Three bonding options were tested: Cement Mortar, Crack Injector 1315, and Epoxy 2200. Standard 100mm x 100mm x 500mm prism samples were molded using the same mixing ratios used in the RC frame. They were then exposed to two load points and split in half, as shown in Figure 3.16. After failure, the fractured parts of the prisms were reconnected to conventional concrete using cement mortar in a series of substrates. Two other sets of prisms were repaired by injecting two different epoxy resins and allowed to cure for one week. In each set, two beam samples were analyzed. In addition, a portion of the broken prism samples with 3 different ECC configurations was re-cast to evaluate the bonding behaviour between composite and concrete surface, as shown in Figure 3.28 , All prisms were re-tested under flexural load until the resistance of the epoxy bond to crack filling was not examined as shown in Figure 3.29 and Figure 3.30.

The load-deflection behaviour of all reconnected samples shows the importance of the binding materials. RC slurry samples repaired with cement slurry have the lowest performance compared to samples that have been recombined with epoxy resin. The sample with conventional concrete and epoxy resin Epoxy 2000 achieved the best performance with greater load capacity and better formability. Epoxy bonded samples show a different peak response when the epoxy is applied or injected onto the surface. Epoxy injection acts as a better binder to make contact between cracked surfaces. Figure 3.31 shows the comparison of the load deflection behaviour of the reconnected ECC samples, in which the portion of the fractured sample is recombined with new conventional concrete containing ECC using two different epoxies.

It shows the best performance in terms of loading and deformation properties using epoxy 2200 with polypropylene fibers as the epoxy injector. In addition, the initial stiffness of the reconnected samples differs from the original samples. The cement mortar sample registered the lowest performance and failure occurred exactly where the reconnection took place while in the epoxy-bonded samples the error concentration was noted slightly outside the joint area. For this reason, it was decided to replace the damaged concrete with ECC concrete with polypropylene fibers, using the epoxy 2200 epoxy resin as an adhesive.



Figure 3.28 Casting of ECC with old prism



Figure 3.29 Testing of reconnected prisms



Figure 3.30 Failed Prisms

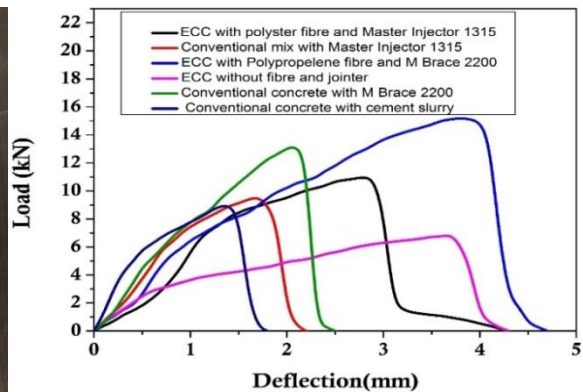


Figure 3.31: Flexural strength of reconnected prisms

3.3.4 Strengthening with ECC

Concrete that was damaged by cyclic loading in the RC frame after failure was removed by supporting the frame with lifters, as shown in Figure 3.32. The damaged and old reinforcement was replaced because it had lost resistance due to several deformation cycles. Before the filling of ECC mix in removed concrete area, the Epoxy 2200 jointer was applied to the surface of the old foundation and joint concrete Figure 3.33 to create a better bond between old and new concrete than ECC concrete were prepared as shown in Figure 3.34 and Figure 3.35. Bricks were filled back into the frame as shown in Figure 3.36 to ECC re-strengthened frame as shown in Figure 3.37.



Figure 3.32 Damaged frame



Figure 3.33 Epoxy applied at joints



Figure 3.34 ECC casting



Figure 3.35 Frame casting



Figure 3.36 Brick infill after casting



Figure 3.37 ECC Restrengthened Frame

3.4 TEST EQUIPMENT

The equipment used to perform various tests on the materials and on the frame included:

3.4.1 Testing Machines

An automatic compression testing machine with a capacity of 3000 kN was used to test concrete cubes of the frame model. Tensile strength of steel bars was determined.

3.4.2 Loading Frame

Deflection controlled quasi static load has been applied at top beam through 200kN capacity hydraulic jack mounted in a rigid steel frame. Load has been applied in step of required deformation for each step and released after required deflection for a particular step. The foundation of the load frame is fixed on the strong RC floor to withstand vertical and horizontal forces. The hydraulic cylinder was configured at a height of 2975 mm above the ground to apply a lateral load to the centre of beam floor of the frame models (Goyal, 2007, Kanwar et al., 2006). The arrangement of the load path test is shown in Figure 3.38. The deflection was recorded with LVDT mounted on each floor to an accuracy of 0.01 mm. Deflection of reaction wall was not measured as it was considered as fixed for small deformation.



Figure 3.38 loading jack at top floor

3.4.3 Impact Hammer

The vibration response was measured by performing impact tests on the structure. The impact stimulus was applied 150 mm from the midsection. Impact Hammer with a hard rubber tip was balanced from a horizontal distance of about 50 mm from the frame (Goyal,

2007, Kanwar et al., 2006). The model of the hammer is PCB 086D50 having sensitivity of 0.23 mV / N, manufactured by OROS-35 Instruments. A typical hammer like the one used is Figure 3.39 and Figure 3.40. The impact consisted of a nearly constant force over a wide frequency range and was able to excite all the resonances in this area. The size of the hammer, the length and the material at the time of impact determined the amplitude and frequency content of the force impulse. The impact tip material generally determines the frequency content. The mass and velocity of the hammer determined the energy content. A series of impact tips made of different materials and exchangeable masses could be selected to expand the frequency ranges and force levels and to test different structures. The types and sensitivities of the power hammer and the accelerometer can be selected and entered in the measurement configuration menu of the recorder.

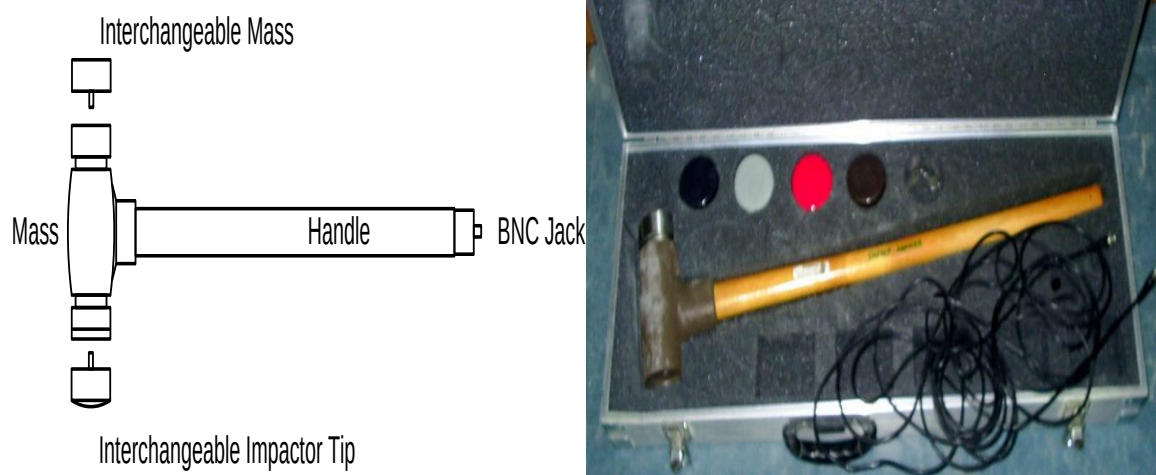


Figure 3.39 Typical impulse-force hammer Figure 3.40 Impact hammer with tips

3.4.4 Accelerometer

The accelerations were measured on each floor of the sample using the DYTRON model PCB-393 C with serial numbers 11197, 11198 and 11199 with a sensitivity of 101.9 mV (m /s²). The accelerometer is shown in Figure 3.41. The data signal for each accelerometer was recorded at various levels of sensitivity to allow adequate resolution of the exponentially decaying vibration signal at all times (Kanwar et al., 2006). To attach the accelerometer to the surface of the test structure, high strength cement gypsum wax was used.

3.4.5 Eight-Channel FFT Analyzer

Spectrum analysis is defined as the conversion of a signal from a time domain representation to a frequency domain representation. There are four forms of Fourier transformation, which are explained below:

- a) Fourier series: Converts an infinite periodic time signal into an "infinite discrete frequency spectrum".
- b) Fourier Integral Transform: Transforms a continuous time signal into a continuous, frequency spectrum.
- c) Discrete Fourier Transform (DFT): transforms a discrete periodic time signal into a discrete periodic frequency spectrum
- d) Fast Fourier Transform (FFT): a computer algorithm for calculating the DFT



Figure3.41 Accelerometers

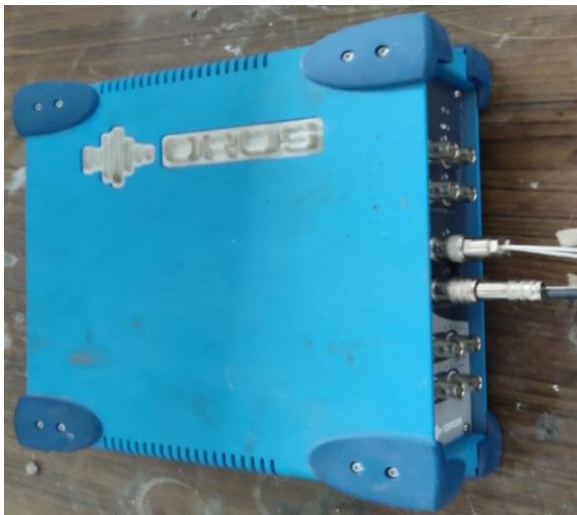


**Figure 3.42 OROS-35 Eight channel
FFT analyzer**

In order to adapt the discrete Fourier transform for use with digital computers, the so-called fast Fourier transform has been developed. The FFT is simply an algorithm for fast and efficient calculation (Formenti, 2000, Finno and Prommer, 1994). Cooley and Tukey discovered the FFT in 1967, much earlier, though it was hard to operate without a digital computer. The FFT algorithm limits the signal and resulting spectrum in some way. For example, the sampled signal to be consisting of a number of samples corresponding to a power of two. The "frequency range" covered by the FFT analysis. depends on the number of samples taken and the sampling rate (Raghavendrchar and Aktan, 1992). For experimental modal analysis, the configuration was changed by connecting the accelerometer and the charge amplifier to a laptop with data acquisition software. In this study, the eight-channel FFT analyzer from OROS-35 Instruments was used, which is shown in Figure 3.42.



Figure 3.43 Computer with eight channels FFT Analyzer

3.5 METHODOLOGY OF TESTING

The test method involved testing both the control and modified RC frames. The test was carried out to determine the behaviour of the lateral load resistance and the vibration characteristics of both sets of frames. The vibration response of the structures was measured using a impact hammer, accelerometer and an FFT analyzer. The device configuration for vibration testing is shown in Figure 3.43. The behaviour of the lateral load resistance implies the determination of the load-deflection ratio in an RC frame model in which the load is applied to the upper floor and the corresponding displacement is measured. From the load deformation curves generated, both the displacement ductility index and the energy dissipation capacity were calculated. Subsequently, the non-linear modeling of the experimental data-verification framework was performed. This procedure is explained in the following subsection.

3.5.1 Damage Loading

To simulate a change in structural rigidity, a hydraulic cylinder was installed horizontally in the desired direction at top floor. The floor of the frame was equipped with a LVDT in the horizontal direction. The relationship between loads and displacement of frame was measured, and then the jack was removed to measure the stiffness index in various damage states. The load of the three different frames is explained as follows:

3.5.2 Load deflection test:

The deflection controlled load was uniformly applied in the original frame with an increase in deflection of 5 mm every cycle from the initial state to the final damage state

(total failure). The damaged frame was then retrofitted by GFRP and CFRP wrapping and tested again with the same pattern. After the second test, ECC was used to strengthen the frame. The load was re-applied with the same pattern until the final damage condition was regained.

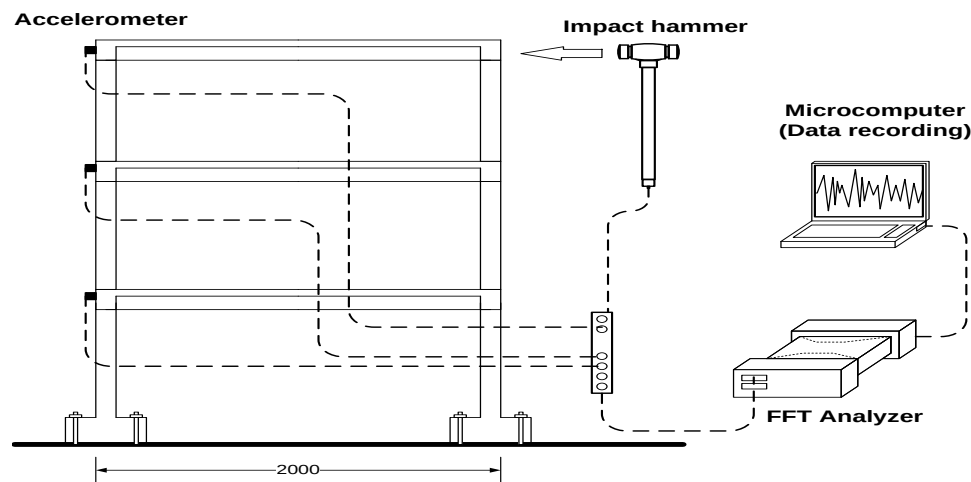


Figure 3.44 Vibration test setup



Figure 3.45 Vibration testing of frame at laboratory

3.5.3 Vibration test

The vibration test was used to determine the condition of an RC structure for the original, which was retrofitted with GFRP and CFRP and with ECC reinforced specimens. An instrumented power hammer was connected to the force input port 1 of an FFT analyzer. In general, the hammer is directly connected to the power channel because the FFT analyzer powers an integrated piezoelectric (ICP) power hammer. The response signal was measured with accelerometers attached to the frame at each floor. Each accelerometer was connected to the vibration input port of the FFT analyzer. The vibration response data was recorded on a

laptop. Initially, the vibration test included the control frame test for both aspects. The properties were re-examined after retrofitting the same frame using GFRP, CFRP. A similar procedure was also adopted for the ECC retrofitted frame. A typical set up is figure 3.44 and actual set up is Figure 3.45.

3.5.4 Data recording

The data of the dynamic structure characteristics such as acceleration behaviour, resonance frequency, mode shape, attenuation, frequency response function and coherence function were obtained by simultaneously sampling the impact force signals and measuring the system response. The time waveforms detected by the impact force and the system response include amplitude and relative phase information. The dynamic properties of the structure could be calculated from the Fourier transforms of the impact and response signals using the FFT analyzer.

CHAPTER 4

STRUCTURAL RESPONSE PARAMETERS

4.1 GENERAL

This chapter discusses the parameters associated with the fundamental reaction of stiffness, resistance, and ductility. The rigidity of the structure is the most important parameter to respond to the service capacity requirements. Similarly, strength is used to control the degree of inelasticity under external loads. Finally, collapse prevention is the most affected by ductility. This chapter also presents the load-deflection behaviour and a discussion of the failure mechanism of infilled frames under lateral loads. The RC frame subjected to various damage levels and are then repaired with GFRP / CFRP and ECC and then the repaired frame has been investigated.

4.2 STRUCTURAL RESPONSE PARAMETERS

The focus of the most important parameters describing the behavior of structures under cross-loading is based on the three response characteristics of stiffness, resistance and ductility.

4.2.1 Stiffness

Stiffness is the ability of a component or set of components to resist deformation under load. It is expressed as the ratio between load and deformation at a given value of one of the two quantities and the corresponding value of the other. Therefore, the stiffness is not a constant value. In Figure 4.1, K_i is the stiffness at a required deformation Δ_i and corresponds to the resistance to the force P_i . When increments or first derivatives of loads and deformations are used, the resulting stiffness is the tangent value. When total loads and deformations are used, the resulting stiffness is the secant value (Taranath, 1998). Stiffness is the relationship between the actions and deformations of a structure and its components, while the flexural stiffness of the element is a function of the lateral resistance mechanisms used. The relationships between geometry, mechanical properties, actions and deformations can be assumed from the principles of mechanics. For RC and masonry structures, stiffness can be taken as a secant at the yield point or at any other point selected in the response curve. Slippage at connections, local deformation and yielding in steel structures are the counterparts to the above discussion on RC structures.

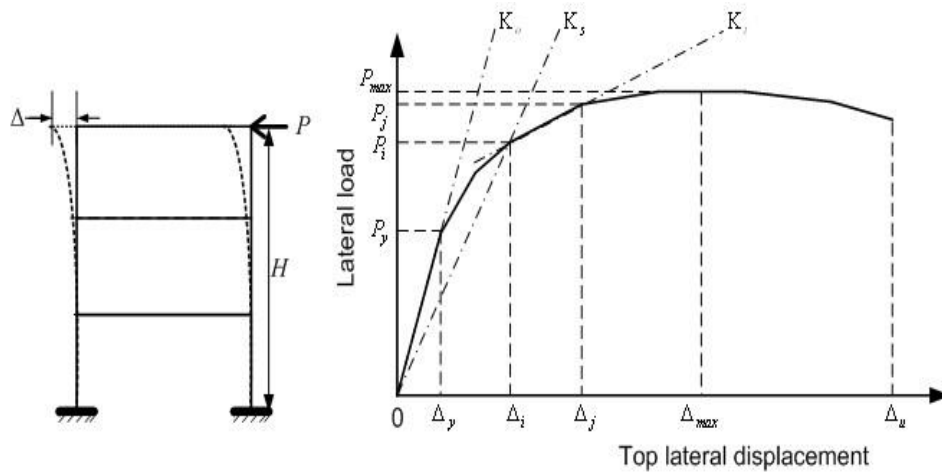


Figure 4.1 Definition of initial, secant and tangent stiffness of structure (Elnashai and Sarno, 2008)

Figure 4.1 illustrates a graphic representation of the structural behavior of a system exposed to lateral loads. The response curve is represented by the lateral load P versus the top horizontal displacement. The initial slope K_0 is the elastic stiffness of the structure, while the secant rigidity is the slope K_s of the line corresponding to a given load level. The initial stiffness is higher than the secant stiffness K_s for conventional materials. For the latter, values of are generally used to define secant rigidity. Deviations in stiffness in the inelastic range are often expressed by the tangent stiffness K_t , which is the slope of the tangent to the response curve for a given pair (Elnashai and Sarno, 2008; McGuire et al., 2000). A decrease in K_t values shows that softening of the structure occurs. When analyzing inelastic structures, secant rigidity is often used to avoid dealing with the negative tangent stiffness beyond the maximum resistance to action.

Depending on the type of loads applied, different types of stiffness can be defined. Structures designed for vertical loads generally have sufficient vertical stiffness. The seismic load or the wind load generates inertial forces due to vibration of the masses. The horizontal components of these inertial forces are often dominant hence lateral stiffness is of paramount importance to structural engineers. The definition of lateral stiffness depends essentially on the region of interest in the response domain. The stiffness of a system is mainly related to the satisfaction of the functionality of the structure under lateral loads (Penelis and Kappos, 1997, Paulay and Priestley, 1992).

4.2.2 Strength

Strength is the ability of a component or set of components to withstand the load in a given reaction. It is not a constant value. The term "capacity" is preferable to the term "strength" to represent both action resistance and the ability to resist deformation. In Figure 4.1, P_y is known as the yield strength, corresponding to the yield displacement (Elnashai and Sarno, 2008). Power defines the ability of a member or assembly of members to resist actions. This ability is generally defined as a function of the type of force applied. Axial, bending and shear strength are used to quantify the capacity of 2-D structures and their components. In the response curve of the structural system, the overall shear capacity of the system base is defined in terms of the base shear at yield or maximum strength with respect to the shear capacity of the base (Motos and Dodds, 2002, Bruneau et al., 1998, Paulay and Priestley, 1992). Alternately, shear capacity can be expressed at any intermediate point between yield and maximum strength. Similarly, bending and axial resistances are evaluated by moment-rotation relationships and axial displacements under the axial load.

The definition of resistance is easier than that of stiffness. The relationships between geometry, mechanical properties and resistance can be derived from the principles of engineering mechanics. These relationships depend on the type of building materials used. Uncertainties in the assessment of structural capacities are attributed as

- i. the randomness of the material properties, in particular the strength parameters,
- ii. geometric properties, e.g. section and size of member, and
- iii. construction quality.

Attainment of shear, flexural, and axial capacities in a structural system can result in damage to structural components. The damage affects the security of a system, but does not necessarily lead to the breakdown of the structure. Collapse prevention is the limit state of behavior that is controlled by ductility.

4.2.3 Ductility

Ductility is the ability of a component or set of components to deform beyond the elastic limit, and is expressed as the ratio between the maximum value of a deformation amount and the same amount in the elastic limit state. In Figure 4.2, the ductility of the displacement is the ratio between maximum or final displacement Δ_u and the yield displacement Δ_y . In other words, ductility is the capability of a material, component,

connection, or structural system to undergo inelastic deformations with acceptable stiffness and strength reduction. In the force-displacement relationships for systems, brittle systems fail after having achieved their resistance to very little inelastic deformation. In force-displacement relationships for systems, brittle systems fail after reaching their strength at very low inelastic deformations. In contrast, the final deformations corresponding to the final lateral forces are greater in the ductile systems than in the brittle systems (Elnashai and Sarno, 2008 and Carlin, 1998). Figure 4.2 shows the definition of structural ductility between a brittle system and a ductile system.

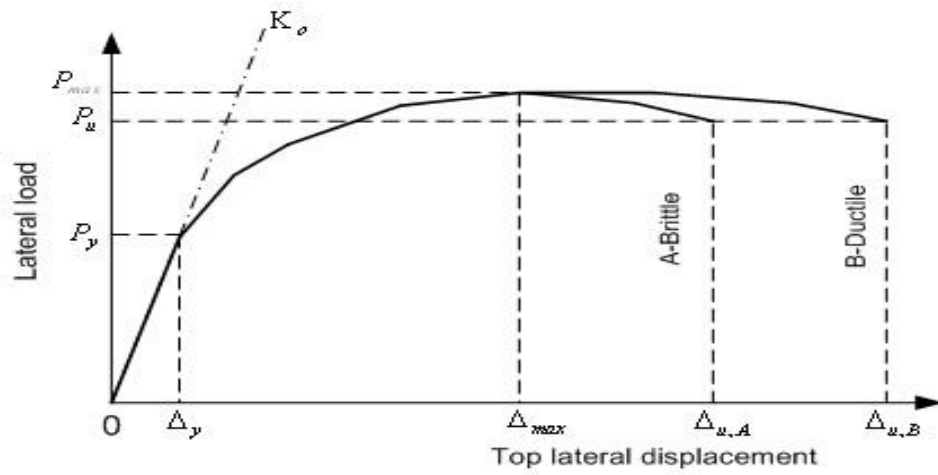


Figure 4.2 Definition of structural ductility (Elnashai and Sarno, 2008)

The response amplitudes of earthquake-related vibrations depend on the energy dissipation of the structures, which depends on the ability of the structure to absorb and dissipate energy through ductile deformations. With low energy dissipation, structural systems can develop stresses that correspond to relatively large lateral loads. Structural ductility or displacement ductility is a global measure of the inelastic performance of sub-assemblages or systems exposed to horizontal loads. Several factors can lead to a reduction in available ductility. These include primarily strain rate effects that cause an increase in yield strength, the reduction in energy absorption due to plastic deformation upon interaction, an excess of resistance that results in the structure failing to yield when it was designed to yield and the tendency of some materials to have a friction fracture. These factors can affect local and global ductility.

4.3 DAMAGE DETECTION METHODS BASED ON NONLINEAR BEHAVIOR OF STRUCTURE

In structural damage detection, a Damage Index (DI) is used to control the damage status of the structure. This value equals zero if there is no damage and equals one if a total collapse occurs. Damage rates can be classified in several ways. The local damage index indicates the damage of the structural elements or part of the structure, while the global index takes into account the damage to the entire structure. There are several parameters for physical reactions to formulate the damage index of a structure. These parameters can be classified as deformation, change of stiffness, energy loss and change of dynamic parameters (Estekanchi and Arjomandi, 2007). A damage index may include a combination of one or more damage variables in its calculation. Some significant damage indices are as discussed below.

4.3.1 Park and Ang Damage Index

Park and Ang (1985) formulated a known damage index to estimate the degree of damage in the structures. This damage index represents the damage caused by the maximum deformation and the damage caused by the energy released during cyclic loading. Both damage components are combined linearly:

$$DI_{Park} = \frac{\delta_{max}}{\delta_u} + \frac{\beta}{\delta_u P_y} \int dE_h \quad (4.1)$$

where δ_{max} is the maximum experienced deformation; δ_u and P_y are the ultimate deformation under monotonic loading and yield strength of the structure respectively, which can be calculated using nonlinear finite element analysis; $\int dE_h$ is the hysteretic energy absorbed by the structural element during the response history; and β is a model constant parameter.

4.3.2 Stiffness damage index

Kanwar et al. (2007) proposed the stiffness damage index DI_k , using an indicator based on the relationship between the stiffness properties of the undamaged element material and the damaged element of the structure. According to this method, the severity of the damage is expressed as a fraction of the change in the rigidity of an element given by:

$$DI_k = \frac{k_j - k_j^*}{k_j} = 1 - \frac{1}{\nu_j} \quad (4.2)$$

Where ν_j is stiffness ratio, k_j is initial stiffness of undamaged element and k_j^* is the initial stiffness of the damaged element of the j^{th} member. The asterisk (*) denotes the damage state.

4.3.3 Displacement Ductility factor and Drift Ratio

At the service level, the damage is also interpreted as the amount of non-structural damage resulting from excessive deformations of the building. Considering the general damage concept, the ductility factor of the displacement and the drift ratio can be used to predict the damage state of a structure.

4.3.3.1 Displacement Ductility

The structural displacement ductility factor, μ is defined as follows

$$\mu = \frac{\delta_u}{\delta_y} \quad (4.3)$$

where δ_y is the structural yield displacement and δ_u is the structural displacement at ultimate damage state.

4.3.3.2 Drift ratio

The drift ratio is defined as follows:

$$Drift = \frac{\delta_m}{H} \quad (4.4)$$

The structural drift is defined as the ratio between the displacement of structure δ_m at the target position m , and the storey height H given by the equation (4.4).

4.3.4 Stiffness degradation

The difference between initial stiffness and secant stiffness showed the post yield behavior of a component. In this study, the deterioration of stiffness as deterioration of the post-elastic stiffness versus the stretch stiffness [represented by $K_{deg\%}$] was calculated by using equation (4.5). The rate of change in the stiffness in the post-elastic range is the measure of the inelastic performance of the component.

$$K_{deg\%} = \left[1 - \frac{(K - K_y)}{K_y} \right] * 100 \quad (4.5)$$

Where: K – Stiffness of each cycle, kN/mm; K_y – Yield stiffness, kN/mm

4.3.5 Damage Index according to FEMA performance levels

Pushover analysis was used to determine the lateral deformability (ductility) of the structure at different levels of seismic performance and the corresponding force modification factor (R). The seismic performance of the structure according to the FEMA guidelines is shown in Figure 4.3.

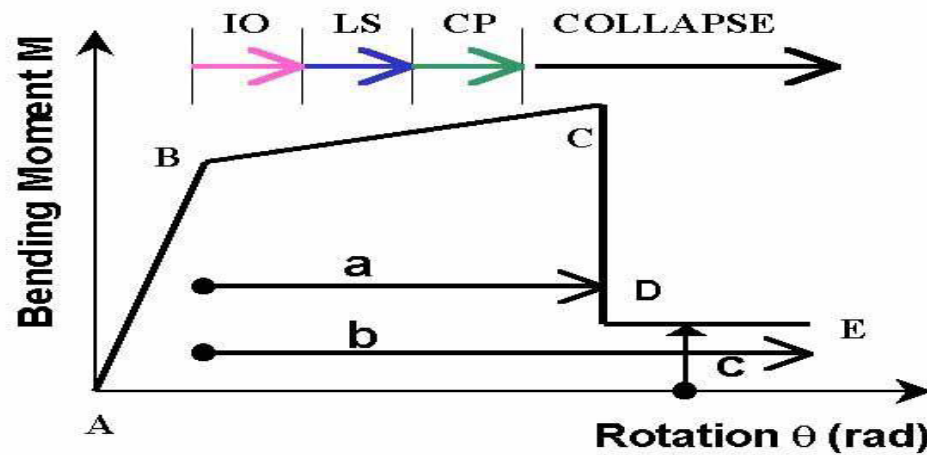


Figure 4.3 Failure mechanism of structure as per FEMA guidelines

Table 4.1: Performance Level and corresponding Damage Index FEMA 356 guidelines

FEMA Performance Level	AB	IO	DC	LS	LSR	CP	C
FEMA Damage Index	0	0.17	0.33	0.5	0.67	0.83	1

1. Linear Limit
2. Immediate occupancy (IO)
3. Damage Control (DC)
4. Life safety (LS)
5. Limited safety structural range (LSR)
6. Collapse Prevention (CP)
7. Collapsed. (C)

4.4 EXPERIMENTAL RESULTS

In order to assess the damage of brick infilled RC frames exposed to lateral loads, the non-linear behavior resulting in deterioration of resistance, stiffness and energy dissipation

capacity are evaluated. It is convenient to use the stiffness damage index and the Park and Ang damage index to calculate the probability of failure of a structure, since the stiffness index is based only on the relationship between changes in structural stiffness. From the Park and Ang damage index, it can be assessed that the failure of the RC elements is due to inelastic deformations and energy dissipation.

This section presents the non-linear behavior of the brick infilled frame and the damage indices.

4.4.1 Control frame

(a) Load Deflection Behaviour

The experimental model of the RC-framed, brick-infilled control frame has been constructed with the ground floor as soft storey. The frame was loaded on the upper floor in a lateral direction as shown in Figure 4.4.

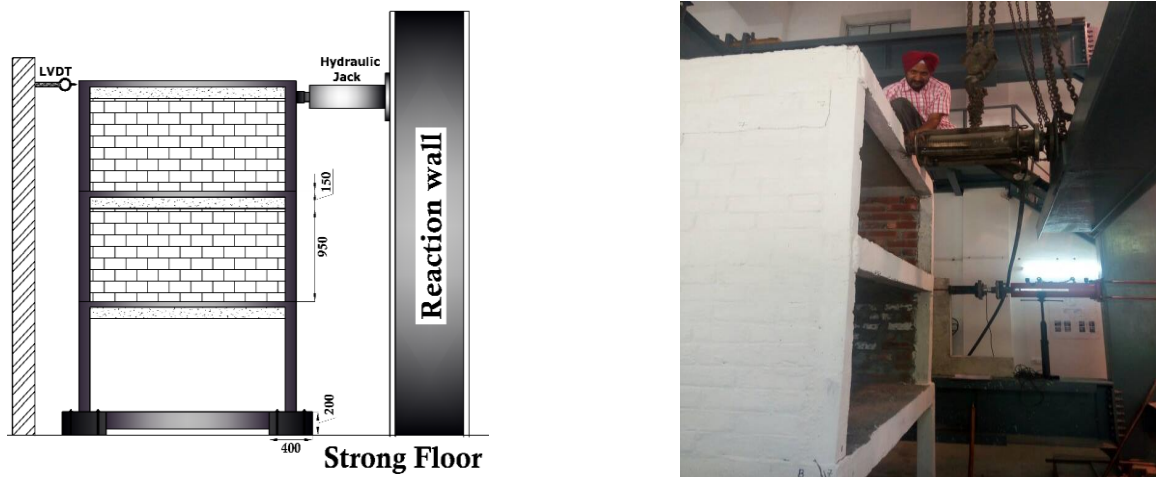


Figure 4.4: Loading set-up in the frames

The quasi static load was evenly applied with a hydraulic jack in small incremental steps as shown in Figure 4.5. The damage indexes have been calculated by using Park Ang Damage Index given in equation (4.1).

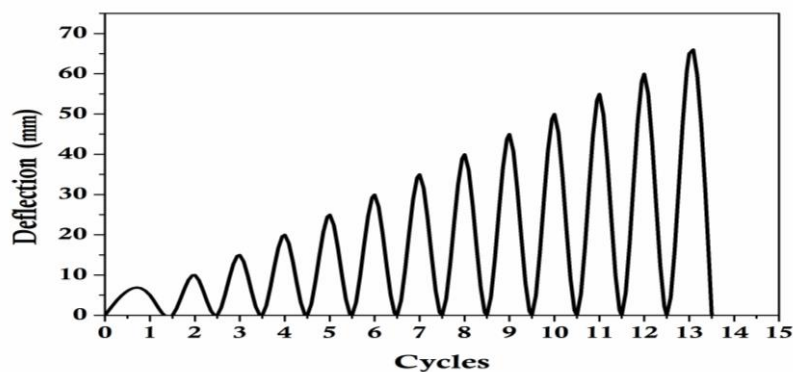


Figure. 4.5. Deflection behaviour of a frame in different cycles

It can be observed that the amount of damage increases with increasing load in each cycle. The brittle nature of the concrete allows the early formation of cracks in the hinge region. The summary of the load and deflection behavior of the control frame is shown in Table 4.2. The hysteresis behavior in the form of load-displacement of the control frame is shown in Figure 4.6.

The experimental results indicate that the initial diagonal cracks in the beam-column joints occurred in a ground floor with a load of 35 kN, which corresponds to a drift ratio calculated by equation (4.4) of 0.35%, indicating the structural elements of the ground floor were most stressed. The damage index of the DI_{Park} system was 0.078 that indicates a moderate state of damage. Large cracks occurred in the bottom column of soft storey and small cracks appeared in the column beam joint connection at a load of 46.2 kN with a displacement of 25 mm (0.877% drift) with DI_{Park} of 0.315. The load deflection behaviour of the Control frame is shown in Figure 4.6. The actual cracking pattern observed at various points on the frame is shown in Figure 4.7(a-i).

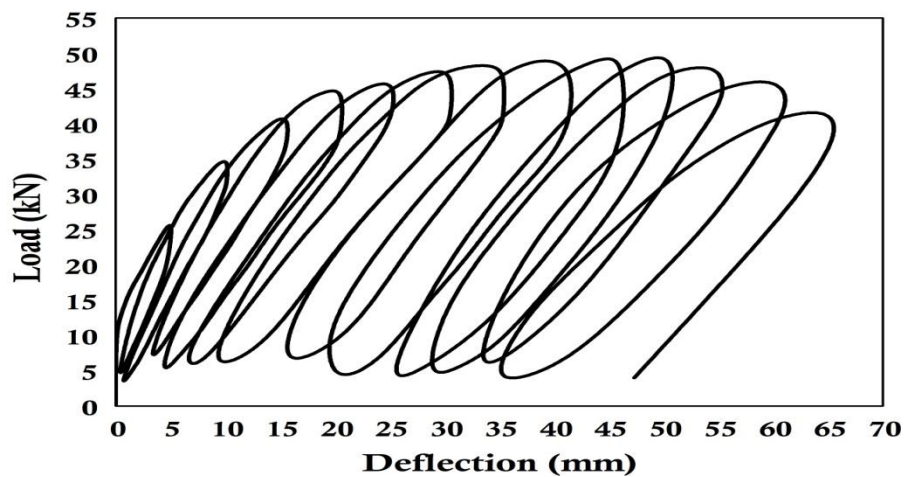


Figure 4.6 Load Deflection behaviour of Control Frame

After removing the load, the frame model returned 25 mm from the original position to a new position, indicating that the yield point was visible at a load of 46.2 kN, which was a severe damage state. The maximum load of the frame, P_{max} is 50.5 kN at a displacement of 50 mm (1.754% drift), resulting in a DI_{Park} of 0.716.

Table 4.2 Load/ Deflection value of Control, FRP Retrofitted and ECC Repaired frame

Deflection (mm)	Drift Ratio (%)	Load (kN)		
		Control frame	FRP Retrofitted	ECC Replaced
0	0	0.00	0.00	0.00
5	0.175	25.90	17.58	21.65
10	0.35	35.00	28.28	31.94
15	0.526	41.00	35.70	39.92
20	0.701	45.00	42.19	46.44
25	0.877	46.20	44.17	51.60
30	1.052	47.90	47.14	54.87
35	1.228	48.50	50.8	58.10
40	1.403	49.50	53.94	59.70
45	1.579	49.90	55.22	60.85
50	1.754	50.50	56.03	61.65
55	1.929	48.50	54.95	61.95
60	2.105	46.60	53.98	59.65
65	2.280	42.30	52.21	57.50

In addition to the beam-column joints, damage occurred in the brick infill (Figure 4.7 (i)). From this level, the load began to decrease with increase in deflection to a maximum load of 42.3 kN with a deflection of 65 mm. The energy dissipation was 4397 kN-mm shown in Figure 4.17. The general trend of DI_{Park} changed shape as the cumulative effects of deformation and energy dissipation were low when the damage was less than moderate. In a greater deformation trend, DI_{Park} changed its shape as the load carrying capacity of structure diminished after large cracks appeared in the structure. At the final state, the crack pattern in joints of soft storey was diagonally shaped, but there was a slight horizontal crack in the masonry infilled frame. This indicates that the shear resistance capacity of the soft storey was low.

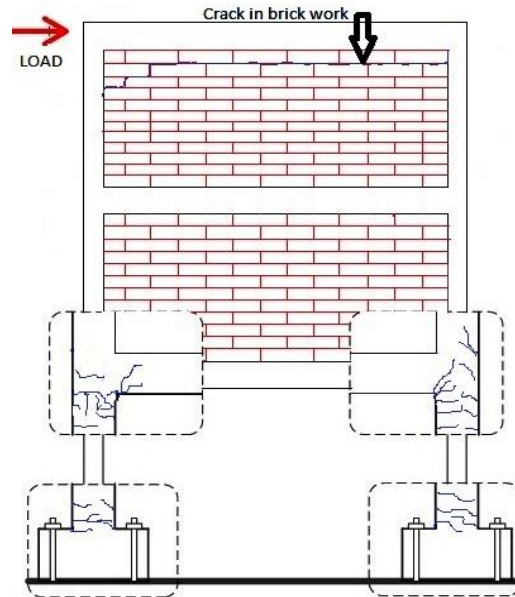


Figure 4.7 Crack pattern in Control Frame

(b) Crack pattern

The results indicate that in the 5th cycle at the state of yield diagonal cracks were observed at the beam-column joints of the soft storey. These opening cracks increased with increasing load. The second and third diagonal cracks appeared in the beam-column joints of the soft storey in the 6th and 7th cycles, respectively. Due to the rigidity caused by the brick infill, no cracks were observed in the first and second floor beam-column joints. However, there was a horizontal crack in the top course of the brick infill at a deflection of 45 mm.



(a) Column-beam joint in A



(b) Column-beam joint in B



(c) Column-beam joint in C



(d) Column-beam joint in D



(e) Column hinge A



(f) Column hinge B



(g) Cracks in Column hinge C



(h) Cracks in Column hinge D



(i) Cracks in brick infill

Figure 4.8: Damages observed in beam-column joints and column-foundation joints in control frame under lateral loading.

The failure modes of the control frame showed that the lateral load capacity became insufficient due to the non-ductile detailing that did not include beam-column joint transverse reinforcement. Therefore, the top and bottom bars move in opposite directions under the combined force. These forces are compensated by the bond stress created between the concrete and the steel bars. Under these circumstances, the plastic hinges were formed by debonded bars, and the beam-column joints lost their ability to carry loads.

The structure suddenly lost its load carrying capacity when cracks from tension surfaces were transferred to compression surfaces of structural elements. In the final state, diagonal and flexural cracks occurred in the column beam joint floor, indicating that the failure mode of the control frame has a combination of a shear mode and flexural mode failure. Flexural cracks have been also observed at the columns near the stub foundation.

The pattern of cracks was critically monitored during the test and marked on the surface of the specimen. Four columns were labeled for identification as A, B, C, D, as presented in Figure 4.9. The brick infilled in the first and second storey made the ground storey soft and therefore the intensity of the failure on the ground storey was high. The plastic hinge formation was observed just below the first floor beam column joint connection and at the column foundation connection. Initially, flexural cracks were observed in the area of the beam column joint, followed by shear cracks in the joint. With increasing deformation, dense cracks were observed at the foundation hinge area as depicted in Figure 4.7 (e-h).

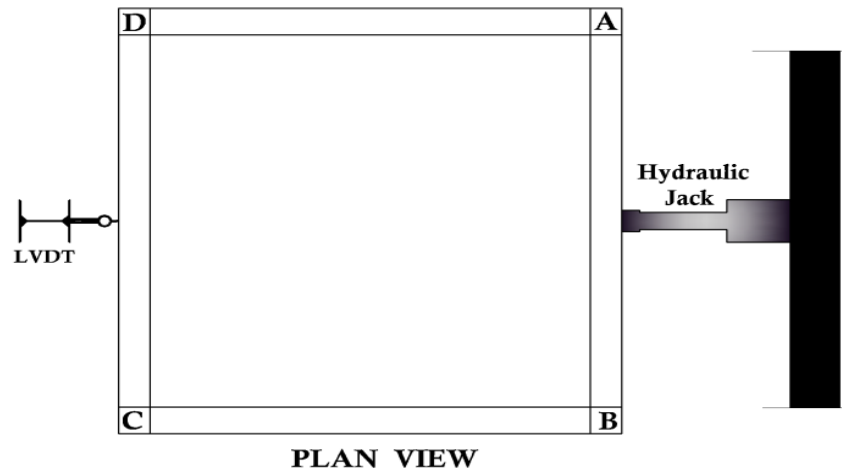


Figure 4.9 Plan of the experimental model

Predominant cracks have been observed in the first floor beam column joint region, while the second floor and the roof showed no signs of cracking due to the soft storey effect. The horizontal crack observed below the joint region reduced stiffness and limited ductility, although it was followed by shear cracks.

As a result, a number of crack on the concrete was observed in the joint region. The effect of the seismic detailing supported the deformation of the RC frame, but the brittle nature of the concrete showed concrete crushing failure at the foundation hinge region. There was no severe damage in the infill horizontal sliding cracks which were observed in the infill at the mortar joints as shown in Figure 4.7(i).

4.4.2 FRP Retrofitted Frame

(a) Load Deflection Behaviour

The lateral load test for the FRP retrofitted frame was done the same way as done in control frame. After completion of the pilot testing on the control frame, the cracks of the damaged frame were grouted with crack Injector epoxy 1315 and the surface was made smooth with putty. The damaged portion of the frame was repaired with GFRP wrapping at the beam-column joints and with CFRP at column hinge with epoxy 2200 according to the procedure explained in Section 3.2.9. The quasi static load was uniformly applied in small increments by means of a hydraulic jack as in the control frame. The hysteresis behavior of the repaired GFRP / CFRP frame is shown in Figure 4.10.

The results also showed that the performance status was obtained in one cycle at a load of 42.2 kN with a displacement of 20 mm, which corresponds to a drift ratio of 0.701%.

The damage rate of the DI_{Park} system was 0.234. There was sliding noise of the GFRP blades in the fifth cycle under 25 mm deflection at a load of 44.2 kN, indicating that crack Injector 1315 was repairing the cracks well in the repaired frame. The fiber slip noise was observed with a bending load of 43.25 kN on the beam-column joints. The relationship between load and displacement can be considered nearly linear if the load is less than or equal to the yield point.

The ultimate state was achieved with a load of 56.03 kN and a displacement of 50 mm from the initial state (1.754% drift). In this state, the DI_{Park} was calculated as 0.708. After this step, noises of fiber and epoxy layers slipping were generated before failure. The load began to decrease with increasing deviation with a maximum load of 52.21 kN with a deviation of 65 mm. The energy dissipation was 4812.7 kN-mm (see Figure 4.17).

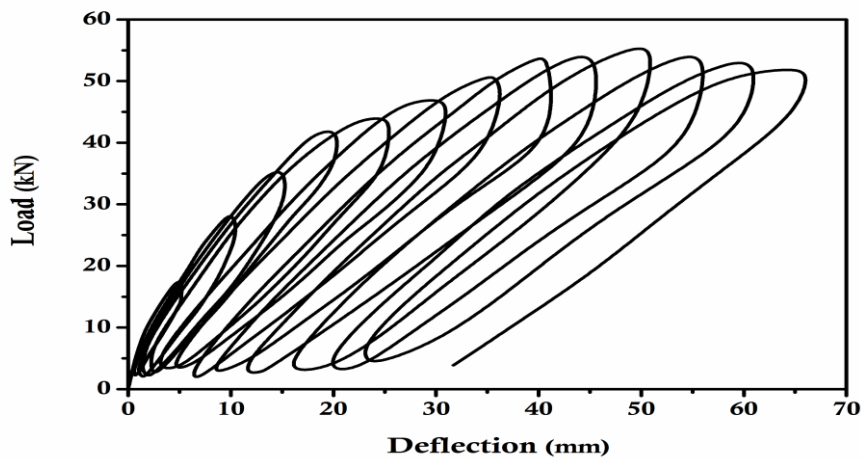


Figure 4.10 Load Deflection behaviour of FRP Retrofitted Frame

The summary of the load and displacement behavior of the GFRP/CFRP retrofitted frame is presented in Table 4.2 together with the results of the control framework. The pattern of cracks in the retrofitted frame is shown in Figure 4.11. No significant flexural cracks were observed in the outer fiber layers. The failure mode of this retrofitted frame was determined by the sliding of the fiber layers. In addition, flexural failures occurred in the connecting joints of the columns and beams on each floor.

The results of the FRP retrofitted frame showed that the use of GFRP wrap for structural reinforcement resulted in significant lateral loading, which increased to 11% compared to the control frame. The FRP sheets wrapped around the structural elements in this manner are intended to provide external confinement and crushing of the concrete cover at larger displacements. In accordance with the change in structural displacements, the ductile

behavior and cumulative energy dissipation of the retrofitted frame after application of GFRP panels has generally been largely restored. There were no cracks in the hinge of the column and no sliding noise from CFRP retrofit up to the maximum load capacity of the retrofitted frame. After the 12th cycle, a column hinge failure occurred below the CFRP envelope (see Figure 4.11 (a-d)). The DI_{park} of the retrofitted frame shows a sudden increase after the tenth cycle. This is possibly because of the occurrence of severe damage in the unconfined region and part of FRP rupture during larger deformation.



(a) Cracks in Column A



(b) Cracks in Column B



(c) Cracks in Column C



(d) Cracks in Column D



(e) Beam-Column Joint A



(f) Beam-Column Joint B



(g) Beam-Column Joint C



(h) Beam-Column Joint D

Figure 4.11: Damage in beam-column and column-foundation in FRP retrofitted Frame under lateral loading

(b) Cracking pattern

The experimental results of frame indicate that after the structure reaches yield load, the bond of flexural GFRP layers delaminate along the interface between the concrete and GFRP. The presence of GFRP in the plastic region of the hinge of the carrier and the column effectively resisted the stress without cracks in the joint. The cracks started above and below the GFRP strengthened region. The effective confinement of FRP allowed the deformation of the frame and limited the early failure of the connection. It was observed that delamination was always achieved in the corners of the beam-column joints in the stress zones. The cracks with sliding noise of the fiber layer began in the 5th cycle and later other cracks formed in the

soft storey column because no crack was visible due to FRP wrapping. After the GFRP sheet debonded, the retrofitted structures lost their load carrying capacity.

The GFRP confinement layers suddenly delaminated with breaking noise at the maximum load. At the edge of the GFRP layers, cracks appeared in the final state and large cracks appeared inside the RC elements behind the strengthening layers. However, there was a wide horizontal crack under each CFRP retrofitted column at the column stub foundation joint, which was preferable to overloading of the footing. In this position, there was a gap of about 5 mm between the edge of the CFRP jacket and the adjacent stub foundation, which indicates the carrying capacity of the column with the help of the reinforcing material. Again, the primary failure mechanism shows the debonding of the FRP sheets. It occurred at the base of the hardly huge cracks in the stress zones of the damaged RC elements. The retrofitted specimen showed reduced loop area during initial cycles due to the FRP confinement. As a result of the employed yielded reinforcement, the yield deflection is high and yield stiffness is low in retrofitted specimen over control specimens.

The retrofitted frame showed almost 11% higher load capacity and better performance even with yielded reinforcement. The maximum load was achieved with a deflection of 50 mm in the control and is 50 mm in retrofitted frame, which showed the FRP confinement resistance to deformation. As the displacement increased, cracks formed above and below the strengthened area, and therefore a reduction in the load was observed. However, the failure of the concrete in the hinge of the column and the diagonal cracks in the joint showed a sudden decrease in carrying capacity. The damaged frame has been retrofitted and restored its original strength. This is an achievement of the study and damaged frame showed better performance than the control frame after retrofitting even after using the yielded reinforcement.

In the retrofitted frame, the internal cracks were injected with epoxy and after a thorough surface preparation, FRP wrapping was done. Therefore, the failure pattern of the modified frame was different from the control frame. The FRP strengthening at the column hinge location and beam column joint made strengthened portion stiffer and hence failure was concentrated in the un strengthened region. Figure 4.11 shows the cracks under and over the strengthened part, followed by rupture of FRP in some places. The primary failure is because of the horizontal crack below the FRP strengthened region. Therefore, the final deformation of the reinforced frame is not identical to the control frame.

4.4.3 ECC Strengthened Frame

(a) Load Deflection Behaviour

In the first repair case local treatment of the cracks has been done with epoxy and wrapped with GFRP/CFRP whereas for once this frame get damaged, the entire retrofitted concrete was removed along with yielded steel bars and replaced with ECC. The ECC repaired frame again tested in the same manner as with control and FRP retrofitted frame. The quasi-static load was uniformly applied in small increments by means of a hydraulic jack as applied in the control and the FRP retrofitted frame.

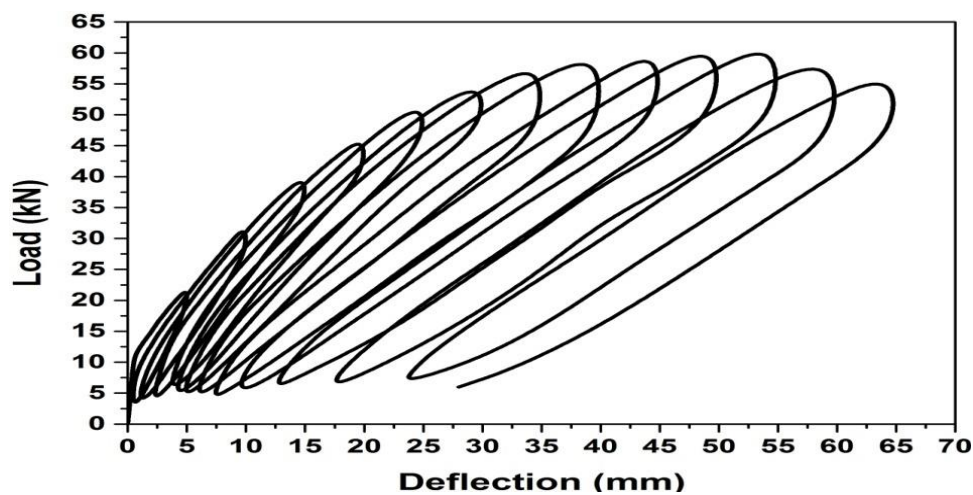


Figure 4.12 Load Deflection behaviour of ECC Strengthened Frame

The hysteresis behavior of the ECC strengthened frame is shown in Figure 4.12. Concrete damaged in the RC frame after a cyclic load failure was removed by holding the frame with jacks. The damaged and old reinforcement replaced because it lost strength due to multiple cycles of deformation. Before the placing of ECC mix in place of old concrete, the Epoxy 2200 jointer was applied to the surface of the old concrete of foundations and joints to create a better bond between the old concrete and the new concrete prepared as ECC concrete. Bricks were filled back into the frame to restore the frame's strength.

The experimental results show that the first cracks in the column occurred in a soft-floor column with a load of 54.9 kN, which corresponds to a drift of 1.052%, indicating that the structural elements of the ground floor were the most heavily stressed. The damage rates of the DI_{Park} were 0.365, a moderate state of damage. Large cracks started to appear in the column of soft storey and small cracks appeared in the column beam joint connection at a load of 58.10 kN with a displacement of 35 mm (1.228% drift), DI_{Park} of 0.438. The load taken by the ECC repaired frame with respect to deflection is mentioned in Table 4.2.

Damage Index is presented in Table 4.3. The crack pattern of this experimental frame is shown in Figure 4.13.



(a) Beam-Column Joint A



(b) Beam-Column Joint B



(c) Beam-Column Joint C



(d) Ductile behaviour of column D

Figure 4.13: Construction and Damage in Column-Beam and Column-Foundation after Loading in ECC Repaired frame

The maximum load taken by the frame (P_{max}) was 61.95 kN with a displacement of 55 mm (1.929% drift), resulting in a DI_{Park} of 0.731. In addition to the column-beam joint, a horizontal crack appeared in the brick infill in a different location. After this level, the load began to decrease with increasing deflection with a maximum load of 57.5 kN with a deflection of 65 mm. The comparative load deflection curve of all frames is shown in Figure 4.10. It was observed that the energy dissipation was 4962.3 kN-mm, as shown in Figure 4.17. The general trend of DI_{Park} is slightly changed because the cumulative effects of deformation and energy dissipation are low when the damage is less than moderate. In a higher deformation trend, DI_{Park} changes shape when the structure starts to lose its load

carrying capacity after cracks occurred in the structure. The crack pattern was diagonally shaped in soft storey joints but light horizontal crack appeared in brick infilled frame. It indicates that shear resistance capacity of soft storey was low.

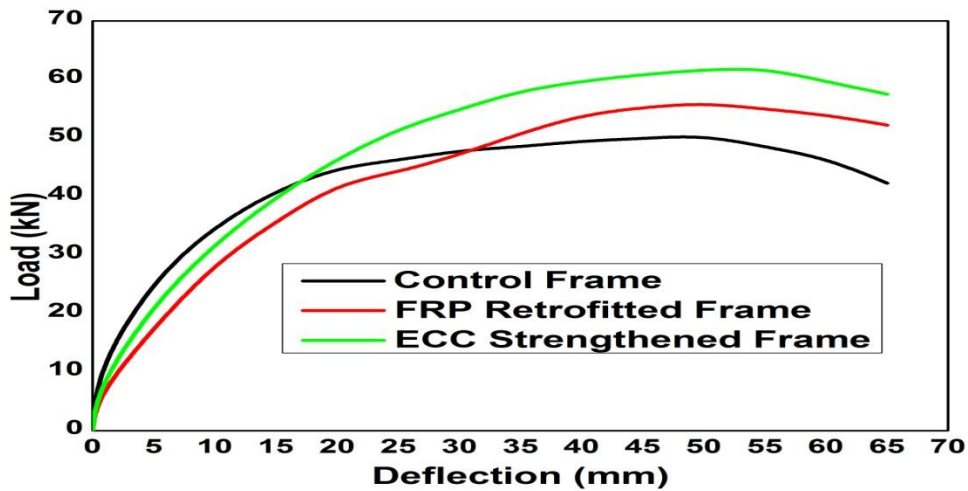


Figure 4.14 Load deflection behaviour of different frames

(b) Cracking pattern

The patterns of experimental cracks in the final state of the control frame models are shown in Figures 4.14 (a) to 4.14(d). The Figure shows that in the yield damage state first diagonal cracks were observed in the column of the soft storey in 6th cycle due to ductile behaviour of ECC. These opening cracks increased with increasing load. The second and third diagonal cracks occurred in the seventh and eighth cycles, respectively, in the soft storey beam-column joints. No cracks appeared in the infilled panels.

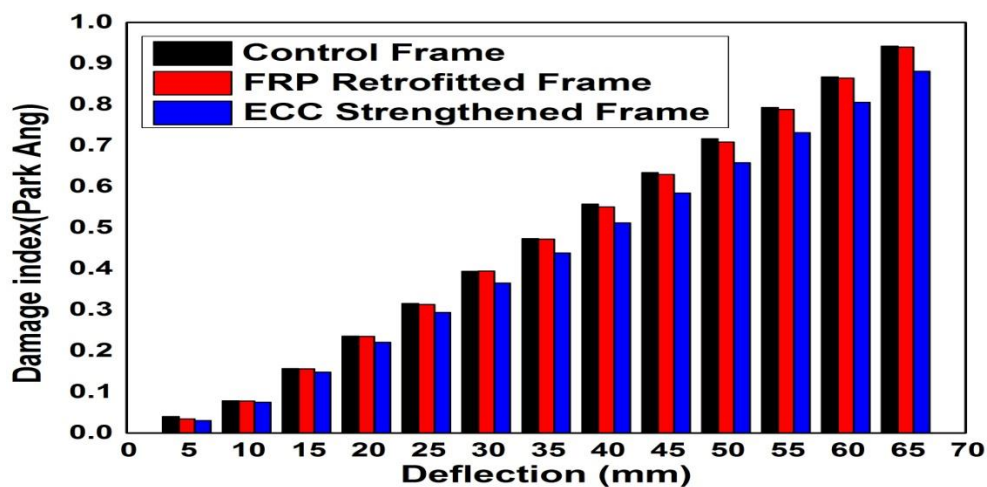


Figure 4.15 Damage Index (Park Ang, 1985)

The hysteretic behaviour of control and ECC strengthened frame with loading as shown in Figure 4.7 and Figure 4.9 was analyzed. The loop clearly demonstrates the

influence composite in resisting the lateral loads in comparison to control frame. The hysteresis curve of control frame shows higher residual deflection in each cycle as compared to frame strengthened with ECC followed by FRP retrofitted frame. This is because of the damage which occurs in the plastic hinge location in control frame under lateral loading. In ECC repaired frame, the composite allowed multiple distributed cracks in the hinge region which allowed the frame to deflect more with lesser residual deflection. The observed residual deflection of control frame at peak load was 45mm whereas in ECC frame it was 25mm which was 55% lesser than control frame. Also the usage of ECC increases the yield deflection from 20mm to 30mm.

Table.4.3 Damage Index in Control, FRP Retrofitted and ECC Strengthened frame [Park Ang 1985]

Deflection	Control Frame	Retrofitted Frame	ECC Frame	Damage Index	FEMA-356 Guidelines (2000)
0	0.00	0.00	0.00	No damage	IO
5	0.04	0.034	0.03	No damage	IO
10	0.0783	0.07772	0.07485	No damage	IO
15	0.15667	0.15614	0.14816	No damage	IO
20	0.23564	0.23484	0.22105	Minor damage	DC
25	0.31553	0.31324	0.29359	Minor damage	DC
30	0.39352	0.39403	0.3652	Major damage	LS
35	0.47315	0.47199	0.43841	Major damage	LS
40	0.5574	0.55057	0.51164	Repairable	LSR
45	0.63413	0.62931	0.58434	Repairable	LSR
50	0.71642	0.70845	0.6579	Beyond repair	CP
55	0.79269	0.7882	0.73159	Beyond repair	CP
60	0.8673	0.86431	0.80509	Collapse/Beyond repair	C /CP
65	0.94202	0.93995	0.88064	Collapse	C

It proves that ECC is more flexible and allows more rotation than control concrete. It was also observed that ECC strengthened frame showed 22.7% higher load carrying capacity than control frame and 11% than FRP retrofitted frame and provided better post yield behaviour even with yielded reinforcement. The maximum load was attained at a deflection of 50mm in control, 50mm FRP retrofitted and at 55mm deflection in ECC strengthened frame. It confirms the influence of ECC in its resistance to deformation. Further as the displacement increases, progressive cracking was observed in the strengthened region. The damaged frame has been restored more strength than the original strength. This is an achievement of the study and damaged frame showed better performance 22.7% more than the control frame and 11% more than FRP retrofitting frame.

Energy dissipation is estimated through the area under the load deformation curve. Figure 4.17 shows the energy dissipation curve of frames. The performances of all the frames was similar up to 45mm deflections. As a result of severe cracking in the hinge region, the control frame failed to show better post peak load carrying capacity. Thus, the decrease in energy dissipation after 45mm was observed in control frame. The absence of sudden failures and better performance beyond the peak of the updated framework showed a gradual and increased energy dissipation capacity beyond the control framework.

The stiffness degradation versus post elastic drift for each tested specimen is plotted in Figure 4.18. The rate of change of stiffness degradation over the post elastic range is the measure of inelastic performance of the component. Sudden or rapid degradation in strength and stiffness shows the brittle performance while low rate of stiffness degradation shows ductile performance. The comparative curve of both the specimens shows that the loss of yield stiffness was high in retrofitted specimens over control. This is because of the employed yielded reinforcement in the RC frame. In the damaged RC frame, the surface concrete in the impaired location has been thoroughly removed and replaced with epoxy putty and the internal cracks were injected with epoxy. But the reinforcement in the hinge region was not replaced and hence all the hinge regions were critically strengthened using external FRP reinforcement. Thus, the retrofitted specimens show higher loss in initial and yield stiffness over control frame.

4.5 Comparison of behavior of different frames

4.5.1 Load-deflection and Stiffness

The control scaled model frame was constructed and tested under quasi-static load. Load was applied by increasing deflection of 5mm in each cycle at the middle of top floor to simulate different level of controlled damage (Figure 4.5). From hysteresis behaviour/load-displacement of Control, FRP Retrofitted and ECC repaired frame the envelope curve of control and retrofitted frames are presented in Figure 4.14.

The tangential part of the load-deformation curve shows similar almost linear behavior upto yield, followed by non-linear behavior. The hysteretic curves of both frames are distinct in terms of residual deflection. The control RC frame shows higher residual deflection than retrofitted frame. The retrofitted specimen showed reduced loop area during initial cycles due to the FRP confinement. As a result of the reinforcement, the yield deflection was high and yield stiffness was low in retrofitted specimen as compared to control specimens. The retrofitted frame showed 11% higher load carrying capacity and better post yield behavior even with yielded reinforcement. Maximum load was attained at a deflection of 45mm in control and at 50mm in FRP retrofitted frame showing the confinement resistance offered by FRP to deformation. As the displacement increases, the cracks were formed above and below the strengthened region and hence a reduction in load carrying capacity was noticed. But the concrete fails by crushing in the column hinge and joint diagonal cracks sudden decrease in load carrying capacity. The structural behavior of both the frames was within LS to CP limits as per ASCE 41-06.

In the hysteretic behaviour of control, retrofitted and ECC repaired frame with loading the loop clearly demonstrates the influence of ductile composite in resisting the lateral loads in comparison to control frame. The hysteresis curve of control frame showed higher residual deflection in each cycle as compared to ECC frame. This is because of the damage which occurred in the plastic hinge location in control frame under lateral loading. In ECC repaired frame, the composite experienced multiple distributed cracks in the hinge region which allowed the frame to deflect more with lesser residual deflection. The observed residual deflection of control frame at peak load was 45mm whereas in ECC frame it was 25mm which was 50% less than control frame. Also the usage of ECC increased the yield deflection from 20mm to 30mm. It proves that ECC is more flexible and allows more rotation than concrete. It is also seen that ECC strengthened frame shows 22.7% higher load carrying

capacity than control frame and provides better post yield behaviour even with yielded reinforcement. Maximum load was attained at a deflection of 45mm in control and at 55mm deflection in ECC frame. It confirms the influence of ECC in its resistance to deformation. Further as the displacement increased, progressive cracking was observed in the strengthened region.

The tangential part of the load-deformation curve shows linear behavior up to yield followed by non-linear behaviour in both frames. As a result of the the yield deflection was high and yield stiffness was low in ECC strengthened frame than control specimens. The load-displacement envelope curve is used to estimate the lateral deformation capacity (ductility) of the structure at different seismic performance and the corresponding force modification factor (R) of the structure based on FEMA guidelines are defined as: Linear Limit, Immediate Occupancy (IO), Damage Control (DC), Life Safety (LS), Limited Safety Structural Range, Collapse Prevention (CP), Collapsed(C) (Figure 4.16).

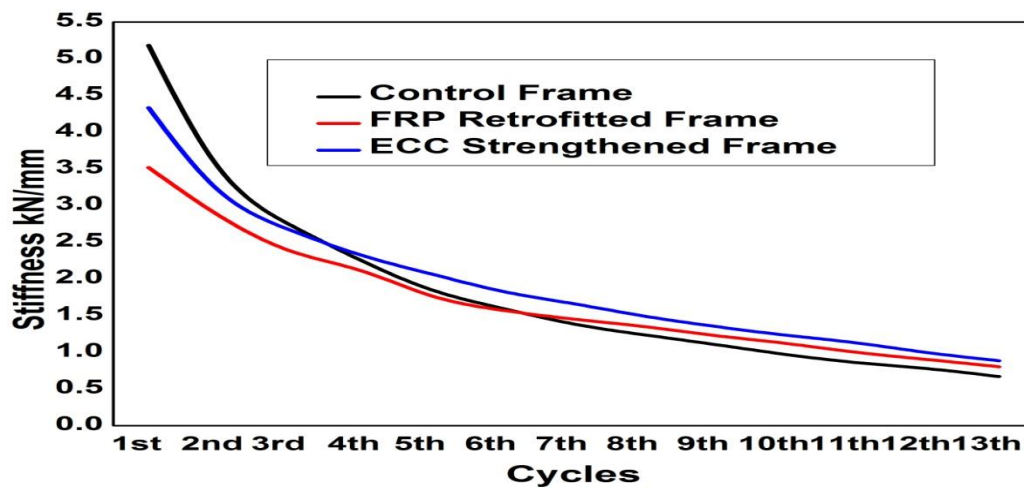


Figure 4.16. Stiffness of all frames (Control and retrofitted)

The concrete crushing failure in the column hinge and joint diagonal cracks showed sudden decrease in load carrying capacity of the control frame. The rate of post peak strength degradation of control frame was higher than the ECC frame. This shows the structural behavior of ECC frame is within LS to CP limits as shown in Table 4.3.

4.5.2 Energy Dissipation

The energy dissipation capacity of a structure is an important parameter for the measurement of its inelastic behavior. The Relative Energy Dissipation (RED) in each cycle is calculated on the basis of the area enclosed by the hysteresis loop. The Cumulative Energy Dissipation (CED) is calculated by summing the energy after each subsequent cycle and

plotted against cyclic displacement. The performances of all frames were similar upto a deflection of 20mm. After that performances of FRP retrofitted and ECC repaired frames were similar but better than control frame upto a deflection of 35mm. As a result of severe cracking in the hinge region, the control frame failed to show better post peak load carrying capacity and a decrease in the energy dissipation capacity after 35mm was noticed in the control frame. On the other hand, in the retrofitted frame, no sudden failure was observed and better post peak performance was reported. The retrofitted frame shows gradual and increased energy dissipation capacity as compared to the control frame figure 4.17.

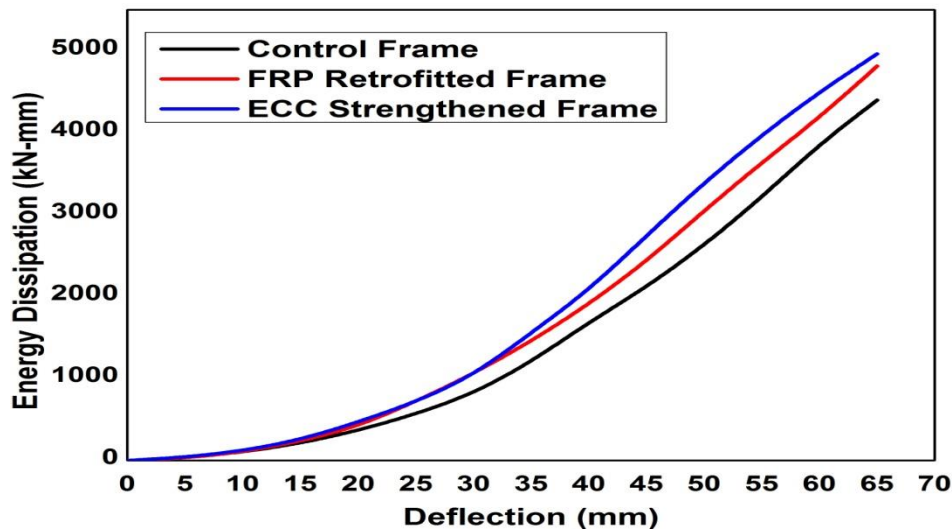


Figure 4.17. Energy Dissipation V/s deflection in different frames

The performance of ECC frame shows better post yield performance due to its ductile behaviour material and the usage of new reinforcement in ECC frame, it showed slow and steady energy dissipation as compared to control frame and FRP retrofitted frame indicated towards better results.

4.5.3 Stiffness Degradation

Many structural components and systems will exhibit some level of stiffness degradation when subjected to cyclic loading. This is especially true for reinforced concrete components subjected to several large cyclic load reversals. Stiffness degradation in reinforced concrete components is usually the result of cracking, loss of bond, or interaction with high shear or axial stresses. The level of stiffness degradation depends on the characteristics of the structure (e.g., material properties, geometry, level of ductile detailing, connection type), as well as on the loading history (e.g., intensity in each cycle, number of cycles, sequence of loading cycles). Sudden or rapid degradation in stiffness showed the

brittle performance while low rate of stiffness degradation shows ductile performance. The comparative curve of all the specimens showed that the loss of yield stiffness was high in control specimens than ECC and retrofitted specimens . This was because of the ductile behaviour of ECC as compared to RCC and retrofitted frame shows lesser stiffness due to yielded reinforcement in the RC frame.

In the damaged RC frame, the surface concrete in the impaired location was thoroughly removed and replaced with epoxy putty and the internal cracks were injected with epoxy. But the reinforcement in the hinge region was not replaced and hence all the hinge regions were critically strengthened using external FRP reinforcement. Thus, the FRP retrofitted specimens showed higher loss in initial and yield stiffness than control frame. But the post elastic stiffness degradation was comparatively low in retrofitted frame as compared to control frame. It illustrates the influence of external FRP strengthening in non-linear performance.

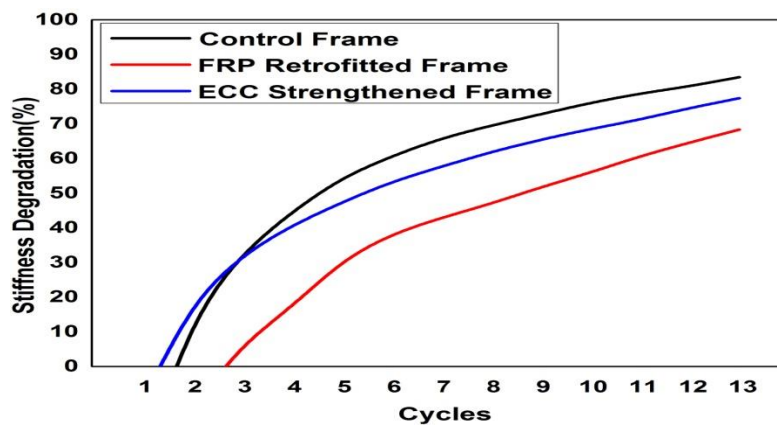


Figure 4.18. Stiffness Degradation in different frames

To evaluate the inelastic behaviour of the control, retrofitted and ECC replaced frames the stiffness degradation is presented in Figure 4.18. The rate of change of stiffness degradation over the post-elastic range is the measure of inelastic performance of the component. Sudden or rapid degradation in strength and stiffness showed the brittle performance while low rate of stiffness degradation shows ductile performance. The comparative curve of the all frames indicates that the loss of yield stiffness was higher in control specimens. In the damaged RC frame, the concrete in the impaired location has been thoroughly removed and replaced with fresh composites and epoxy injected to fill the internal cracks but the reinforcement in the hinge region was not replaced. Thus, the ECC frame specimens show higher loss in initial and yield stiffness than control frame. But the post

elastic stiffness degradation was comparatively low in ECC frame than control frame. It illustrates the influence of ductile behavior of ECC in improving the non-linear performance.

4.5.4 Crack pattern

Predominant cracking was observed in the first floor beam-column joint region whereas the second floor and roof had no evidence of cracking because of the soft storey effect. In the ground floor, the witnessed horizontal crack below the joint region reduced the stiffness and restricted the deflection capacity. As a result, the cracks and higher lateral displacement of concrete occurred in the joint region. The effect of seismic detailing supports the RC frame during deflections but the brittle nature of concrete exhibits concrete crushing failure at the foundation hinge region. There was no severe damage in the infill but some horizontal sliding cracks were noticed in the infill at the mortar joints.

In retrofitted frame, the internal cracks were injected with epoxy and after a thorough surface preparation, FRP wrapping was done. Thus, the failure pattern of the retrofitted frame was different from control frame. The FRP strengthening at the column hinge location and beam-column joint portion stiffer and hence failure was concentrated in the unstrengthened region. Figure 4.13(a-h) shows the cracks below and above the strengthened portion followed by FRP rupture in few locations. The primary failure was because of the horizontal crack below the FRP strengthened region. Thus, the ultimate deformation of the strengthened frame was identical with the control frame.

In the ECC frame, all the columns were replaced with ECC after removing the damaged concrete with beam and slab up to 300mm from both sides. Figure 4.14(a-d) shows the behaviour after damage. Fine cracks and inclined cracks appeared throughout the column length with the increase of deflection proving the effectiveness of ECC in bridging the cracks and in providing ductility. With higher deflection, there was no evidence of brittle shear failure.

4.6 Closing Remarks

It can therefore be concluded from these results of microfibers in the composite acts as a micro-level reinforcement to resist early crack growth. It contributes to the improvement of the energy dissipation capacity and the rigidity maintenance capacity. so resulting in ECC repaired frame showing better results followed by the FRP retrofitted frame. The FRP retrofitted frame shows improvement even with the old reinforcement steel bars as compared to control frame. It clearly points towards the efficacy of ECC in resistance to deflection that

ECC can be used in the plastic hinge region of RC frame to restore the damaged structure into a perfect form with improved damage tolerance capacity, inelastic rotations, energy dissipation capabilities and stiffness retention capacities. Micro-fibers in the composite act as crack arrestors.

CHAPTER 5

VIBRATION MONITORING AND MEASUREMENTS

5.1 GENERAL

Actual structures are made up of components that have a limited amount of stiffness, mass, and imperfect energy transfer properties. They can be characterized by physical properties of the structure such as Young's modulus, shear modulus, bulk modulus and mass distribution. When an external force is applied anywhere on the component, a reaction is generated. The magnitude of the response is proportional to the magnitude of the force for a linear system, but the direction of the response depends on the physical properties of the structure and components of the force (Burgueno et al., 2001, Ramsey, 1983). In the case of multiple inputs, the principle of superposition applies for linear systems such that responses from two or more inputs may be added vectorially. As a result, the application of external or internal loads during operation of the structure results in non-zero response and under certain conditions results in very high amplitude vibrations at certain frequencies or instabilities of static or dynamic monitoring the structural state.

It is important to be able to accurately predict these reactions, instabilities, or large amplitude vibrations. The common goal of vibration monitoring is to predict the response to a particular force in various damage states (Baghiee. et. al., 2009). For structures, the specific frequencies at which resonance amplitudes occur are described as "natural frequencies" of the structure. These frequencies and the corresponding distribution of the amplitude are global properties. Amplitude distributions in natural frequencies and vibration modes are referred to as natural vibration modes.

This chapter describes the basics of the theory and the prediction of the response to vibrations as well as various parameters used to model damage. The vibration monitoring technique is also applied to brick-infilled RC frames to refer to damage in different stress states. In addition, measurements are performed on infilled frames at different damage levels and repaired with GFRP / CFRP and ECC.

This chapter highlights the effectiveness of vibration monitoring techniques to assess global damage in various types of control and repaired RC frames.

5.2 VIBRATION MEASUREMENT TECHNIQUES

The vibration responses of the structures are related to the structural weaknesses associated with the resonance behaviour, for example, natural frequencies are excited by operating forces. It can be shown that the complete dynamic response of a structure in a certain frequency range can be considered as a set of single vibration modes (He and Fu, 2001, Ramsey, 1983). Each result has a characteristic natural frequency, damping and mode shape. By using these modal parameters to model the structure, problems such as specific resonances can be investigated and solved.

- i. "Resonant frequency" or "natural frequency"
- ii. "Structural damping"
- iii. "Frequency Response Functions" (FRF)
- iv. Mode shapes

Modal parameters can be calculated from a series of frequency response measurements between a reference point and several measurement points. Such points are generally referred to as degrees of freedom (DOF). Modal frequencies and damping ratio are found in all frequency response measurements in the structure, except those where the excitation or response measurement is in a position where the displacement is zero. Therefore, these two parameters are called "global parameters" (Formenti, 2000).

Frequency response measurements are facilitated by the use of a fast Fourier signal two-channel analyzer. The excitation force is measured by accelerometer and the resulting signal is fed to one of the inputs. When using a vibration exciter, a generator module must be installed in the analyzer. To measure the vibration of the structure, the reaction is measured by an accelerometer and the resulting signal is fed to another input. Consequently, the frequency response represents the acceleration of the structure, since the measured quantity is the complex ratio of the acceleration to the force in the frequency range (FRF). There are two ways to measure the vibration response. For the excitation of the percussion hammer, the response position of the accelerometer is first set and used as a reference position. The impact with hammer is used to stimulate the structure in the position corresponding to the model DOF. Second, for excitation of the vibrator, the excitation point is set and used as the reference point of the accelerometer moves across the structure. For structures number of degrees of freedom, additional response channels can be added to allow easier and faster measurements.

5.2.1 Determination of modal frequencies

In a one-degree-of-freedom (SDOF) structure, the equation for motion in all cases is an ordinary second-order differential equation. The natural resonance or frequency of the system is defined by

$$\omega_n = \sqrt{\frac{k}{m}} \quad (5.1)$$

Where "k" is the stiffness of the structure and "m" is the structure's mass. In the multi-degree-of-freedom system, the values of each resonant frequency are evaluated by the intrinsic solution (Paz, 1997, Newmark, and Rosenblueth, 1971).

In the laboratory test, the frequency with which it occurs is determined by the analyzer software. A resonance frequency is identified as a peak in the vibration response of the structure, such as in the FRF diagram, the displacement diagram, the velocity diagram or the acceleration diagram. The frequency resolution determines the accuracy of the frequency measurement. Greater accuracy can be achieved by decreasing the frequency range of the band-based measurement or by performing a zoom measurement about the frequency of interest.

5.2.2 Determination of modal damping

There are two well-known classical methods for the determination of the viscous damping at resonance namely method of medium power and the method of logarithmic decrease. This work used the latter method because it is easier and more user-friendly.

Logarithmic Decrement method

The logarithmic decrement is a measure of the rate of decomposition of the free vibration. It is probably the simplest and most widely used experimental damping technique. The original idea of using the vibration-free case of a viscous damped system was to determine the frequency information of musical tones. The equation describing the damped system of free vibration represented by the response in Figure 5.1 is:

$$y(t) = Ae^{-\zeta \omega t} \sin(\omega_d t + \phi) \quad (5.2)$$

This equation represents a pseudo-harmonic motion with an exponentially decremental amplitude and a phase angle (Paz, 1997, Key, 1988, Newmark, and Rosenblueth, 1971). It is clear that the vibrations of the damped structures normally disappear over time at

a rate that can be used as a measure of the amount of damping. In the logarithmic decrease method, the amplitude of movement at the beginning of a cycle and the amplitude at the end of the cycle are measured as a function of the damping period. From the Euler formula of the poorly padded case, therefore, the expression result is.

$$\frac{A_1}{A_2} = e^{\zeta \omega_d \tau_d} \quad (5.3)$$

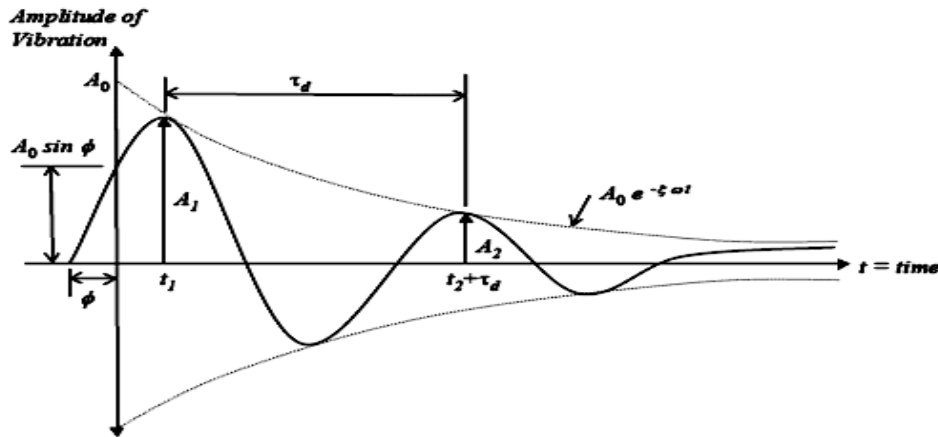


Figure 5.1 Free-vibration response of an under-damped system

For small damping ($\zeta < 0.2$) the damping ratio can be obtained from the equation

$$\zeta = \frac{1}{2\pi} \ln \frac{A_1}{A_2} \quad (5.4)$$

Significant advantage of the traditional logarithmic reduction system is that equipment and instrumentation requirements are minimal and free vibration can be introduced by many convenient and appropriate methods. i.e. simple vibration hammers or environmental excitation.

5.2.3 Determination of the Mode Shape

The mode shape is a property of vibration of a multiple degrees of freedom (MDOF) system. The natural mode of the systems without damping depends on the modal frequency. An example of a 3-DOF system is shown in Figure 5.2. Before considering the properties of frequencies and modes, it is useful to consider some of the properties of stiffness and mass matrices found in structural dynamics (Clough and Penzien, 1975). The stiffness matrix K and the mass matrix M were related to the strain energy and the kinetic energy, respectively. It was seen that both K and M were symmetric.

The free vibration equation of an undamped MDOF system, written in symbolic matrix form is

$$\mathbf{M}\ddot{\mathbf{x}} + \mathbf{K}\mathbf{x} = 0 \quad (5.5)$$

Where $\mathbf{M}$ and $\mathbf{K}$ are $N \times N$ matrices and $\mathbf{x}(t)$ is the corresponding $N \times 1$ vector of physical or generalized displacement coordinates.

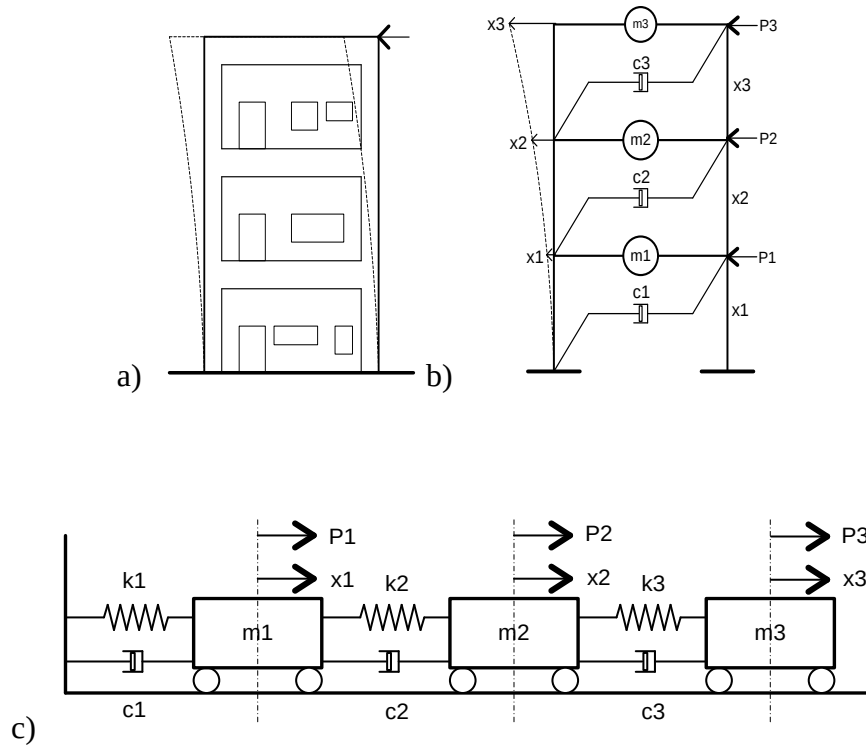


Figure 5.2 A model of a three degree of freedom system

The N^{th} order algebraic Eigen-value problem is

$$[\mathbf{K} - \omega^2 \mathbf{M}] \mathbf{X} = \mathbf{0} \quad (5.6)$$

For there to be a non trivial solution, it is necessary that

$$\det(\mathbf{K} - \omega^2 \mathbf{M}) = 0 \quad (5.7)$$

This is called the characteristic equation. When the determinant is expanded, there results a polynomial equation of degree ω^2 , whose roots are the Eigen-values, or squared natural frequencies, ω_r^2 . Corresponding to each Eigen-value ω_r^2 there will be an eigenvector, or natural mode, Φ_r , where

$$\Phi_r = \begin{Bmatrix} X_1 \\ X_2 \\ \vdots \\ X_N \end{Bmatrix}_r \quad \text{where } r = 1, 2, \dots, N \quad (5.8)$$

The modes are determined only within a constant multiplier. Therefore, the modes can be scaled in any convenient way.

5.3 Dynamic Equation of Motion

In vibration monitoring technology, the dynamic behavior of the structure is the goal of the tests. The dynamic properties, which include natural frequency, mode shape, and viscous damping, are products of mass, stiffness, material's resistance, joints, structural geometry, and damage state of the structure. The difficult task is to monitor the damaged condition of the structures. Damage to a structure can alter mechanical properties such as stiffness and modal parameters. The extent of change depends on both the severity and the position of the deterioration (Paz, 1997, Clough and Penzien, 1975). A single damage event will affect all vibration mode differently. This effect is in many ways the basis for the identification of damaged limbs, as it can be predicted. If the damage is expressed as a reduction of the cross-sectional properties, then a disturbance of the equation of motion may result in the quantitative relationships between the damage and the resulting changes in the modal parameters. The equation of motion for an intact SDOF system is

$$m\ddot{x} + c\dot{x} + kx = p(t) \quad (5.9)$$

where " m " is the mass, " c " is the damping coefficient, k is the stiffness coefficient and $p(t)$ is external force, and $\ddot{x}$, $\dot{x}$ and x are acceleration, velocity, and deflection, respectively. Flexural element, the stiffness force can be obtain as

$$kx = EI \frac{\partial^4 x}{\partial y^4} \quad (5.10)$$

where E and I are elastic modulus and section moment inertia of intact element, and x is the flexural deflection along the distance y . In RC structures, cracks may often reduce stiffness without loss of mass. Lee and Shin (2001) suggested an approach to determine damaged status of structural element. This approach uses damage index to represent the damage induced change in stiffness by the degradation of elastic modulus as follows

$$E_d = E(1 - DI) \quad (5.11)$$

where E_d is the effective elastic modulus in the damaged state and DI is the damage index which may characterize the state of damage. Damage index equal to zero indicates the intact state, while damage index equal to one indicates the complete rupture of material due to damage. Assume that the damages in an element are uniform through the thickness of element. Then, the dynamic equation of motion for the flexural element in the damaged state as follows

$$m\ddot{x} + c_d\dot{x} + (1 - DI)kx = p(t) \quad (5.12)$$

where c_d is the damage damping coefficient. Damping brings the system to rest in finite time.

5.3.1 Impulse Excitation Response

The excitation force is important to conducting accurate vibration testing. The excitation force can be random, sinusoidal, periodic or transient ones. The decision on excitation is based on the optimum method for accurate measurement data that suit test objectives and the structure. The time domain force signal for impact excitation is a pulse with uncontrollable frequency contents. Impact excitation is convenient to use and very portable for field and laboratory test (He and Fu, 2001). It because no physical connection between the excitation and the structure. Impact test avoids the problem of interaction between them. Impulse excitation frequently encounters a force of very large magnitude that acts for a very short time but with a time integral that is finite (Craig and Kurdila, 2006). Impulsive force by the notation P is given by

$$P = \int p(t)dt \quad (5.13)$$

An impulsive force of magnitude P/t_d with a time duration of t_d is shown in Figure 5.3. Under free vibration, it can be found that the undamped response of SDOF system with initial conditions $x(0)$ and $\dot{x}(0)$ behaved according to the equation

$$x(t) = \frac{\dot{x}(0)}{\omega_n} \sin \omega_n t + x(0) \cos \omega_n t \quad (5.14)$$

Hence, the response of this system initially at rest and excited by an impulse $\bar{P}\delta$ is

$$x(t) = \frac{P}{m\omega_n} \sin \omega_n t \quad (5-15)$$

When damping is present, it can be started with the free vibration equation with initial conditions $x(0)$ and $\dot{x}(0)$ behaving according to the equation 5.16. The response of this system will be

$$x(t) = e^{-\zeta \omega_n t} \left(\frac{\dot{x}(0) + \zeta \omega_n x(0)}{\omega_n \sqrt{1-\zeta^2}} \sin \sqrt{1-\zeta^2} \omega_n t + x(0) \cos \sqrt{1-\zeta^2} \omega_n t \right) \quad (5-16)$$

Hence, the responses of this system initially at rest, $x(0)=0$ and $\dot{x}(0)=0$ and excited by an impulse P for $t > 0$ are

$$\ddot{x}(t) = \frac{P}{m\omega_d} e^{-\zeta \omega_n t} \left\{ \left[(\zeta^2 \omega_n^2 - \omega_d^2) \sin \omega_d t \right] - \left[2\zeta \omega_n \omega_d \cos \omega_d t \right] \right\} \quad (5-17)$$

$$\text{where } \omega_d = \frac{2\pi}{\tau_d} = \omega_n \sqrt{1-\zeta^2} \quad (5-18)$$

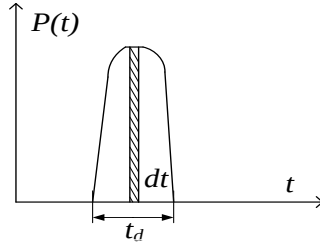


Figure 5.3 Short duration impulse excitation force

5.3.2 Relationship between Time Domain and Frequency Domain

The theoretical foundation employs FFT to analyze transient response, specifically the Fourier transform. The Fourier transform converts a function $y(t)$ defined over $-\infty < t < \infty$ into a function $Y(\omega)$, where ω is a frequency parameter (Craig and Kurdila, 2006). The definition is

$$Y(\omega) = \int_{-\infty}^{\infty} y(t) e^{-i\omega t} dt \quad (5-19)$$

If the transform $Y(\omega)$ is known, the corresponding time function may be found by applying the inverse Fourier transform,

$$y(t) = \frac{1}{2\pi} \int_{-\infty}^{\infty} Y(\omega) e^{i\omega t} d\omega \quad (5-20)$$

The basic properties of a Dirac delta function circumvent this difficulty. Given that the integral extend the limits of integration. The transform is proportional to a Dirac delta function. Let's take the argument of this function to be $\omega \pm \Omega$ because the finite value occurs at $\omega \pm \Omega$. The inverse transform of $\delta(\omega \pm \Omega)$ is

$$p(t) = \frac{1}{2\pi} \int_{-\infty}^{\infty} P(\omega) e^{i\omega t} d\omega = \frac{1}{2\pi} \int_{-\infty}^{\infty} \delta(\omega \pm \Omega) e^{i\omega t} d\omega = \frac{1}{2\pi} e^{\pm i\Omega t} \quad (5-21)$$

This establishes the transform $e^{\pm i\Omega t}$ pair as $2\pi\delta(\omega \pm \Omega)$. Another important transform pair is the one for an impulse in the time domain. The integration property of a Dirac delta function yields

$$P(\omega) = \int_{-\infty}^{\infty} p(t) e^{-i\omega t} dt = \int_{-\infty}^{\infty} \delta(t - \xi) e^{-i\omega t} dt = e^{i\omega\xi} \quad (5-22)$$

The conjugation transform of a Dirac delta function, $\delta(t - \xi)$ is $e^{i\omega\xi}$. This solution concludes that an impulse excites all frequencies with equal magnitude, with all transformed values being real if the impulse occurs at $\xi = 0$.

5.3.3 Frequency Response Functions

The measurement for experimental modal analysis is to acquire frequency response function data from a test structure. For some time-domain analysis it is to obtain either the free decaying impulse response or the response due to ambient excitations. While, there are a variety of methods available to carry out measurement. As a result, the data contains a group of FRF that can be used for modal analysis of the structure (Maia et al., 2003; Ramsey, 1983). There are different types of excitation force on structure. Theoretically, the type of force does not matter as the FRF is defined as the ratio between the responses and force (He and Fu, 2001).

For a typical single input and single output system, the time waveform of an excitation force to the system is $f(t)$ and the time waveform of the system response is $x(t)$. The Fourier spectra for input, $f(t)$, and output, $v(t)$, are $F(\omega)$ and $X(\omega)$, respectively. The frequency response function, $H(\omega)$, is defined as the ratio of the Fourier transforms of the system output $X(\omega)$ to the system input $F(\omega)$ and given by:

$$H(\omega) = \frac{\text{Output}}{\text{Input}} = \frac{X(\omega)}{F(\omega)} \quad (5-23)$$

It is required for FRF that the input signal $F(\omega)$ is nonzero at all frequencies of interest (Craig and Kurdila, 2006). Depending upon the selected parameters for the system input and output, the representations of FRF are as follows:

- (i) Accelerance (g/lbf or g/N) = Acceleration / Force
- (ii) Mobility (in./lbf sec or m/N sec) = Velocity / Force
- (iii) Dynamic Compliance (in./lbf or m/N) = Displacement / Force

where g is acceleration due to gravity, and g is equal to 386 inches./sec² or 9.81 m/sec². The system output is given by acceleration g and input is given by force N .

In MODF system under harmonic excitation, the mode superposition for the steady state response $\mathbf{x}(t)$, based on the modes $\mathbf{\check{s}}_r$ of the undamped system, is given by

$$\mathbf{x}(t) = \sum_{r=1}^N \frac{\mathbf{\check{s}}_r \mathbf{\check{s}}_r^T \mathbf{P}}{\mathbf{K}_r} \frac{1}{\left[1 - (\omega / \omega_r)^2\right] + i(2\zeta_r \omega / \omega_r)} e^{i\omega t} \quad (5-24)$$

where ω is the forcing frequency in rad/s, which is varied over the frequency range of interest in the modal test. The steady state displacement response at coordinate j due to harmonic excitation of unit magnitude only at coordinate k is called the frequency response function for the response at j due to excitation at k (Craig and Kurdila, 2006). When the FRF deals with displacement per unit force, it is called a receptance. The receptance has the form

$$H_{jk}(\omega) = H_{x_j / p_k}(\omega) = \sum_{r=1}^N \frac{\phi_{jr} \phi_{kr}}{K_r} \frac{1}{\left[1 - (\omega / \omega_r)^2\right] + i(2\zeta_r \omega / \omega_r)} \quad (5-25)$$

The mobility (i.e., velocity output at j per unit force input at k) for MDOF systems with modal damping is given by

$$H_{v_j/p_k}(\omega) = i\omega \sum_{r=1}^N \frac{\phi_{jr}\phi_{kr}}{K_r} \frac{1}{\left[1 - (\omega/\omega_r)^2\right] + i(2\zeta_r \omega/\omega_r)} \quad (5-26)$$

The accelerance (acceleration output at j per unit force input at k) is given by

$$H_{a_j/p_k}(\omega) = -\omega^2 \sum_{r=1}^N \frac{\phi_{jr}\phi_{kr}}{K_r} \frac{1}{\left[1 - (\omega/\omega_r)^2\right] + i(2\zeta_r \omega/\omega_r)} \quad (5-27)$$

and the phase (lag) angle $\alpha(\omega)$ is given by

$$\tan \alpha_r(\omega) = \frac{2\zeta_r(\omega/\omega_r)}{1 - (\omega/\omega_r)^2} \quad (5-28)$$

In modal testing, accelerance can measure the response in a wide frequency range by using accelerometer. Therefore, in structural health monitoring, the accelerance is more convenient than mobility and dynamic compliance (Maia et al. 2002; Ramsey, 1983). Accelerance is the FRF used most commonly and the forcing frequency is usually given as f hertz = $\omega/2\pi$. Obviously, FRF contain information about natural frequencies, ω_r , damping ratios, ζ_r and mode shapes, ϕ_r .

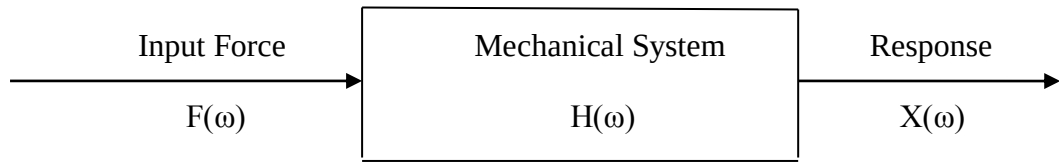


Figure 5.4 Single input and single output system (He and Fu, 2001)

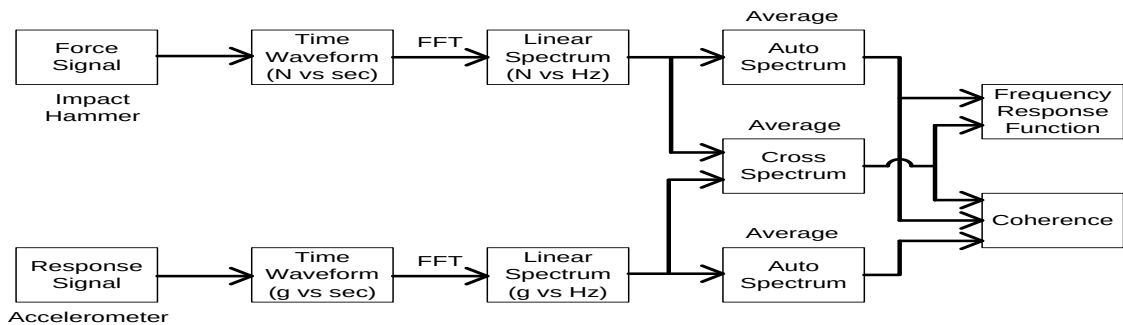


Figure 5.5 Block diagram of signal process

The FRF is measured by sampling both the excitation force and the response signals simultaneously, as shown in Figure 5.4. The time waveforms detected by the system's excitation and response include amplitude and relative phase information. An excitation force instrument was connected to the force input channel of an FFT spectrum analyzer. In general, the excitation force instrument was directly connected to the force channel because the FFT analyzer powers an integrated piezoelectric circuit. The response signal was measured with an accelerometer connected to the response input port of the FFT analyzer. The block diagram of the signal process is shown in Figure 5.5. The spectrum analysis is defined as the transformation using the Fourier transform from a time domain signal representation into a frequency domain representation. A discrete periodic time signal is to be converted into a discrete periodic frequency spectrum for use with a digital computer, for which the FFT has been developed. The modal test is the process by which FRFs are measured and used to estimate the physical properties of the structure under test. There are three main types of modality tests: from Single Input to Multiple Output, from Multiple Input to Single Output and from Multiple Input to Multiple Output.

5.4 MEASUREMENT CONSIDERATIONS

5.4.1 Selection of Impact Hammer

To measure the response to vibration in this study, one must first configure a suitable impulse hammer, transducer, and FFT analyzer. The mass and hardness of the hammer tip may be selected to provide the desired magnitude and duration of the force pulse. Hammers can be built with weights from a few grams to several kilograms. They can cover a wide frequency range. In general, heavier hammers are more effective at lower frequencies. However, if the hammer is too heavy and the structure is too weak, then the hammer may overload or rebound to cause a double strike (Formenti, D., 2000). On the other hand, if the hammer is too light, there will not be enough energy imparted to the structure under test.

The hammer tip also affects the impact. A steel tip provides a short-lived force with large amplitude. A rubber tip ensures a long-lasting power with low amplitude. A plastic point lies somewhere between steel and rubber (Dincal, 2005, Barecchia., Sf.). In the frequency range, a rubber tip offers a narrow frequency range, a steel tip a wide frequency range and a plastic tip is somewhere in between. The specimen also plays a role in the selection of the tip, since the structure under test should not be permanently damaged. Impact Hammers are supplied with calibration values for each hammer / tip combination. The

appropriate peak may be selected based on the desired frequency range and the mass of the structure under test.

The strongest influence on the selection of the appropriate parameter is the frequency with which the measurement is important.

5.4.2 Selection of sensors / Transducers

As a rule, path measurements record large signals at low frequencies, while acceleration measurements are effective at high frequencies. A suitable vibration sensor or transducer means:

- i. Displacement detected by a proximity sensor with a displacement-proportional output signal
- ii. Speed detected by a seismic transducer
- iii. Acceleration detected by an accelerometer

Accelerometers are widely used transducers for testing dynamic properties. The weight of a selected accelerometer is important because it can change the dynamic properties of the structure being tested. Dynamic mass loading is the change in the physical properties of the structure caused by the addition of the mass of the accelerometer. In this test, multiple weight accelerometers were used to minimize the effect of dynamic mass loading. As explained in chapter 3.4, acceleration positions are measured in the modal of each floor of the frame. Accelerations measured with PCB-393C with a sensitivity of 101.9 mV / (m / s²) (Model 11199, 11197 and 11198, respectively).

5.4.3 Mounting of Sensors

There are several ways to attach the accelerometer to the surface of the test structure, such as: stud attachment, magnetic attachment or the use of a thin wax layer (Formenti, D. 2000). To minimize the loss of high frequency signals in the transmission path, the contact surface must be flat, smooth, clean and free of rust and color. Screw mounting is the most reliable method and allows better transmission of high frequency signals. Epoxy wax is also a convenient method of temporary installation. The test results show that on clean, flat surfaces a frequency response can be achieved which corresponds to the installation with a thin layer of wax. As the thickness of the wax at the interface between the accelerometer and the sample increases, the first resonance of the system decreases, thereby limiting the frequency response of the fixturing. To get the best possible response on flat surfaces, use as little wax

as possible on the interface and press the accelerometer firmly against the test structure in a circular motion to assemble it.

5.4.4 Data Acquisition and Display

The FRF measurement parameters can be set before acquiring measurement data, such as frequency range, number of signal lines, unit type, and so on. In addition, the corresponding parameters for the force channel and the response channel can be selected. The FRF is complex and there are three sizes, frequency, size and phase. This corresponds in frequency to the real part and the imaginary part. The interpretation of impact test data can be significantly improved by using a suitable graphic display. Typical display formats include time waveforms and frequency spectra of impact force and system behavior, size, phase, real part and imaginary part of the FRF and the coherence function. To display the data, a linear or logarithmic display for FRF can be selected. However, only a linear amplitude screen is used for the time, phase, and coherence waveform functions.

5.5 FORCE WINDOW AND CHECK MEASUREMENT DATA

5.5.1 Force window

In general, it is desirable to have a forcing function that is short in time duration to provide the FRF for the structural system. This short duration signal in the time domain corresponds to broadband frequency content in the frequency domain. Since the impact force signal represents a very small portion of the digital record analyzed, even very low level noise may produce significant errors in the auto-spectrum of the force signal and FRF estimation (Formenti, D., 2000; Sek, nd.). This is especially true if the duration of the force signal is short relative to the total sample time. Such noise is not reduced by averaging with the usual FRF estimate because the noise is additive in the force auto-spectrum. Random noise can be greatly reduced by multiplying a force window by the force signal.

The force window has unity gain for the portion of the record which includes the force signal, and sets the remaining samples in the record to zero prior to Fourier processing. This procedure introduces no distortion as long as none of the force data are attenuated and the background noise is broadband in nature.

The captured time waveforms of both impact force and system response contain amplitude and relative phase information. The FRF and coherence function can be computed from the Fourier transforms of both impact and response signals.

5.5.2 Check bad impacts

Three types of bad impacts are common problems in impact testing, namely: overload, under load, and more than one impact (also referred to as double hit) (Formenti, D., 2000). Overloading is the most difficult problem to overcome in impact tests. When an overload occurs, the impact force signal is clipped, causing distortion in the measurement. In some cases, an anti-aliasing filter may smooth a clipped signal to make a distorted measurement look good. To detect overload problems, the unfiltered force signal should be checked before it passes through the anti-aliasing filter. In the FRF measurement, the overload force level can be set in unit of lb or Newton based on the experience of testing.

The opposite problem is under load due to low impact force level or improper gain setting. If the impact force is not adequate, the measured FRF does not represent the dynamic characteristics of the measured system. Therefore, under load needs to be checked before signal processing. The under load force level can be specified as a percentage of the overload limit. It is important to avoid more than one impact or impactor rebounds. If more than one impact occurs within the data record, the Fourier transforms of the pulses tend to cancel at certain frequencies, creating sharp notches in the force spectrum. This can cause significant errors in the FRF measurement at these frequencies due to a low signal to noise ratio in the force spectrum. Furthermore, the force window should not be used to eliminate secondary impacts from the force record prior to Fourier processing. In this case, the response will still include the effects of multiple impacts, thus resulting in an erroneous estimate of the FRF. The problem of a double hit can be detected by checking any secondary peak(s) within a data record. It is important to avoid impact hammer rebounds. If more than a single impact occurs within the data record, the Fourier transforms of the pulses tend to cancel at certain frequencies, due to a low signal to noise ratio in the force spectrum. Furthermore, the force window should not be used to eliminate secondary impacts from the force record prior to Fourier processing. In this case, the response will still include the effects of multiple impacts, thus resulting in an erroneous estimate of the frequency response function.

5.5.3 Check data quality

To determine the quality of time wave form of response data and the FRF estimation, it is not sufficient to know only the relationship between input and output. The question is whether the system output is caused by the system input or by other factors. For example, noise and nonlinear effects can cause large outputs at various frequencies, inducing errors in

estimating the FRF (Baghiee et al., 2009; Ooi et al., 2004; Formenti, D., 2000). The influence non-linearity and the degree of noise contamination in the FRF is measured by calculating the coherence function. Figure 5.14 shows bad response data of time response record and FRF plot, respectively.

The coherence function can be calculated from the averaged auto-spectrum and cross spectrum of the impact force and the system response. The coherence function is dimensionless and its value is always between 0 and 1. If the coherence is equal to 1 at any specific frequency, the system is said to have perfect causality at that frequency. In other words, the measured response power is caused totally by the measured input power and the system is linear. If the value of coherence function is equal to 0, the two signals are totally uncorrelated. A coherence value less than 1 at a given frequency indicates that contamination exists in the two signals in terms of noise and nonlinear effects. Generally, the measured data are good or acceptable if the coherence value is greater than or equal to 0.75. In applications, low coherence at anti-resonance frequencies is not generally a cause for concern about data quality. At such frequencies the machine or structural response is very small and may be near the noise floor of the instrumentation. Thus, the low coherence merely indicates a reduced signal-to-noise ratio. On the other hand, one should be aware that the coherence would always be unity for single impact estimation.

5.6 RESULTS AND DISCUSSION OF DYNAMIC MEASUREMENTS

The detection of structural damage using the information contained in the vibration signatures is a useful technique for monitoring the state of the structure without destroying it (Kanwar et. al., 2006, Burgueno. et. al., 2001, Formenti, 2000). In this experimental set up, changes in the structural dynamics of the damaged frame model were investigated using the impact hammer excitation test with the software program OROS-35. An accelerometer was installed on each floor to measure transient response. The program was configured for a free zoom measurement with a frequency range of 0 to 100 Hz. This frequency range included the three main equalization modes of all frame models. After applying two load steps, the dynamic properties represented records in OROS-35 based on a linear analysis configuration. These records included the hammer trigger diagram, the timing chart, and the frequency response function diagram that indicated the amplitude of the vibration along with the frequency.

In the molding method based on the identification of damage, FRF is a useful tool (Baghiee et al., 2009, Dincal, 2005) because it relates to three main structural parameters: mass, damping, and stiffness. Although it is difficult to calculate the FRF by the classical method, the modal analysis always takes the form of the structural mode from the measured FRF (He and Fu, 2001). To avoid this complicated effect, many of the researchers involved mention only the measured baseline FRF, as the higher FRF mode is sensitive (Cakar and Sanliturk, 2003 and Kanwar et al., 2006). The FRF-based injury index is one of the most effective ways of monitoring progressive damage to structures. It characterizes the global structural damage by measuring the variation or change of the fundamental frequency and the size of the FRF of a system. Therefore, the size of the measured FRF is presented and discussed in this section. This chapter presents all dynamic recordings of experimental frames. Based on the change in dynamic properties, the FRF damage index, DI_{FRF} was used to monitor the harm of experimental frame models.

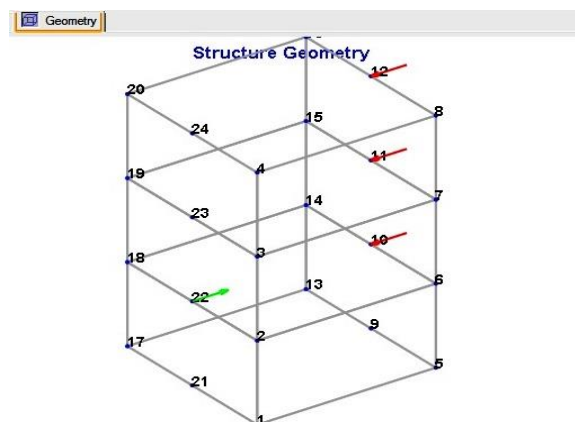
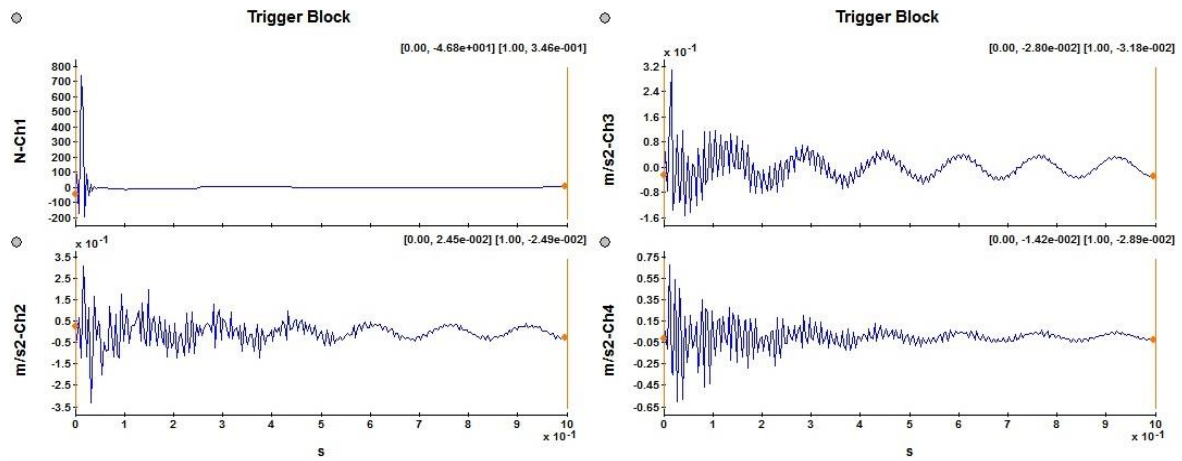
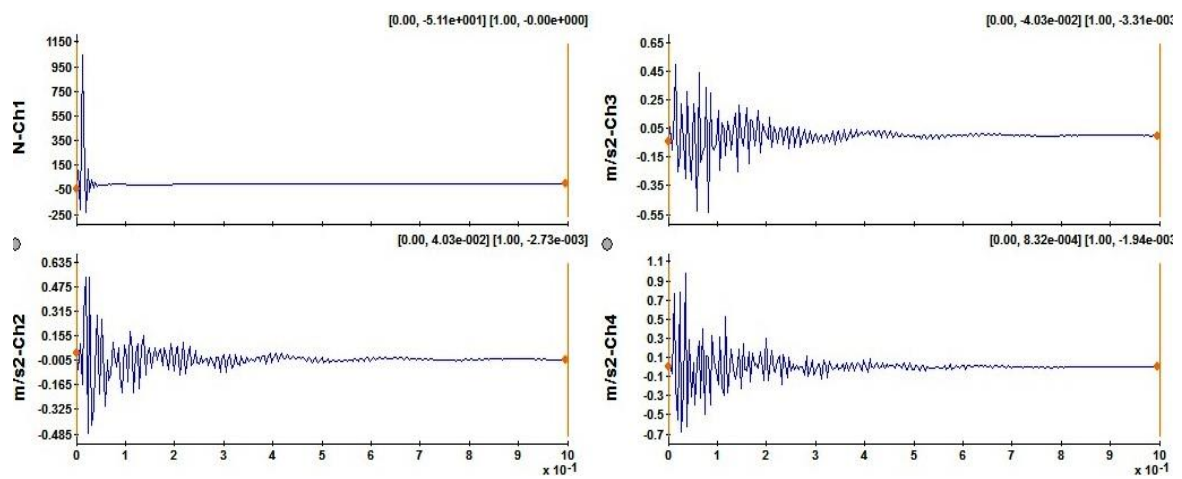


Figure 5.6 Geometry of frame model in OROS -35

The dynamic properties of a structural system can be estimated from the Frequency Response Function (FRF). The frame geometry has been inserted into OROS -35. A hard rubber tip hammer was used to stimulate the frame at node no 22,23 and 24 respectively and then corresponding reaction was measured through accelerometer fixed at node 10,11 and 12 as shown in Figure 5.6. The impact force plot given by all accelerometers (trigger block) of healthy and damaged frames are shown in Figure 5.7 in which first plot of trigger block shows the impact force signal and corresponding response signal. Which transform into time wave form through linear spectrum to auto spectrum coherence of frame was determined. Through cross spectrum of force signal and response signal frequency response function of the frame was calculated. The Time History plot is shown in Figure 5.8.



(a) Healthy Frame



(b) Damaged Frame

Figure 5.7 Trigger block of healthy and damaged frame

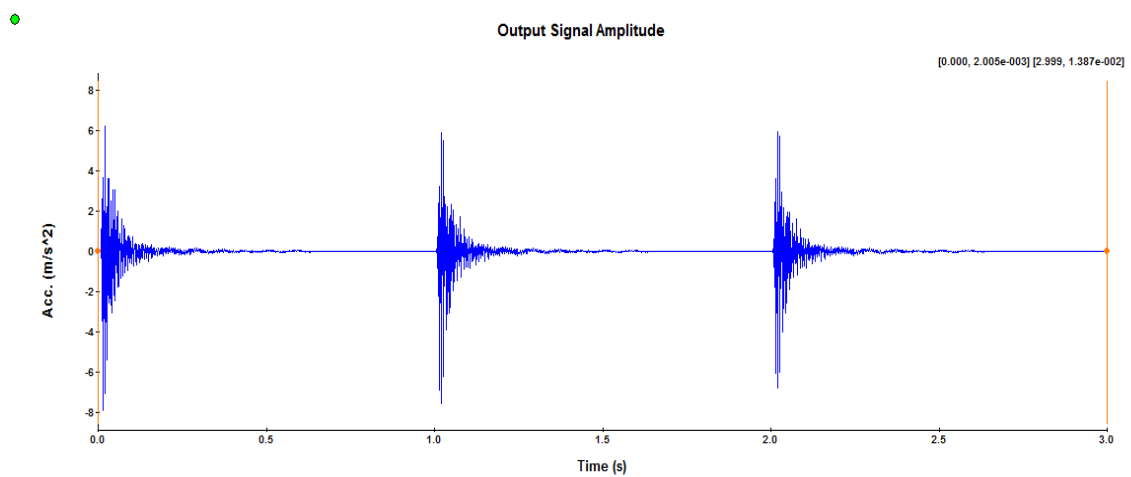
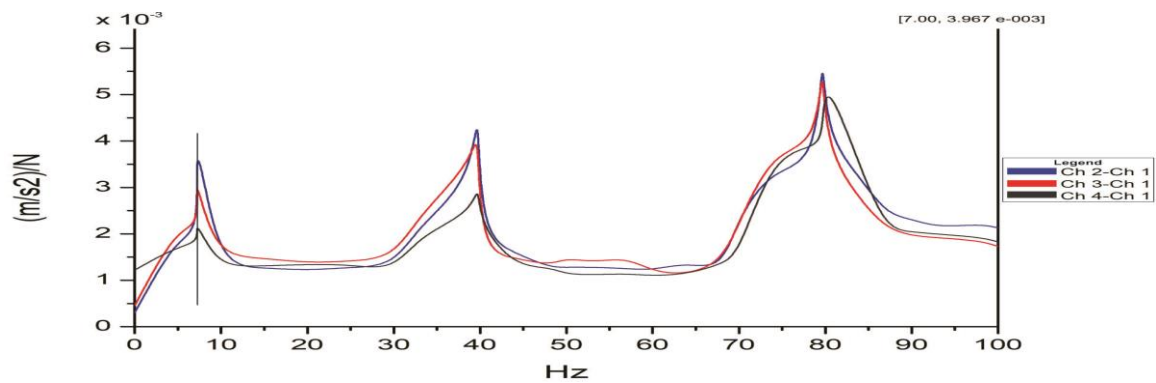


Figure 5.8 Time History Plot

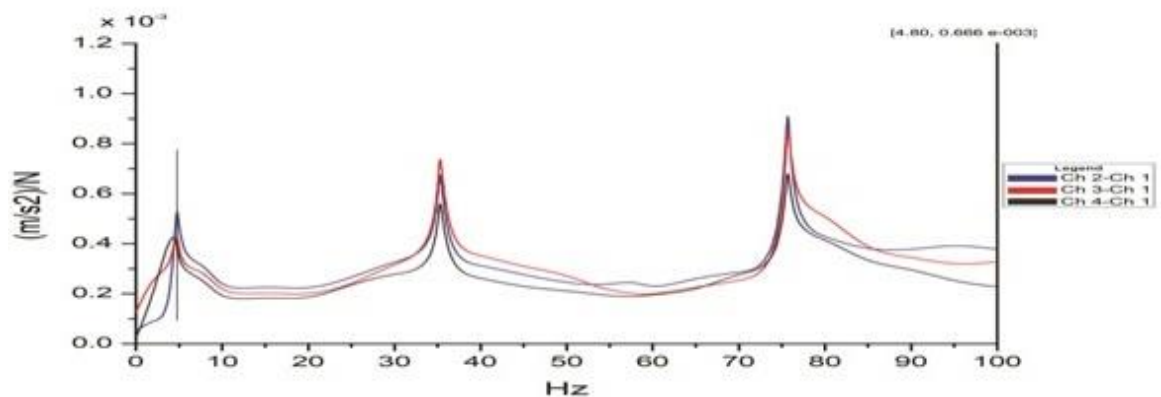
In this study, the impact hammer and accelerometer has been used to detect the FRF response, and the logarithmic decrement defines the equivalent damping response. when load applied to the frame upto a controlled displacement, damage occurred in the frame. When damage started in the frame Frequency and FRF starts decreasing and damping starts increasing.

5.6.1 Control frame

The dynamic properties of a structural system can be estimated from the Frequency Response Function (FRF). The FRF-based injury index is one of the most effective ways to monitor health. Changing the fundamental frequency and size of a system's FRF defines the damage behavior. In this study, an impact hammer and an accelerometer were used to detect the FRF, and the logarithmic decay defines the equivalent damping behavior. A hard rubber tip hammer was used to stimulate the structure.



(a) Healthy Frame



(b) Damaged Frame

Figure 5.9: Frequency Response Function for control brick infilled frame

The dynamic characteristics of the structure were automatically recorded using the OROS-35 software using the Fast Fourier Transform Spectrum Analyzer. The usual goal of vibration measurement is to predict the reaction when force is applied in different states of damage. The specific frequencies at which "resonance amplitudes," occur are called "eigen frequencies" of the structure. These frequencies and the corresponding distribution of the amplitude are global properties. Structural mode shapes are generalized from the problem of eigen value corresponding to natural frequencies.

Figure 5.9 shows the frequency response of a healthy and damaged image. Control samples showed a natural frequency of 7-4.8 Hz with a change in the damage rate of 0-0.832. The natural frequency decreased with increasing damage level and shows a reduction of the natural frequency by 31.4% at the failure stage. It was found that the size of the FRF decreased while the damping ratio increased with increasing deformation. Therefore, the control frame showed a decrease in FRF size of 83.2% and an increase in the damping ratio of 69.6%.

Table 5.1: Frequency, FRF, Damping Ratio and Damage Index of control frame

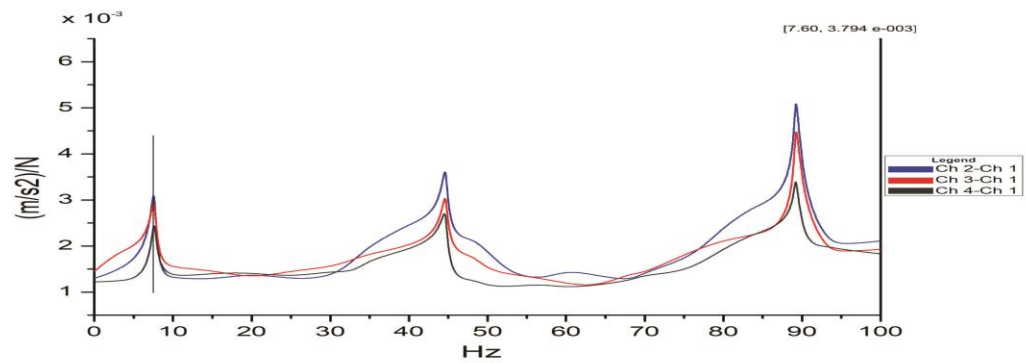
Cycle	Load (kN)	P/P(max.)	Natural frequency of each mode, Hz			FRF magnitude of each floor, 10^{-3} g/N			Damping Ratio	DI (1)	DI (2)	DI (3)
			$\omega 1$	$\omega 2$	$\omega 3$	FRF1	FRF2	FRF3				
0	0	0	7	39.8	79.7	3.967	5.038	6.262	0.250	0	0	0
2	35	0.69	6.9	38.4	79.1	3.618	4.632	5.738	0.258	0.087	0.080	0.083
4	45	0.89	6.7	37.2	78.4	3.324	4.193	5.24	0.321	0.162	0.167	0.163
6	47.9	0.95	6.4	36.8	77.7	2.741	3.491	4.38	0.351	0.309	0.307	0.300
8	49.5	0.98	5.9	36.3	76.8	2.077	2.641	3.283	0.373	0.476	0.475	0.475
10	50.5	1.00	5.2	35.9	76.2	1.367	1.757	2.143	0.385	0.655	0.651	0.657
12	46.6	0.92	4.9	35.4	75.9	0.872	1.016	1.295	0.412	0.780	0.798	0.793
13	42.3	0.84	4.8	35.2	75.7	0.666	0.916	1.159	0.424	0.832	0.818	0.814

In the initial state, it was observed that the natural frequency of the first, second and third modes was 7.0 Hz, 39.8 Hz and 79.7 Hz, respectively. It was observed that the structural damping based on the logarithmic decrease method was initially 0.25. The size of the FRF of the upper, second, and first floors was 6.262×10^{-3} g / N, 5.038×10^{-3} g / N, and 3.967×10^{-3} g / N, respectively.

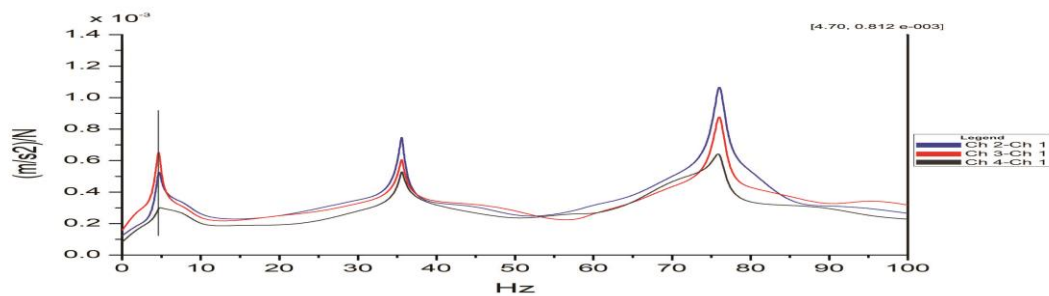
$^{-3} \text{ g} / \text{N}$ respectively. At initial cracking, the DI_{FRF} was 0.087 which corresponded to 35 kN load. The maximum damage frequencies of the first, second, and third modes were 4.8 Hz, 35.2 Hz, and 75.7 Hz, respectively. The upper floor FRF size was $1.159 \times 10^{-3} \text{ g} / \text{N}$, followed by the second and first floor with a size of $0.916 \times 10^{-3} \text{ g} / \text{N}$ and $0.666 \times 10^{-3} \text{ g} / \text{N}$. In the final state, the damping ratio was 0.424 and the DI_{FRF} was 0.832. The summary of stress, natural frequency, FRF, damage index and structural damping is shown in Table 5.1. Compared to the FRF of the upper and second floor, it was observed that the FRF of the first floor decreased. This shows the effect of the increase in floor mass on the floor's FRF, which is located immediately below it in a multistory building.

5.6.2 FRP Retrofitted Frame

Figure 5.10 shows that the adapted frame pattern has a similar frequency response. The observed natural frequency of the control varied from 7.6 to 4.7 Hz with a change in the rate of damage from 0 to 0.767. On the first floor, the natural frequency at the failure stage was reduced by 38.2%. This demonstrates the effectiveness of the gain technique used to restore the frame.



(a) Healthy Frame



(b) Damaged Frame

Figure 5.10: Frequency Response Function for FRP repaired brick infilled frame

There was a slight reduction in FRF size and the damping ratio showed a remarkable power dissipation of the FRP retrofitted frame over the control frame. The observed DI_{FRF} reduction of the FRP retrofitted frame was 78.60% reduction in FRF size and 68.48% increase in damping ratio as compared to control frame. The summary of the dynamic properties of the FRP retrofitted and the corresponding failure rates are shown in Table 5.2. In the initial state, the natural frequencies of the first, second and third modes were 7.6 Hz, 44.7 Hz and 89.1 Hz, respectively. The damping ratio was 0.257. The size of the upper floor FRF was 6.028×10^{-3} g / N, followed by the second floor (4.156×10^{-3} g / N) and the first floor where it was 3.794×10^{-3} g / N. With a load of 47.14 kN, DI_{FRF} was observed as 0.283 and there was an audible sound of FRP slip. In the final damage state, the damping ratio of the structure was 0.433 and the frequencies of the first, second and third modes were 4.7 Hz, 35.6 Hz and 76 Hz, respectively.

Table 5.2: Frequency, FRF, Damping Ratio and Damage Index of FRP Retrofitted frame

Cycle	Load (kN)	P/P(max.)	Natural frequency of each mode, Hz			FRF magnitude of each floor, 10^{-3} g/N			Damping Ratio	DI (1)	DI (2)	DI (3)
			$\omega 1$	$\omega 2$	$\omega 3$	FRF1	FRF2	FRF3				
0	0	0.00	7.6	44.7	89.1	3.794	4.156	6.028	0.253	0	0	0
2	28.58	0.51	7.2	42.3	86.4	3.498	3.82	5.486	0.283	0.078	0.080	0.089
4	42.19	0.75	7	39.7	84.3	3.21	3.513	5.122	0.341	0.153	0.154	0.150
6	47.14	0.84	6.7	37.4	81.7	2.72	2.971	4.326	0.363	0.283	0.285	0.282
8	53.94	0.96	6	36.9	78.5	2.168	2.377	3.461	0.378	0.428	0.428	0.425
10	56.03	1.00	5.4	36.2	76.9	1.519	1.655	2.405	0.394	0.599	0.601	0.601
12	53.98	0.96	4.8	35.8	76.2	0.932	0.978	1.487	0.419	0.754	0.764	0.753
13	52.21	0.93	4.7	35.6	76	0.812	0.928	1.287	0.433	0.767	0.757	0.786

Table 5.2 shows that the average natural frequency decreases by approximately 38.2% compared to the initial state. The change in damping ratio results in an average increase in damping ratio of approximately 71.14%. The upper floor FRF size was 1.287×10^{-3} g / N, the second floor was 0.928×10^{-3} g / N, and the first floor was 0.812×10^{-3} g / N first floor

at 78.6% the initial state. The DI_{FRF} loss rate was 0.767. Although the GFRP wrap increased the dynamic properties of the structure well.

5.6.3 ECC Strengthened frame

The ECC replaced frame showed a similar frequency response. The FRF responses were analyzed and the test results are presented in Table 5.3. It is observed that there has been a decrease in the natural frequency and the frequency response function or an increase in the damage rate from 0 to 1. The ECC frame showed a change in the natural frequency from 8 Hz to 4 Hz with the change in the damage rate from 0 to 0.74. It has been observed that the frame replaced by ECC at the natural frequency at the failure stage indicates better ductility of the ECC-repaired frame.

It was found that the FRF size decreased and the damping ratio increased with increasing deviation, as shown in Figure 5.11. The ECC repaired frame showed a reduction in FRF size of 75.95% with progressive damage. On the other hand, the damping ratio showed a significant increase of 74.90% compared to the control frame and the modified FRP frame, where it was 69.6% and 71.14%, respectively.

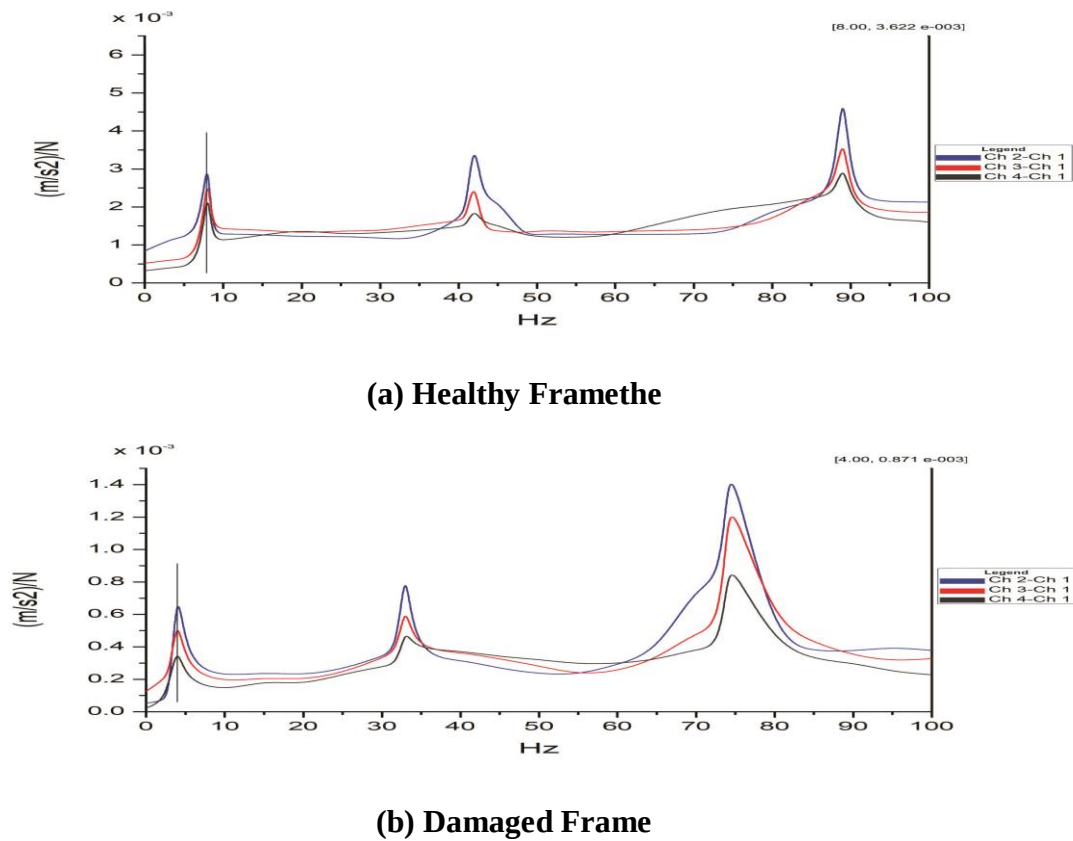


Figure 5.11: Frequency Response Function for ECC Repaired Brick Infilled Frame

This indicates a better power dissipation of the repaired ECC frame. The damage tolerance capacity of the frames was estimated using the Damage Index (DI) and showed that with increasing load in each cycle the damage level also increased. The ductile nature of the concrete allowed the early formation of cracks in the hinge area.

Table5.3: Frequency, FRF, Damping Ratio and Damage Index of ECC replaced frame

Cycle	Load (kN)	P/P(max.)	Natural frequency of each mode, Hz			FRF magnitude of each floor, 10^{-3} g/N			Damping Ratio	DI (1)	DI (2)	DI (3)
			$\omega 1$	$\omega 2$	$\omega 3$	FRF1	FRF2	FRF3				
0	0	0.00	8	42	89	3.622	3.981	5.654	0.251	0	0	0
2	31.94	0.52	7.9	41.2	85.3	3.355	3.682	5.187	0.283	0.073	0.075	0.082
4	46.44	0.75	7	40	82.1	3.105	3.377	4.785	0.341	0.142	0.151	0.153
6	54.87	0.89	6.2	37.1	79.4	2.677	2.794	4.142	0.363	0.260	0.298	0.267
8	59.7	0.96	6	36	77	2.127	2.294	3.242	0.378	0.412	0.423	0.426
10	61.95	1.00	5	34	75.4	1.491	1.588	2.32	0.394	0.588	0.601	0.589
12	59.65	0.96	4.2	33.2	74.2	0.971	1.067	1.632	0.429	0.731	0.731	0.711
13	57.5	0.93	4	33	74	0.871	0.984	1.442	0.439	0.740	0.732	0.721

The Damage Index plot in Figure 5.12 shows early damage to the control frame compared to the ECC repaired frame.

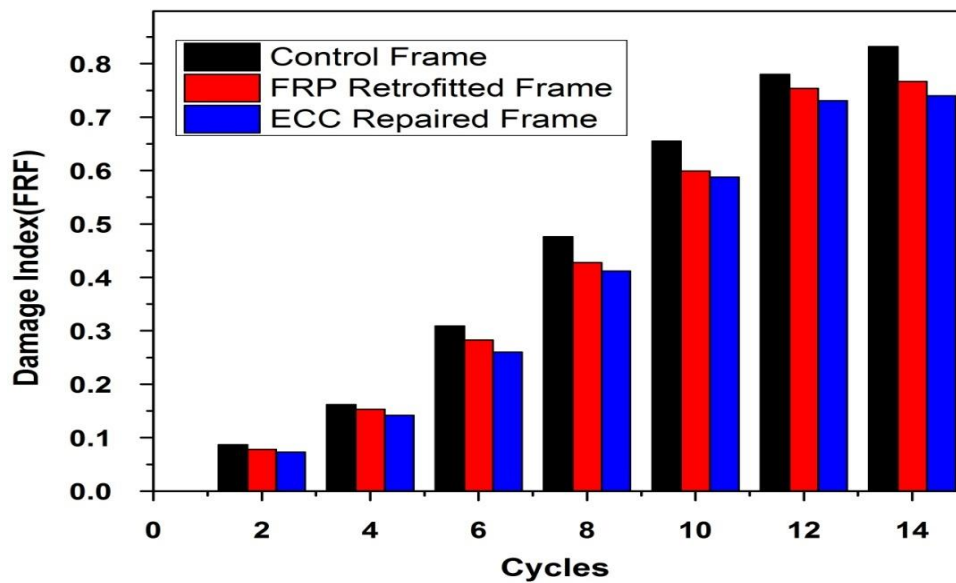


Figure 5.12:Damage index

5.7 CLOSURE

It can therefore be concluded from these results that the FRP retrofitted frame shows improvement even with the old reinforcement steel bars as compared to control frame and the strain and bonding behavior of ECC arrest the sliding of the rebar and allows the rebar to yield during inelastic rotation. The presence of microfibers in the composite acts as a micro-level reinforcement to resist early crack growth. It contributes to the improvement of the energy dissipation capacity and the rigidity maintenance capacity. The change of resonant frequency is large at fundamental mode as compared to higher mode with increase in damage levels. The structural damping increases when stiffness of structure decreases. The damage tolerance capacity of the frame has been estimated using the Damage Index (DI_{FRF}) based on FRF magnitude and it showed that as the load increases in every cycle, the damage level also increases. Damage Index plot shows early damage in control frame in comparison to FRP retrofitted and ECC repaired frame. So resulting in ECC repaired frame shows better results followed by the FRP retrofitted frame.

CHAPTER 6

NONLINEAR AND DYNAMIC FINITE ELEMENT MODEL AND COMPARATIVE ANALYSIS

6.1 GENERAL

This chapter discusses the theory related to both nonlinear FE analysis and dynamic FE analysis. All the necessary steps to create the models are explained in detail. The comparative analysis between FE models and experimental results of infilled frame is presented. The dynamic responses of the original and FRP-strengthened frame are simulated. The modal assurance criterion and coordinate modal assurance criterion, which are derived from mode shape, are used to identify damage on structures.

6.2 NON-LINEAR FINITE ELEMENT ANALYSIS

It is important to realize that the whole RC structure does not have to be analyzed using a full nonlinear formulation. However, a simplified or linear formulation can be used in many cases. It is a matter of engineering knowledge and assessment, whether the inaccuracies due to a simplified formulation are acceptable or not. FE method is a numerical technique that is used to simulate the structural behavior and the failure mechanism of structures. ATENA is FE analytical software that has versatile applications in nonlinear analysis of structure. In short, this is a useful program in the nonlinear analysis of RC members (Vimuttasoongviriya et al., 2009 and 2010; Cervenka et al., 1991, 1992, and 1995). This FE software program (ATENA-3D v5) is popular in Europe where it is applied in studies to simulate the behavior of experimental frames.

6.2.1 Equivalent uniaxial concrete law

The nonlinear behavior of concrete in the biaxial stress state is described by the relationship between effective stress σ_c^{ef} and the equivalent uniaxial strain ε^{eq} . The equivalent uniaxial strain is produced by the stress of concrete σ_{ci} with modulus E_{ci} associated with the direction i . Within this assumption, the nonlinearity representing a damage is caused only by the governing stress σ_{ci} . The details can be found in Chen (1982). The equivalent uniaxial strain is calculated as

$$\varepsilon^{eq} = \frac{\sigma_{ci}}{E_{ci}} \quad (6.1)$$

Unloading is a linear function close to the origin. The equivalent uniaxial stress-strain diagram for concrete and an example of the unloading point U are shown in Figure 6.1. The relation between effective stress and equivalent uniaxial strain depends on a load history. The numbers of the diagram parts in Figure 6.1 are used in the results of the analysis to indicate the state of damage of concrete. A change from loading to unloading occurs when the increment of the effective strain changes sign. If subsequent reloading occurs, the linear unloading path is followed until the last loading point U is reached again. Then, the loading function is resumed.

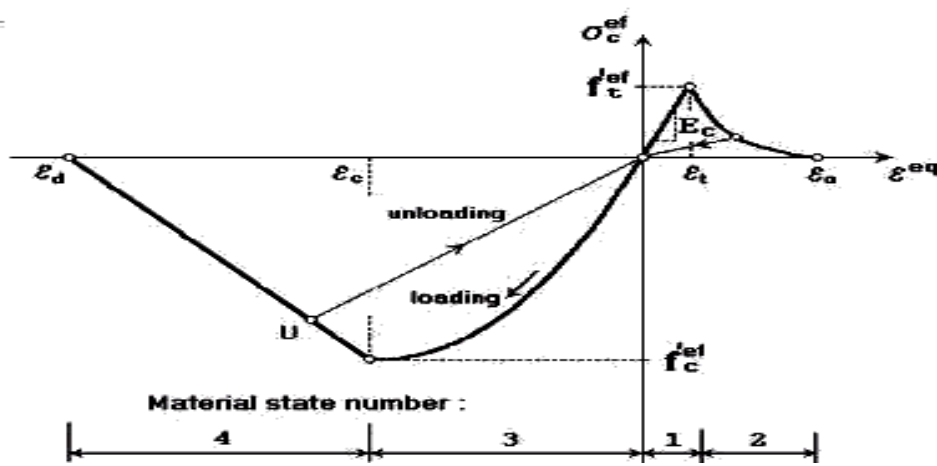


Figure 6.1 Uniaxial stress-strain relationship for concrete (Chen, 1982).

6.2.2 Tension before concrete cracking

The behavior of concrete under tension and without cracks is assumed to be linear elastic from initial state to the effective tensile strength point. The tensile strength without cracks is calculated as

$$\sigma_c^{ef} = E_o \varepsilon^{eq} \quad \text{when } 0 \leq \sigma_c \leq f_t'^{ef} \quad (6-2)$$

where E_o is the initial elastic modulus of concrete, and $f_t'^{ef}$ is the effective tensile strength derived from the biaxial failure function.

6.2.3 Tension after cracking of concrete.

The exponential crack opening law is used to model the concrete crack opening as shown in Figure 6.2. This function of crack opening was derived experimentally by Hordijk (1991). The function of crack opening is calculated as

$$\frac{\sigma}{f_t^{ef}} = \left\{ 1 + \left(c_1 \frac{w}{w_c} \right)^3 \right\} \exp \left(-c_2 \frac{w}{w_c} \right) - \frac{w}{w_c} (1 + c_1^3) \exp(-c_2) \quad (6.3)$$

where w is the crack width, w_c is the crack opening at the complete release of stress, σ is the normal stress in the crack. Values of the constants are, $c_1 = 3$, $c_2 = 6.93$, respectively. G_f is the fracture energy needed to create a unit area of stress-free crack, f_c^{ef} is the effective tensile strength derived from a failure function. The crack opening displacement w is derived from strains according to the crack band theory.

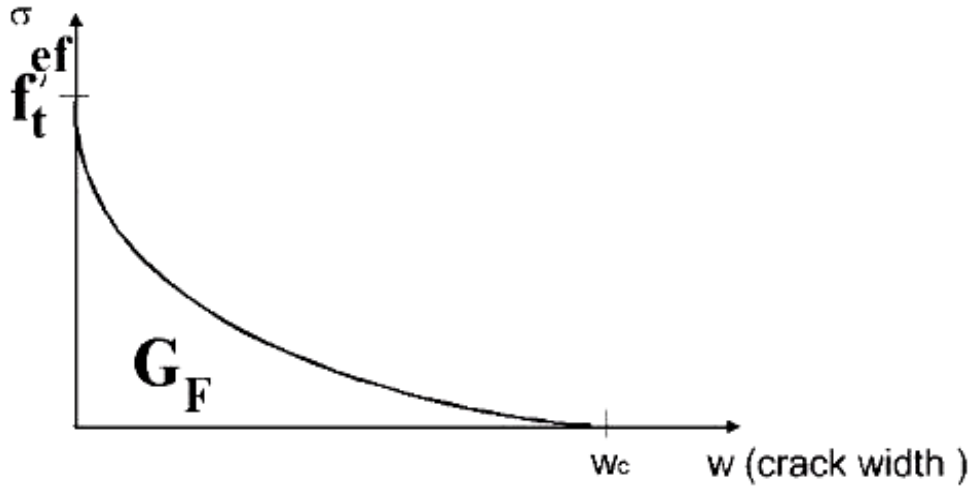


Figure 6.2 The exponential crack opening law for concrete (Hordijk, 1991).

6.2.4 Compression before peak stress of concrete

The nonlinear behavior of compression follows the Committee Euro International du Beton Model (CEB-FIP, 2010). The formula has been adopted for the ascending branch of the concrete stress-strain law in compression as shown in Figure 6.3. This formula enables a wide range of curve forms and is appropriate for normal as well as high strength concrete.

$$\sigma_c^{ef} = f_c^{ef} \frac{qx - x^2}{1 + (q-2)x} \quad (6.4)$$

where f_c^{ef} is concrete effective compressive strength, x is normalized strain, q is shape parameter, ε is strain, ε_c is strain at the peak stress, E_o is initial elastic modulus and E_c is secant elastic modulus at the peak stress. As a consequence of the above assumption, distributed damage which is considered before the peak stress is reached, contrary to localized damage, is considered after the peak.

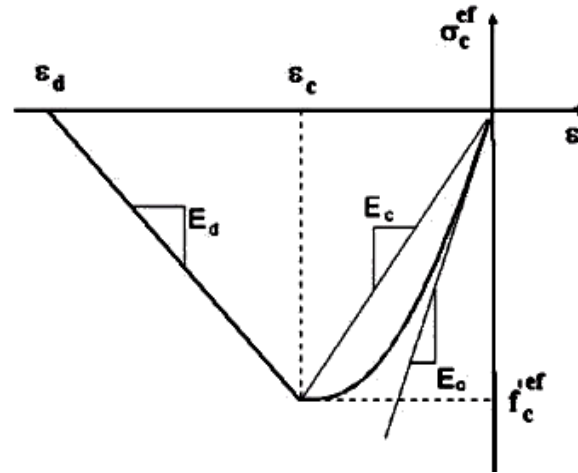


Figure 6.3 The nonlinear behavior of compression for concrete (CEB-FIP, 1990).

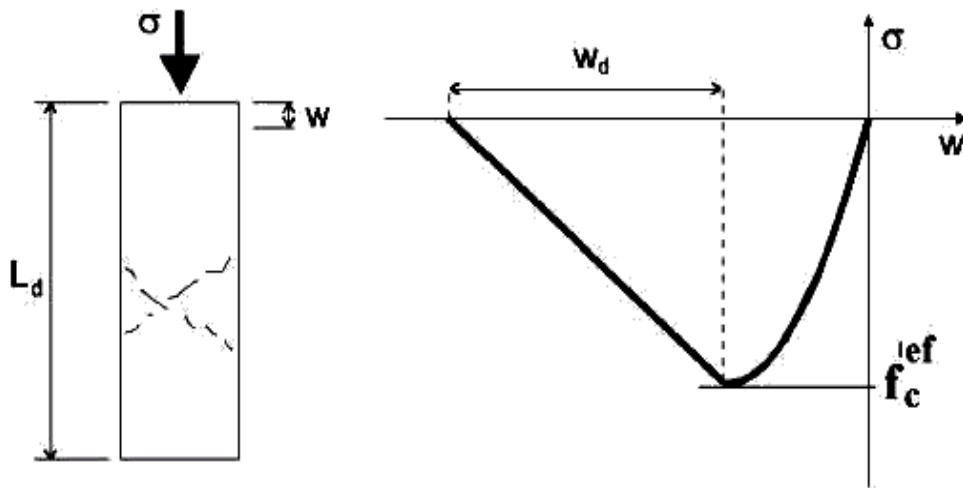


Figure 6.4 The compression behavior for concrete after peak stress (Van Mier, 1986).

6.2.5 Compression after peak stress of concrete

The fictitious compression plane model is based on the assumption that compression failure is localized in a plane normal to the direction of principal stress. All post-peak compressive displacements and energy dissipation are localized in this plane. It is assumed that this displacement is independent of the size of the structure. This hypothesis is supported by experiments conducted by Mier (1986). This assumption is analogous to the Fictitious Crack Theory for tension, where the shape of the crack-opening and the fracture energy are defined and are considered as material properties. The softening law is transformed from a fictitious failure plane, Figure 6.4, to the stress-strain relation valid for the corresponding volume of continuous material. A slope of the softening law is defined by means of the softening modulus E_d . This formulation is dependent on the size of the finite element mesh.

Fictitious compression plane model is applied to simulate compression after peak stress of concrete. The slope of softening part of stress-strain diagram is defined by two points: a peak of the diagram at the maximal stress and a limit compressive strain, ε_d , at the zero stress. This strain is calculated from a plastic displacement, w_d , and a band size, L_d , according to the following expression:

$$\varepsilon_d = \varepsilon_c + \frac{w_d}{L_d} \quad (6.5)$$

6.2.6 Fracture process, crack width of concrete

The process of crack formation can be divided into three stages as shown in Figure 6.5. The uncracked stage is before a tensile strength is reached. The crack formation takes place in the process zone of a potential crack with decreasing tensile stress on a crack face due to a bridging effect. Finally, after a complete release of the stress, the crack opening continues without the stress (Cervenka et al., 1991, 1992, and 1995). The crack width w is calculated as a total crack opening displacement within the crack band.

$$w = \varepsilon_{cr} L_t' \quad (6.6)$$

where ε_{cr} is the crack opening strain, which is equal to the strain normal to the crack direction in the cracked state after the complete stress release.

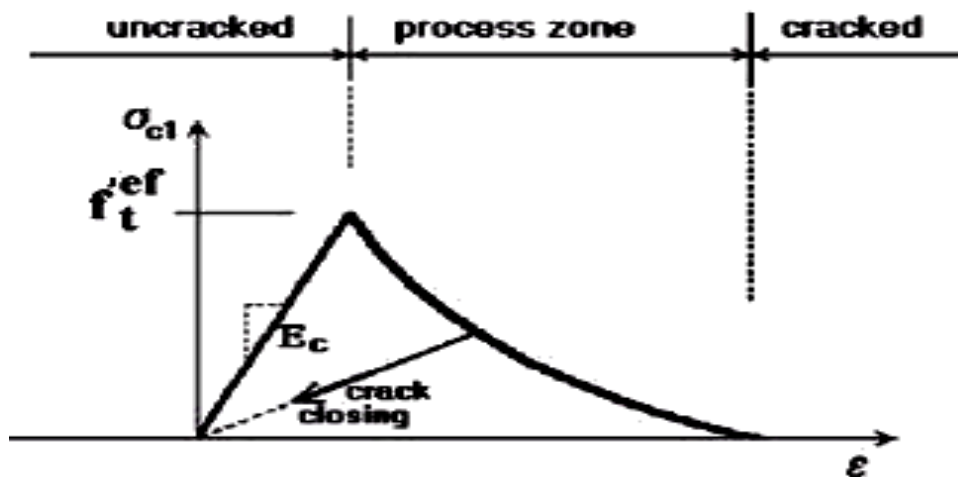


Figure 6.5 Stages of crack opening (Cervenka et al, 1991).

Figure 6.5 shows that the smeared model based on the refined crack band theory can successfully describe the discrete crack propagation in plain, as well as reinforced concrete. It

is also possible, that the second stress, parallel to the crack direction, exceeds the tensile strength. Then the second crack, in the direction orthogonal to the first one, is formed using the same softening model as the first crack.

6.2.7 Compressive failure of concrete

A biaxial stress failure criterion according to Kupfer et al. (1969) is used as shown in Figure 6.6. In the compression-compression stress state the failure function is

$$f_c'^{ef} = \frac{1+3.65a}{(1+a)^2} f_c' \quad (6.7)$$

where σ_{c1} , σ_{c2} are the principal stresses in concrete and f_c' is the uniaxial cylinder strength. In the biaxial stress state, the strength of concrete is predicted under the assumption of a proportional stress path. In the tension-compression state, the failure function continues linearly from the point $\sigma_{c1} = 0$, $\sigma_{c2} = f_c'$ into the tension-compression region with the linearly decreasing strength:

$$f_c'^{ef} = f_c' r_{ec} \quad (6-8)$$

where r_{ec} is the reduction factor of the compressive strength in the principal direction 2 due to the tensile stress in the principal direction 1.

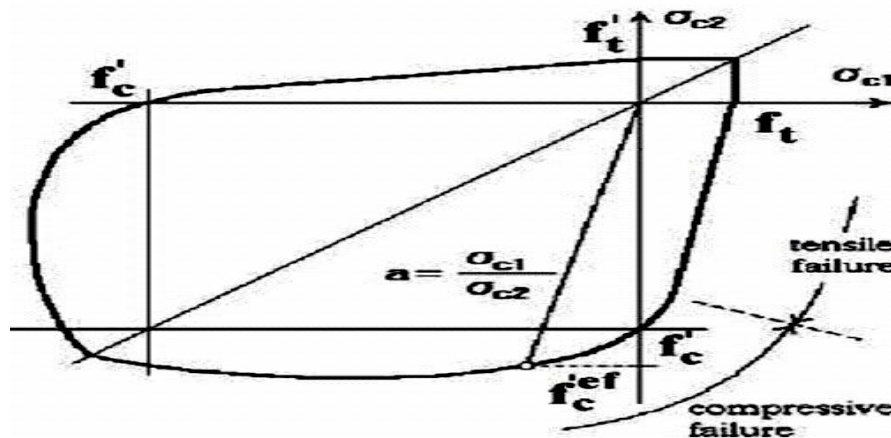


Figure 6.6 Biaxial failure function for concrete (Kupfer, 1969).

6.2.8 Tensile failure of concrete

In the tension-tension state, the tensile strength is constant and equal to the uniaxial tensile strength, f_t' . In the tension-compression state, the tensile strength is reduced by the relation:

$$f_t'^{ef} = f_t' r_{et} \quad (6.9)$$

where r_{et} is the reduction factor of the tensile strength in the direction 1 due to the compressive stress in the direction 2.

6.2.9 Model of smeared crack

The smeared crack approach for modeling of the cracks is adopted in ATENA. Within the smeared concept two options are available for crack models: the fixed crack model and the rotated crack model. In both models the crack is formed when the principal stress exceeds the tensile strength. Although the rotated crack model is assumed that the cracks are uniformly distributed, but no shear strain occurs on the crack plane and only two normal stress components be defined (Crisfield, 1989). Therefore, this research used the fixed crack model.

In the fixed crack model (Darwin, 1974), the crack direction is given by the principal stress direction at the moment of the crack initiation. During further loading this direction is fixed and represents the orthotropic material axis. The principal stress and strain directions coincide in the uncracked concrete because of the assumption of isotropy in the concrete component. After cracking the component is introduced. The weak material axis m_1 is normal to the crack direction, the strong axis m_2 is parallel with the cracks. In a general case the principal strain axes ε_1 and ε_2 rotate and need not to coincide with the axes of the orthotropic m_1 and m_2 . This produces a shear stress on the crack face as shown in Figure 6.7. The stress components σ_{c1} and σ_{c2} denote the stresses normal and parallel to the crack plane.

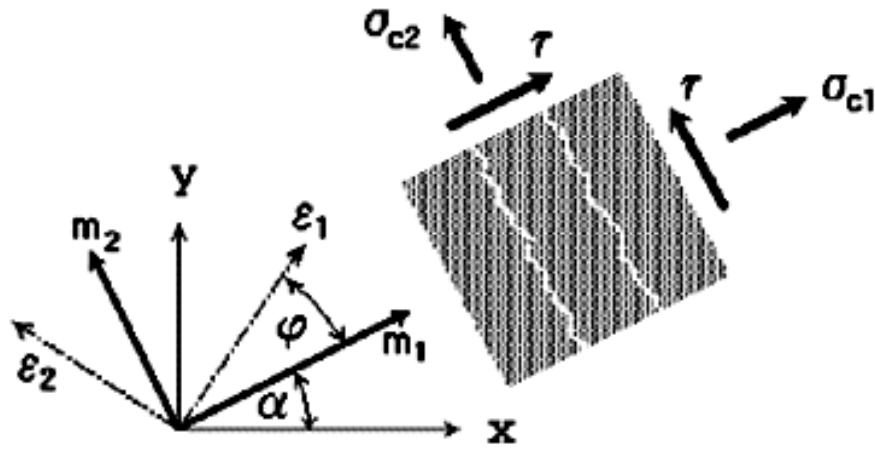


Figure 6.7 Fixed crack model of concrete (Drawin, 1974).

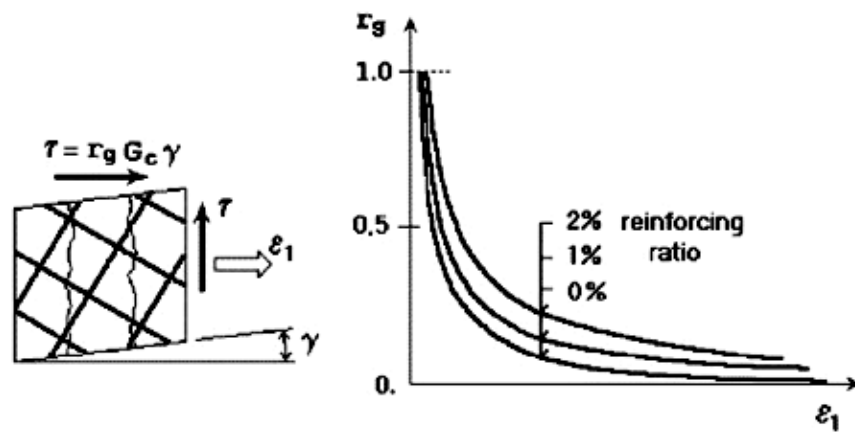


Figure 6.8 Shear retention factor of concrete (Kolmar, 1986).

6.2.10 Shear stress in cracked concrete

The shear modulus (Kolmar, 1986) is reduced with growing strain normal to the crack, as shown in Figure 6.8. This presents a reduction of the shear stiffness due to the crack opening.

$$G = r_g G_c \quad (6-10)$$

where r_g is the shear retention factor, G is the reduced shear modulus, G_c is the initial concrete shear modulus.

6.2.11 Compressive strength of cracked concrete

A reduction of the compressive strength after cracking in the direction parallel to the cracks in ATENA is calculated as following:

$$f_c'^{ef} = r_c f_c' \quad (6-11)$$

where r_c is compressive reduction factor, f_c' is compressive strength of concrete, and the constant c is 0.45, respectively. This function is based on Kolleger et al. (1988) which has the form of the Gauss's function as shown in Figure 6.9. For the zero normal strain ε_v , there is no strength reduction, and for large strains, the strength asymptotically approaches the minimum value $f_c'^{ef} = c f_c'$.

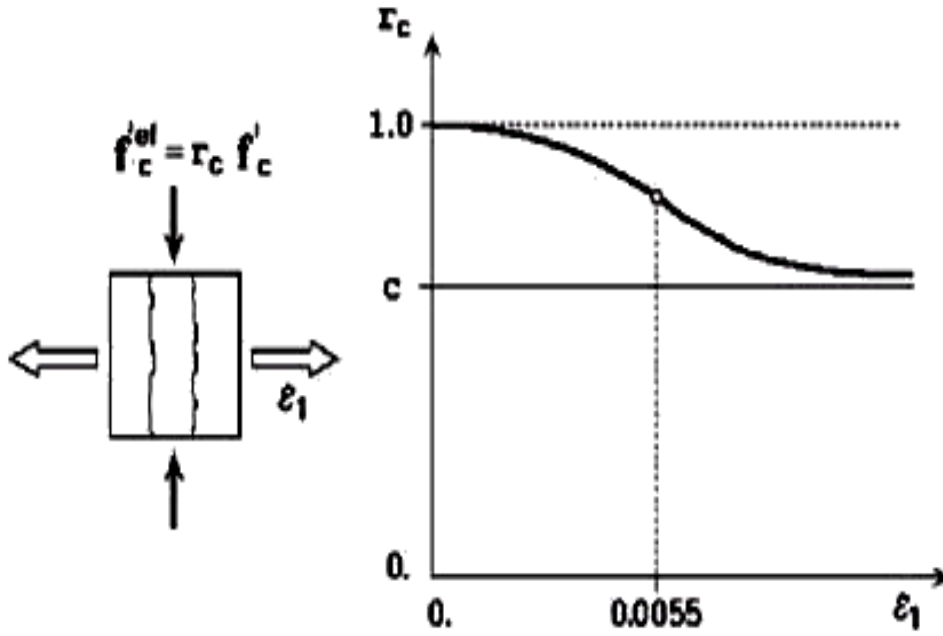


Figure 6.9 Compressive strength reduction of cracked concrete (Kolleger et al, 1988).

6.2.12 Reinforcement stress-strain law

Stress-strain law for the reinforcement steel bar is the bilinear law. In the bilinear law, elastic-perfectly plastic, is assumed as shown in Figure 6.10. The initial elastic part has the elastic modulus of steel E_s . The second line represents the plasticity of the steel with hardening and its slope is the hardening modulus E_{sh} . In case of perfect plasticity $E_{sh} = 0$. Limit strain ε_L represents limited ductility of steel.

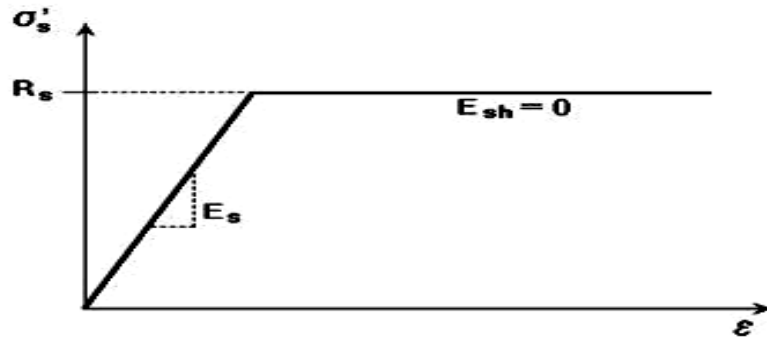


Figure 6.10 The bilinear stress-strain law for reinforcement.

6.2.13 Fiber-Reinforced polymer model

In general case of RC structure, the elemental geometric modeling of concrete has been done using 3D solid brick element with 8 up to 20 nodes in ATENA as shown in Figure 6.11. The 3D solid brick element has three degree of freedom at each node: translations in the nodal x, y and z directions. This is an iso-parametric element, capable of plastic deformation, cracking in three orthogonal directions, and crushing. The most important aspect of this element is the treatment of nonlinear material properties.

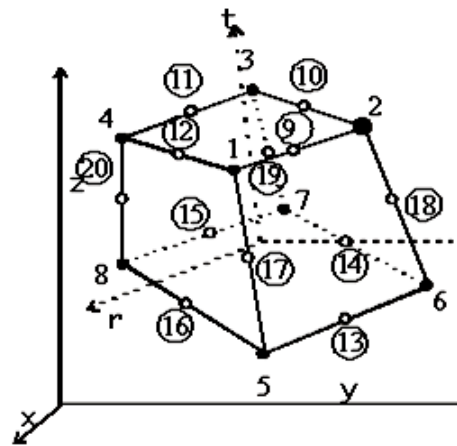


Figure 6.11 Geometry of 3D solid brick element with 8 up to 20 nodes.

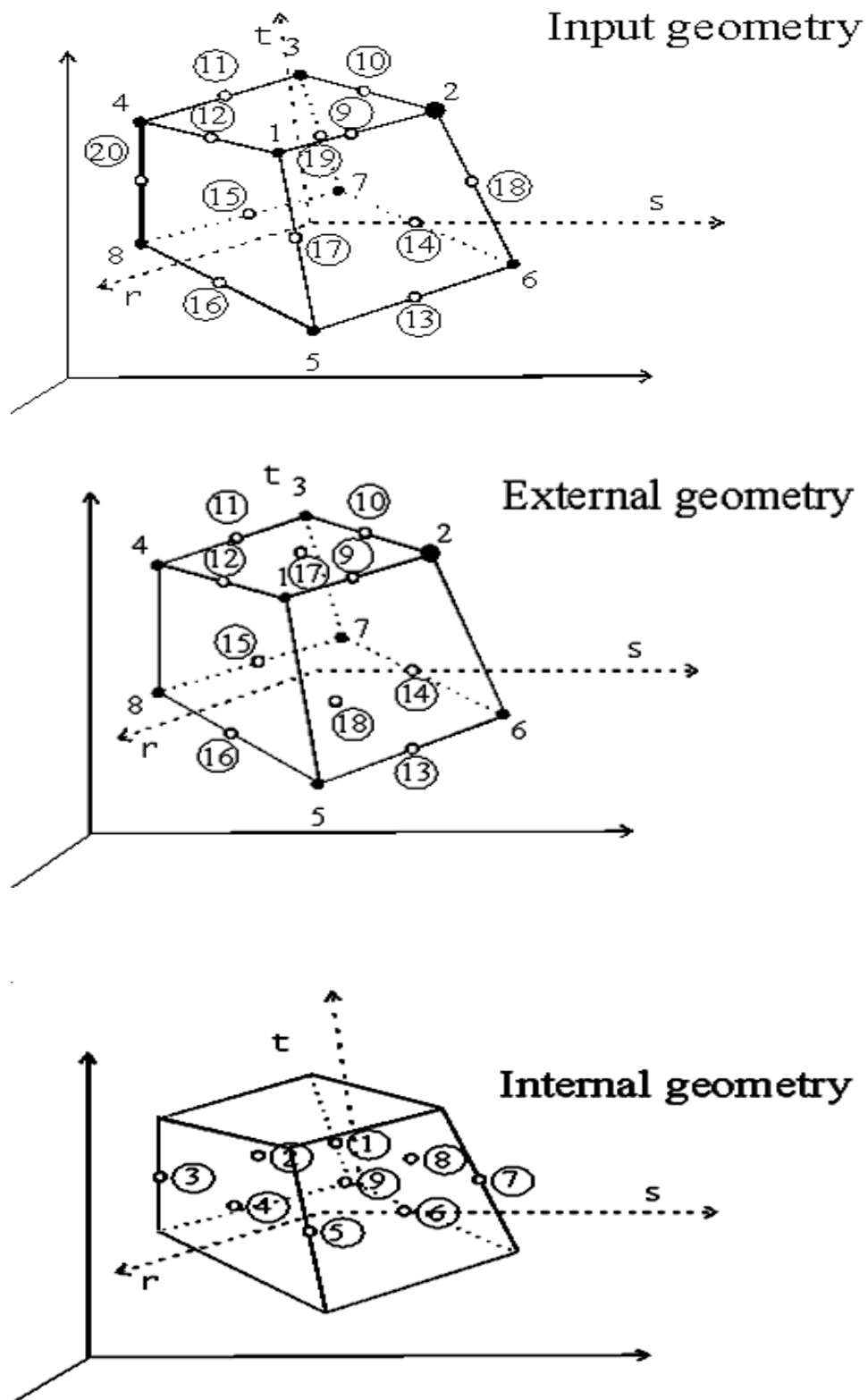


Figure 6.12 Geometry of shell element for FRP material.

FRP modeling : The FRP modeling can be done as a 3D shell element in ATENA. The Ahmad shell element implemented in ATENA (Ahmad et al., 1970), described in ATENA

theory manual. The present Ahmad element belongs to group of shell element formulation that is based on 3D elements concept. It can be used to model thin as well as thick shell or plate structures.

Geometry of the FRP : The FRP can be modeled as a shell element in ATENA. The Ahmad shell element used the 20 nodes iso-parametric brick element. This is needed, in order to be able to use the same pre and post-processors support for the shell and native 3D brick element. After the 1st step of the analysis, the input geometry will automatically change to the external geometry from Figure 6.12. As nodes 17 and 18 contain only so called bubble function, the element is post-processed in the same way as would be the iso-parametric brick element. Internally, all of the element's vectors and matrices are derived based on the internal geometry as also depicted in Figure 6.12.

Property of the FRP element: FRP is an repetitive shell element. In the following general shell element theory concept, every node of an element has five degree of freedom, e.g. three displacements and two rotations in planes normal to mid surface of element. In order to facilitate a simple connection of this element with other true 3D elements, the five degrees of freedom are transformed into x, y, z displacement of a top node and x, y displacement of a bottom node. The two nodes are located on the normal to mid-surface, passing thru the original mid-surface element's node.

6.3 NON-LINEAR FINITE ELEMENT ANALYTICAL RESULTS

An eight-node brick element was used to model the concrete as shown in Figure 6.13. The element is capable of plastic deformation, cracking in three orthogonal directions, and crushing. A poly-line element was used to model the steel bars. Two nodes are required for this element. Each node has three degrees of freedom. The steel for the FE model was assumed to be an elastic and perfectly plastic material. A 20 node quadratic 3D brick shell element was used to model the FRP composite (ATENA Revision 2005/01, 2005). This element allows for different material layers with different orientations. The FRP components consist of two constituents which are combined at a macroscopic level and are not soluble in each other. One constituent is the reinforcement, which is embedded in the second constituent-epoxy. The reinforcing material is in the form of anisotropic materials of glass fibers which are typically stiffer and stronger than the epoxy (Ferracuti and Savoia, 2006; Kachlakev et al., 2001). Nodes of FRP layered shell elements are connected to those of adjacent concrete solid element in order to satisfy the perfect bond assumption. Figure 6.13

shows a schematic representation of FRP composites. The material properties of epoxy and GFRP/CFRP sheet are presented in Table 6.2. An eight-node brick element was used to model steel plates at the loading point at the top floor, and the supports as shown in Figure 6.14 and 6.15. A 25 mm thick steel plate was added at the supporting and loading location in order to avoid stress concentration problems. By taking advantage of the symmetry of the frame, a symmetrical half of the full frame was used for modeling.

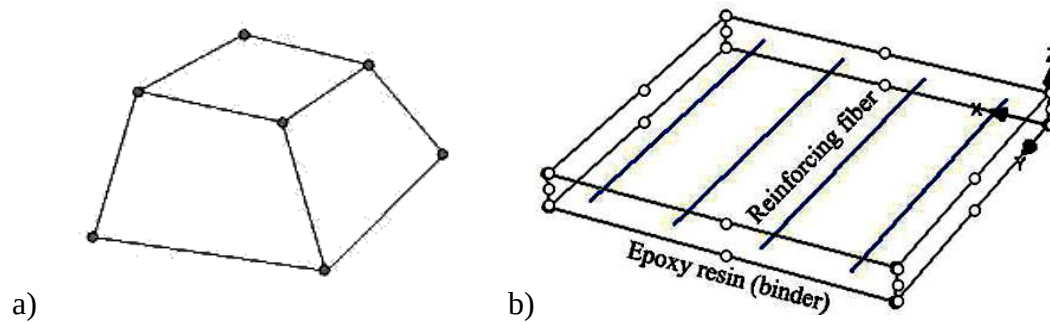


Figure 6.13 Element types: a) 8 nodes brick element model for concrete and steel plate and b) 20 nodes shell element model for FRP composite.

The typical FE model of the control and retrofitted frame is shown in Figure 6.14 and Figure 6.15. In a plane of symmetry, the deflection of frame in the direction perpendicular to the plane remained at zero. Reinforcements and meshes with supports in the frame are shown in Figures 6.16 and 6.17. The monitoring points were measured in the middle of the upper floor, where the greatest horizontal deflection of frame can be expected.

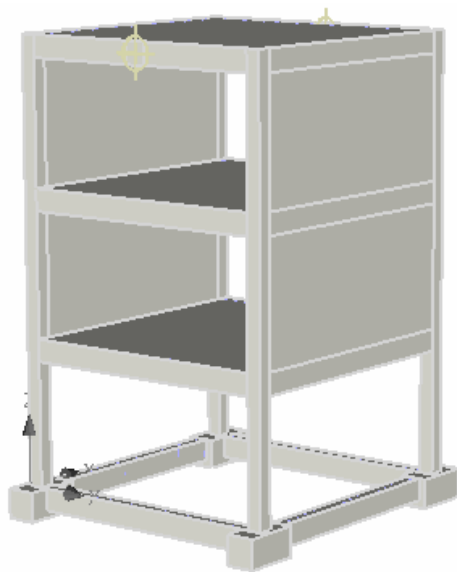


Figure 6.14 View of FE model



Figure 6.15 View of FE model of Retrofitted Frame

Table 6.1 The detail of material properties for FE modeling

Compressive strength of concrete cutes (average)		
Element type for ATENA	3D Nonlinear Cementitious 2	
Property of Frame	Fresh concrete of 22.46 MPa	
Property of Brick work	Elastic modulus of 23,696 MPa*	
	Brick work block 20.46 MPa	
Steel plate		
Element type for ATENA	3D Elastic Isotropic	
Elastic modulus	200000 MPa	
Poisson’s ratio	0.300	
Steel reinforcement bars		
Element type for ATENA	Reinforcement	
Elastic modulus	200000 MPa	
Yield strength of steel bars	10 mm diameter bar	415 MPa
	8 mm diameter bar	415 MPa
	6 mm diameter bar	415 MPa

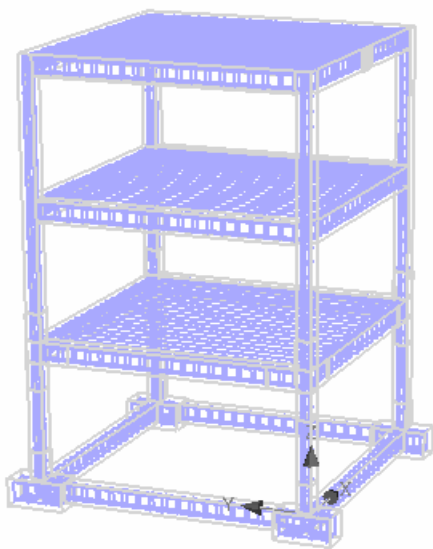


Figure 6.16 Reinforced Frame

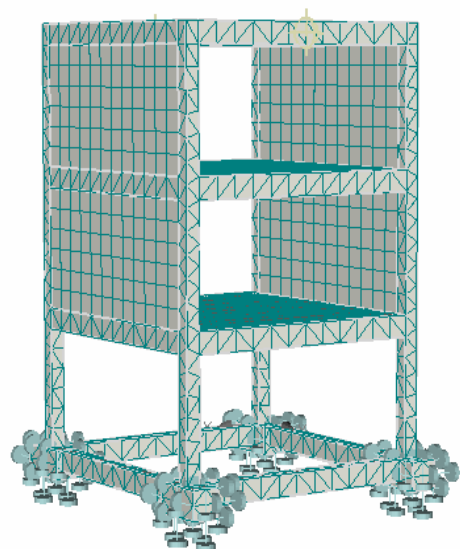


Figure 6.17 FE Mesh with support

6.3.1 Control frame

In non-linear FE analysis, the ATENA program was used in Newton-Raphson equilibrium iterations to update the rigidity of the model. The loads applied to an FE model are divided into a series of load increments, called load steps. According to model code 90 of CEB-FIP (2010), the "modulus of elasticity" of the fresh concrete of the control frame was 23,696 MPa. For greater accuracy, the stiffness damage index for the damaged elements should be adjusted in this study. The parameters used in this index depend on the cross-section and feature of the structural element material. Because the properties of an element of the FE model differ from those of the real model (Kanwar et al., 2007 and Supaviriyakit et al., 2004).

This index was modified in the modulus of elasticity degradation and applied to the FE model. The results of the FE analysis show that the final state was reached with a load of 52.35 kN with a displacement of 50 mm.

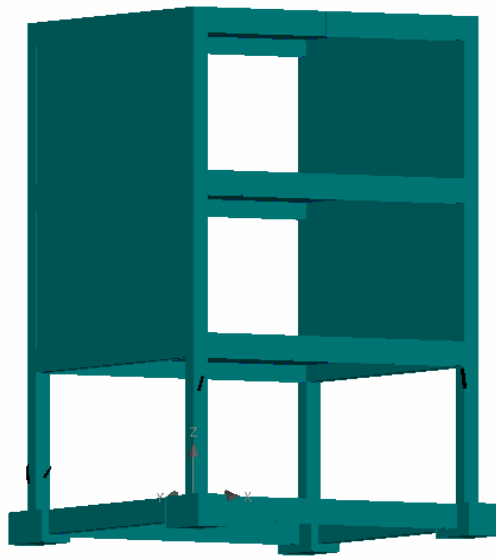


Figure 6.18 First Crack appeared in Frame

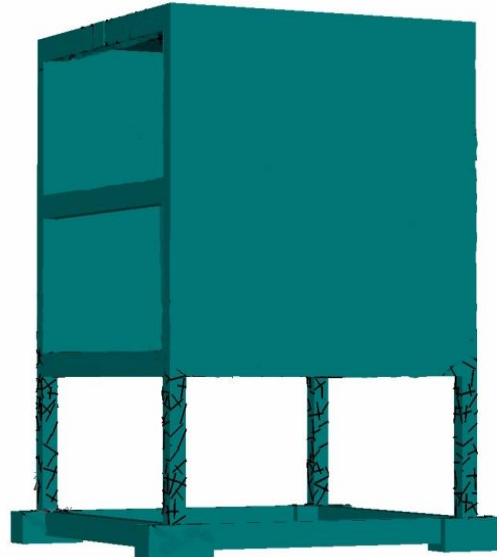


Figure 6.19 Damaged Frame column

First crack appeared in column of frame shown in figure 6.18 The crack pattern of the FE model is shown in Figure 6.19. It shows that in the final state, cracks have occurred in the first-floor beam column joints. In the final state, most of the primary cracks occurred in columns of the first and second floor. The crack pattern in masonry infilled had a diagonal shape, indicating that the cut resistance of this frame was weak. The lateral load capacity of the FE model increased by approximately 3.7% compared to the experimental control frame shown in figure 6.20. The crack appeared in experimental and FE model shown the same

pattern of cracks as shown in figure 6.21 which shows that the behaviour of FE model is similar to experimental frame.

Table 6.2 The detail of material properties for FE modeling

Steel reinforcement bars (continues)		
Specific material weight	78.5 MN/m	
Coefficient of thermal expansion	0.000015 1/K	
Mechanical properties of FRP composite		
Element type for ATENA	Reinforcement	
Element type for SAP 2000	Thin Shell Element	
Fiber type	GFRP (G sheet EU-900)	CFRP
Thickness	0.34 mm	1.2mm
Density	2.6 g/cm ³	1.8 g/cm ³
Tensile Strength	1.59 GPa	3.00GPa
Elastic Modulus	63 GPa	228GPa
Mechanical properties of epoxy		
Element type for ATENA	3D Bilinear Steel Von Mises	
Elastic modulus	3.05 GPa	
Compression	78 MPa	
Tension	84 MPa	
Specific weight	9.83 kN/m ³	
Poisson’s ratio	0.4	

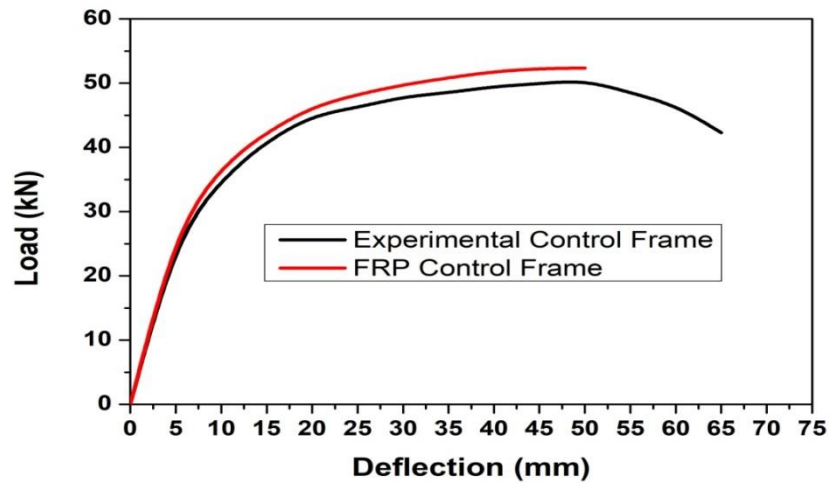


Figure 6.20 Load deflection behavior of Experimental Control and FE Control model

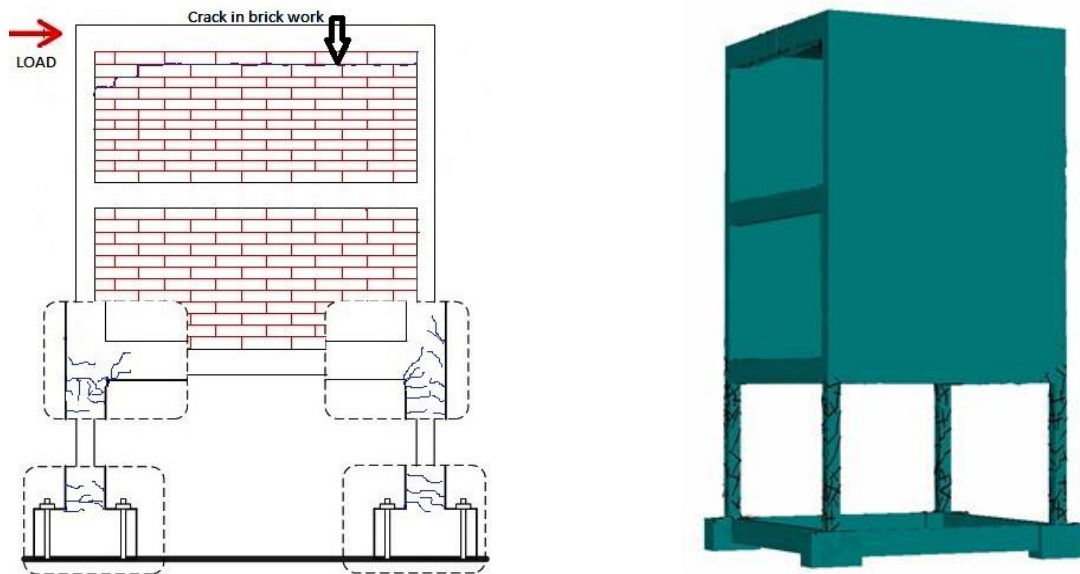


Figure 6.21 Crack patterns of Control Frame: a) experimental test and b) FE model.

6.3.2 FRP Retrofitted Frame

In case of reinforcing the FE model by using an FRP wrap, a 20-node 3D square brick shell element was used to model the FRP connection. The frame model was reinforced with epoxy resin and the damaged elements were wrapped with fiber sheets (GFRP and CFRP). The first part of the reinforcing material was epoxy resin with a thickness of 1 mm. The second part was a 0.34 mm thick GFRP panel at the column beam joint and a 1.20 mm CFRP panel at the bottom of the column. Both FRP materials were applied as reinforcement in two directions.

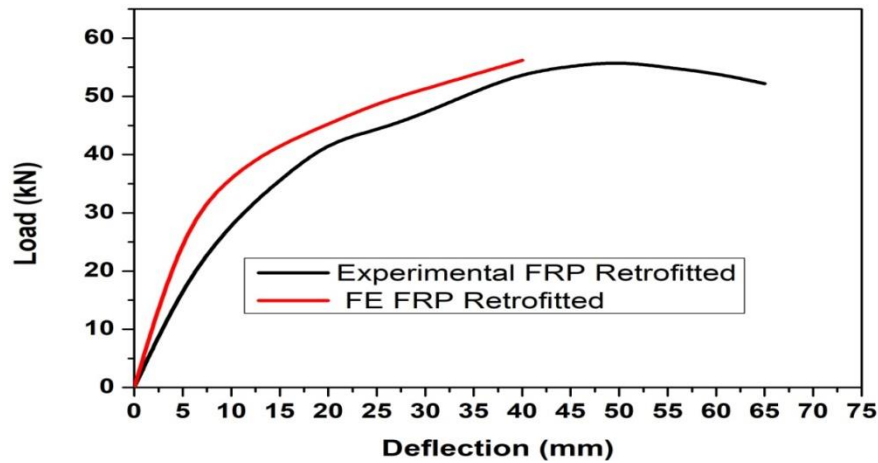


Figure 6.22 Load deflection behavior of Experimental Retrofitted and FE Retrofitted model

The first direction was along the axes of the structural element and the other direction was perpendicular to the first direction. From the experimental results in the final state of the control frame, the DI_k value of 0.942 was used to calculate the "Young's modulus" of concrete after damage. As a result, the modulus of elasticity of the retrofitted frame concrete was 1374 MPa. The FE analysis for the Retrofitted frame was performed in a similar process as the Control Frame. The analysis result of the frame shows that the end load was reached at a load of 56.03 kN and an offset of 50 mm. The properties of the epoxy resin and the FRP film are shown in Table 6.2.

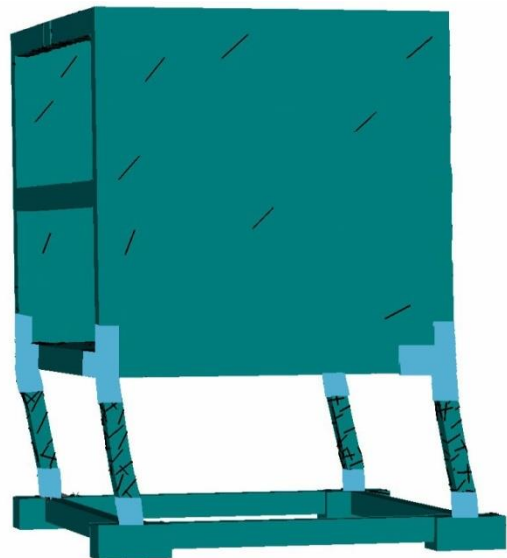


Figure 6.23 Deflected FRP retrofitted experimental and FE frame.

The first crack appeared in the retrofitted frame similar as control frame. At the end of the test, cracks appeared in the FRP layers at the terminals. There were severe cracks in stress areas of the GFRP layer on the column beam joint. However, it is observed that cracks also occurred in the masonry infill of RC elements.. FE's analysis results support the use of enveloped FRP for structural reinforcement. The lateral load capacity increased by approximately 4.18% compared to the experimentally retrofitted frame. The comparison of the experimental and the retrofitted frame is shown in Figure 6.22. The cracks developed and deflection observed in the FE model is similar to the cracking pattern developed in experimental frame.

In Control frame maximum cracks developed in the column beam joint of ground floor and in retrofitted frame maximum cracks observed below the retrofitted area. Deflection of the experimental frame and FE model is shown in Figure 6.23. FE model frame results are almost similar to the experimental results.

6.3.3 ECC Strengthened frame

FE analysis for ECC frame was performed in the similar process as that for the control frame. The crack pattern at final state of ECC frame is shown in Figure 6.24. The FE analytical results of ECC Frame showed that the ultimate load was achieved at load of 63.9 kN and a displacement of 55 mm. The load versus displacement behavior of frame is shown in Figures 6.25. At the end of the test, there were number of cracks occurred in whole column. However, it is observed that cracks also appeared in column-beam connections of ECC frame.

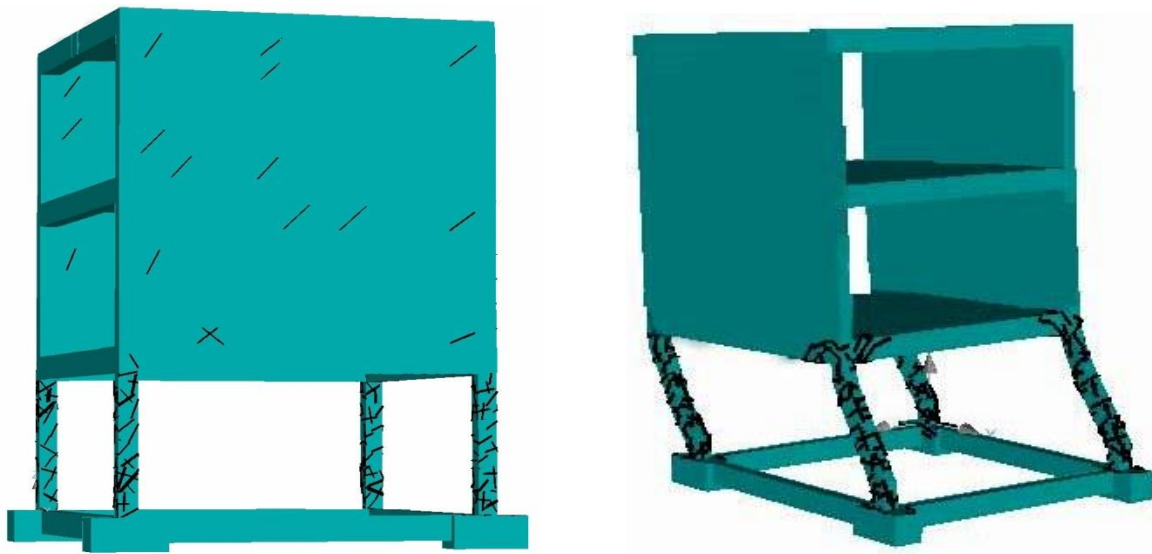


Figure 6.24 Crack pattern in ECC repaired FE frame.

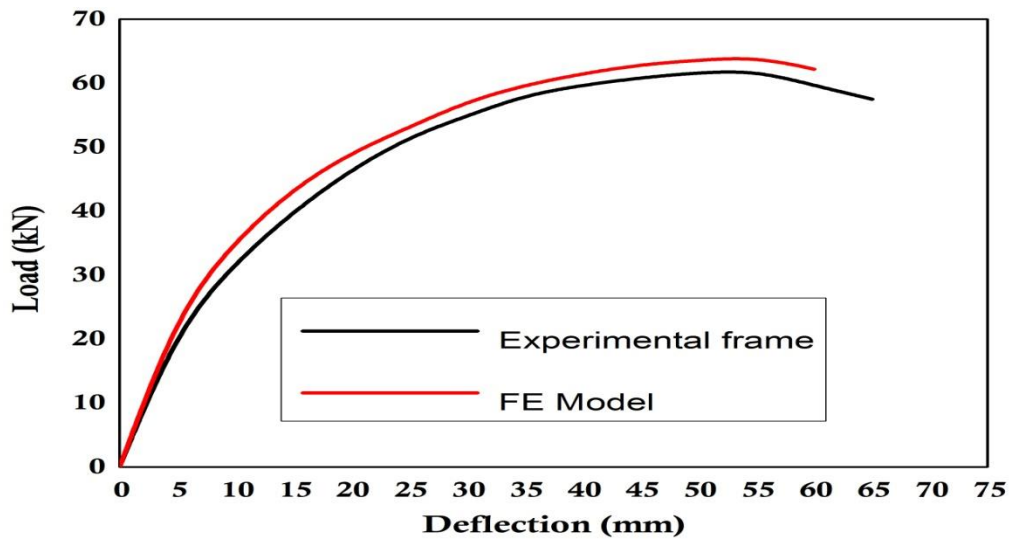


Figure 6.25 Comparison of Experimental and FE model Frame

Lateral load capacity increased approximately 22%, compared to Control Frame and approximately 13.7%, compared to FRP retrofitted frame. It is observed that cracks also occurred in the masonry infill of RC elements. The lateral load capacity increased by approximately 3.15% compared to the experimental ECC frame as indicated in Figure 6.25.

The deflected frame in the final state of the ECC frame is shown in Figure 6.26. FE's analysis results support the experimental results of frame.

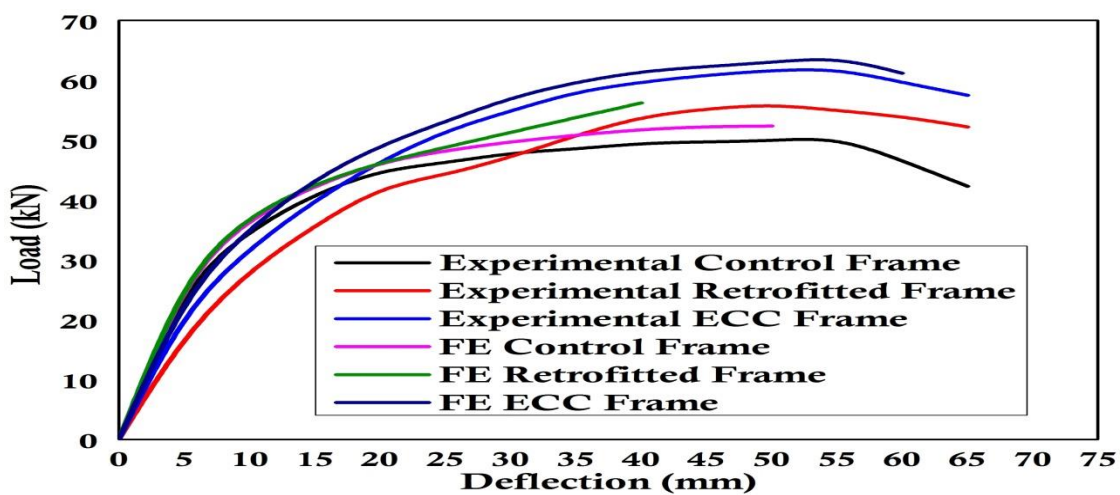


Figure 6.26 Comparison of all Experimental and FE model Frame

Overall comparison of load deflection behaviour of Experimental and FE model shown in figure 6.26. In which performance of ECC frame was very good as compared to both frames followed by FRP retrofitted frame. Stiffness of all frames Figure 6.27 shows that maximum stiffness degradation was occurred in control frame followed by FRP retrofitted frame whereas initial stiffness of ECC frame was low as compared to control and

retrofitted frame but stiffness at maximum deflection was better than both frames due to the ductile behaviour of ECC. The stiffness plot of FE frame is almost similar to experimental frames.

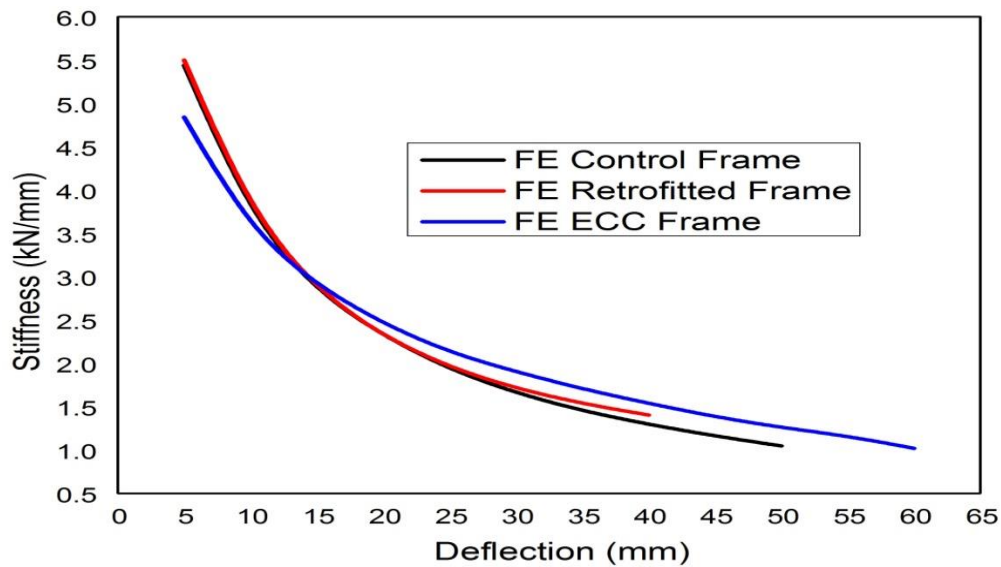


Figure 6.27 Stiffness of all FE frames

Stressed area plot showed the maximum stress developed at the column beam joint of ground floor. These results were almost similar to the experimental results. The maximum stressed area in frame is shown in Figure 6.28.

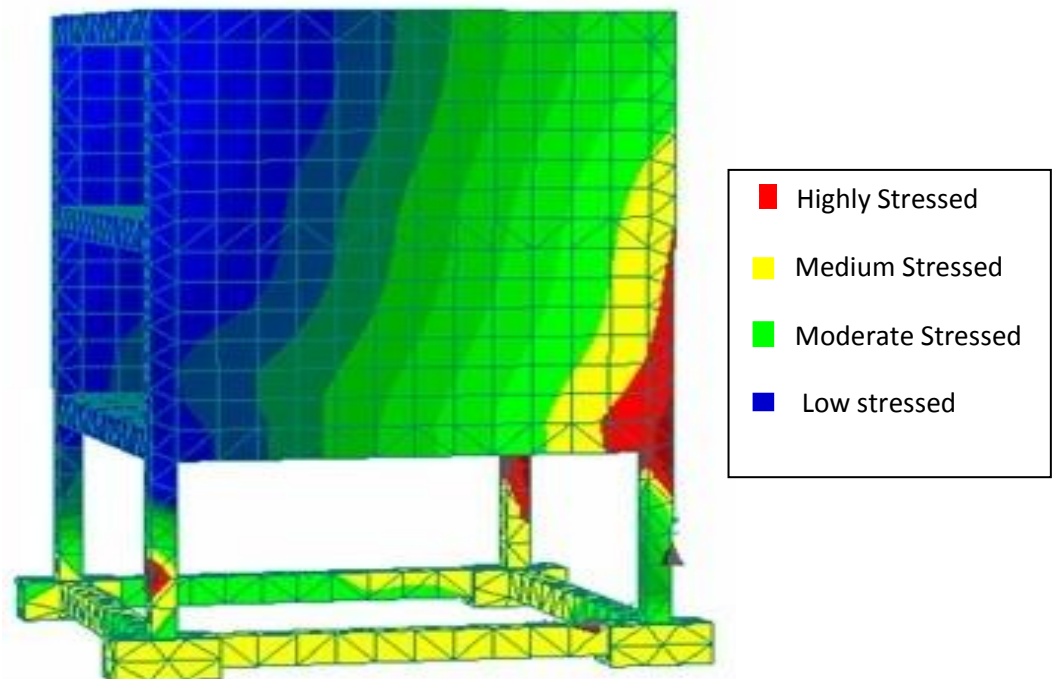


Figure 6.28 Stressed area shown in FE frame

6.4 CLOSURE

It can therefore be concluded from these results that the FEA shows cracks developed and deflection observed in the FE model is similar to the cracking pattern developed in experimental frame. In Control frame maximum cracks developed in the column beam joint of ground floor and in retrofitted frame maximum cracks observed below the retrofitted area. Stressed area plot showed the maximum stress developed at the column beam joint of ground floor. So resulting in FEA frame results were almost similar to the experimental results.

CHAPTER 7

CONCLUSIONS

7.1 GENERAL

The infill walls in a building frame provide higher stiffness and strength. But their effect on the structure performance is ignored due to lack of adequate information about the behaviour of frame with infill walls. Masonry infills are frequently used in RC frames in form of interior partition wall and exterior walls as a large proportion of total mass of the frame. Masonry infill walls in frame structures have been long known to affect the strength and stiffness of the infilled frame structures.

In seismic zones, ignoring the frame–wall interaction is not always on the safe side, because under lateral loads, the infill walls show increased stiffness. This increased stiffness can further results in change in the seismic demand due to the significant reduction in the natural period of the composite structural system. The composite action of the frame–wall system changes the magnitude and the distribution of straining actions in the frame members, i.e., critical sections in the infilled frame differ from those in the bare frame, which may lead to un-conservative or poorly detailed designs.

The damages observed in buildings and other structures during an earthquake initiates the need of exploring the possible new techniques for retrofitting and strengthening the structures. In addition to this up gradation of existing structure with respect to current codal recommendation or to satisfy the current requirements also give a need to strengthening of existing structures. Generally retrofitting of these types of buildings include addition of new members such as shear wall and bracing or strengthening existing members by Fibre Reinforced Polymer (FRP) like Glass Fibre Reinforced Polymer (GFRP) and Carbon Fibre Reinforced Polymer (CFRP), steel or RC jacketing etc. Strengthening of infill may also be done using FRP or steel plate bonding.

The aim of this work was to investigate the load deflection and dynamic behavior and characteristics of brick infilled RC frames subjected to lateral loading till failure. The failed brick infilled RC frames have been repaired using two different approaches. First by retrofitting with GFRP and CFRP composites and afterwards by replacing the damaged portion of frame with Engineered Cementitious Composite (ECC). Control frame, FRP retrofitted frame and ECC strengthened frame have been also simultaneously monitored Diagnostic technique. Non-linear FE analysis has been performed

brick infilled RC frame and FRP retrofitted brick infilled RC frame to compare experimental and analytical results. The calibrated FE model with experimental study will be very useful in performing parametric study for infill bare and retrofitted frames. Following conclusions are drawn based on the present study,:

7.2 Quasi Static Behavior

1. In the control brick infilled RC frame under increasing lateral load, maximum damage has been observed in the frame at soft storey. Only a small horizontal crack appeared in the brick masonry which shows that masonry infill provides better stiffness to the structure.
2. The comparison of hysteresis behavior of control brick infilled RC frame to the RC bare frame [Vimuttasoongviriya et al., 2009] under quasi static load shows that brick Infilled RC frame post yield performance is much better than the RC bare frame.
3. Hysteresis behavior of the FRP retrofitted brick infilled RC frame shows better post yield behavior and enlarged loop area under cyclic loading than the control brick infilled RC frame. Also the residual deflection is lesser in FRP retrofitted frame in comparison to control RC frame due to better damage tolerance. It shows that the bond mechanism between the FRP sheets and the concrete significantly affects the crack pattern and offers better elastic and inelastic behavior for retrofitted RC frames.
4. FRP retrofitted frame failed to show better initial and yield stiffness than the control frame. However with lesser initial stiffness, the FRP retrofitted frame provides better post yield stiffness retention as compared to control frame with improved load carrying capacity at same deflection level (say 50 mm). This proves that FRP strengthening may be an effective method in restoring the structural integrity of RC brick infilled frames.
5. Results of retrofitted frame shows that the use of FRP wrap for structural strengthening provides significant lateral load capacity, increasing to almost 11% as compared to control frame. The FRP sheet wrapped around the structural element in this manner are intended to provide external confinement and prevent crushing of concrete cover at larger displacement levels. The damaged frame has been retrofitted and restored its original strength. This is an achievement of the study and damaged frame showed better performance than the control frame after retrofitting even after using the yielded reinforcement.

6. Although the stiffness of the damaged RC structure has been regained significantly by retrofitting with GFRP and CFRP, the failure of the retrofitted frame is due to failure of columns repair below GFRP and column foundation joint below the CFRP wrap.
7. ECC strengthened frame showed 22.7% higher load carrying capacity than the control frame and possess better post yield behaviour. The maximum load has been attained by frame at a deflection of 55 mm as compared to control and FRP frame. The enlarged loop area showed better dissipation energy level and increased damage tolerance capacity of the adopted ECC strengthening technique. The damaged frame has been restored more strength than the original strength. This is an achievement of the study and damaged frame showed better performance 22.7% more than the control frame and 11% more than FRP retrofitting frame. So Repair with ECC is even better than the retrofitting with FRP.
8. Although ECC strengthened frame showed the higher loss in initial and yield stiffness it offered better post elastic stiffness retention as compared to control frame. It illustrates the influence of ductile behaviour of ECC and strengthening in non-linear performance.
9. Damage Index (calculated by Park-Ang method) of the FRP retrofitted and ECC strengthen frame has been reduced by 0.22% and 6.5% as compared to control frame. It shows the better performance of ECC strengthened frame over FRP retrofitted frame.

7.3 DYNAMIC BEHAVIOUR

Based on vibration monitoring and correlation of damage indexes based on the change in dynamic characteristics of structure, the following main conclusions can be drawn:

1. Damage Index calculated with respect to change in FRF magnitude (DI_{FRF}) of FRP retrofitted and ECC strengthened frame have reduced, indicating better performance as compared to control frame.
2. The natural frequencies of all modes of frames have been observed in a frequency range of 0 to 80 Hz. The change of resonant frequency is more for fundamental mode as compared to higher modes with increase in damage levels. The natural frequency of various modes were observed to be 7.0 Hz, 39.8 Hz and 79.7 Hz. These values of frequencies were reduced to 4.8 Hz, 35.2 Hz, and 75.7 Hz respectively after damage. Damping Ratio and the Damage Index DI_{FRF} was 0.424 and 0.832 respectively.
3. FRP retrofitted frame shows 69.6% reduction in FRF magnitude with progressive damage. On the other hand, damping ratio showed significant increase of 71.14% as compared to control frame. In general, the structural damping increases when stiffness of

structure decreases. the average natural frequency decreases by approximately 38.2% compared to the initial state. The upper floor FRF size was $1.287 \times 10^{-3} \text{ g / N}$, the second floor was $0.928 \times 10^{-3} \text{ g / N}$, and the first floor was $0.812 \times 10^{-3} \text{ g / N}$ first floor at 78.6% to the initial state. The DI_{FRF} loss rate was 0.767. Although the GFRP wrap increased the dynamic properties of the structure well.

4. The ECC replaced frame showed 75.95% reduction in FRF magnitude with progressive damage. On the other hand, damping ratio shows significant increase of 74.90% as compared to control frame.
5. The damage tolerance capacity of the frame has been estimated using the Damage Index (DI_{FRF}) based magnitude and it was shown that as the load increases in every cycle, the damage level also increases. Damage Index plot shows early damage frame in comparison to ECC repaired frame.

7.4 THEORETICAL RESULTS AND ANALYSIS

1. Nonlinear FE load-displacement plots exhibited similar trends with those of the experimental frame model, but the plot showed the FE model is slightly stiffer than the experimental frame. This may be attributed to micro-cracks present in the actual elements and assumptions of perfect bond between concrete and steel bars as well as between concrete and FRP.
2. The ultimate loads of the nonlinear FE models showed good agreement with the experimental tests. Crack patterns at the final state in the FE model correspond quite well with the failure modes of the experimental frames.
3. The Stiffness of brick infilled RC frame model obtained by FE analysis has been found to be 3.7% more than the experimental brick infilled RC frame, whereas, for FRP retrofitted brick infilled RC frame it is found to be 4.18% stiffer than the experimental frame and ECC frame appears 3.15% stiffer than experimental frame.

7.5 SCOPE FOR FUTURE WORK

1. The present study was limited to single RC frame, the same study should be conducted to establish the repeatability. Further, more number of frames should be tested to establish the effect various parameters such as ductile detailing, retrofitting techniques and loading arrangement on behaviour of frames.
2. The load-deformation behavior of brick infilled RC frame needs to be investigated by using cyclic loading as well.

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