Machine learning model of dhajji dewari wall panels and confinement effect on steel fiber reinforced HSC beams

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ABSTRACT

An Artificial Neural Network model was developed for lateral load performance of Dhajji Dewari retrofitted by inexpensive Carbon Fiber Reinforced Polymer retrofitting technique. This model was developed by considering five input variables (Strengthening, Opening, Retrofitting, Time and Load) for parametric analysis of timber walls. Developed Neural Network model is based on Layer Recurrent network type with two hidden layers and each hidden layer possess twenty nodes and one output layer having one node in the form of displacement. It can be used to compare the experimental data with the model data. Neural Network model consumes less energy and resources. It also saves time and waste materials. This developed Neural Network model is only applicable to experimental data covered by the paper followed however its exposure can be later extended on availability of data.

In second part of this research work focus was on Steel Fiber Reinforced High strength concrete (SFHSC) beams with variable confinements (CFRP and Steel sheet) to improve brittleness and flexural capacity. High strength concrete (HSC) has the added advantage over conventional concrete i, e. increased modulus of elasticity and stiffness which also increases its brittleness. To reduce its brittle behavior steel fibers are being used so that its application in construction industry may be increased. In this research work five high strength steel fiber reinforcement concrete beams with two different confinements (CFRP & Steel sheet) were made for comparison. Steel sheet and CFRP strip are also applied on beams to study improved flexural behavior.

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CHAPTER 1

INTRODUCTION

1.1 Background

Dhajji dewari is a conventional structure type found in the western Himalayas. It is a straightforward construction technology that can be easily built using nearby materials, timber, and masonry infill with mud mortar.

University degrees in engineering infrequently address such forms of construction and research into *dhajji dewari* buildings is negligible. Design guides are limited to anecdotal findings, common sense principles and rules of thumb. Although valuable, these have not been properly validated through thorough engineering testing and analysis.

This research project applied Neural Network (NN) on dhajji *dewari* wall panels to check whether NN could be accurately applied to determine its experimental performance when subjected to lateral loads.

High-Strength Concrete (HSC) has an increased modulus of elasticity, which increases stability and reduces deflections and is more brittle. At present, the use of High Strength Concrete (HSC) is on the rise and improvement in the brittle behavior through composite action is the need of hour to overcome the problems associated with High Strength Concrete. By adding steel fibers brittle behavior of high strength concrete can be reduced. It not only bridges the gap between the cracks formed within concrete but also imparts rigidness to concrete. Different methods have been studied till date to enhance the properties of concrete. Confinement of concrete is one such method as it increases the flexural strength of high strength concrete Beams.

Therefore, this research works on a relatively new composite of steel fiber reinforced High Strength concrete and different confinements on concrete beam specimens.

1.1.1 Dhajji Dewari

A traditional construction technique in Kashmir and Northern areas of Pakistan is Dhajji Dewari. This construction technique consists of a frame and some filling materials to fill the gaps between frame members. These frames are made up of timber and to fill the gaps between members mud and stones are used because these materials are locally available. Due to large availability of the materials in Northern areas of Pakistan such construction can be easily done. This type of construction is very sustainable due to its flexibility in use, affordable, abundantly available, environment friendly and durability. [1].

Dhajji means “Patch work quilt” and Dewari means “Wall” in Kashmiri Language [2]. When compared with modern building techniques Dhajji Dewari even with single layer of infill poses more effective use of materials. [4].

Shape of Dhajji structure is rectangular and is mostly built as a single-story building but in Indian occupied Kashmir multi stories can also be found. [5]. Similar construction type with little modification is used around the World. For example, in France (Colombage), Germany (Fachwerk), Turkey (Bagdadi, Himis and Dizeme), Greece, Italy and Portugal (Gaiola) and in Britain as (Half Timbered) [3]. All these traditional structures are mainly differentiated based on the choice of infill material [6].

These traditional structures behave well against Earthquake [2]. For construction of reinforced and combined masonry structure various codes, standards and guidelines are available globally that seeks to deliver minimum quality of design, material, and workmanship. In some parts of Pakistan masonry and concrete structures are considered complex and costly. In contrast, Dhajji construction, which uses conventional methods of construction and locally available material, is easy to construct and has performed well in Earthquake, but often ignored by decision makers and donors as there were only empirical evidence to justify its performance. Dhajji construction is non-ductile in nature and prone to extensive damage and collapse in moderate and severe Earthquake [1]. Therefore, it is necessary to study the vulnerability of Dhajji Dewari construction and to develop the cost-effective strategies and technical guideline for its restoration with long term goal of stopping considerable loss of life in earthquakes.

Inhabitants adopted traditional way for construction & rehabilitation after the experience of 2005 earthquake because:

- Dhajji Dewari construction performed well.
- Easy and economical way of construction
- Materials were locally available i e, wood, and stone.
- Reinforced Concrete material was inaccessible.

1.1.2 Steel Fiber Reinforced HSC Beams

High strength concrete (HSC) can be used in high-rise buildings to minimize the size of structural components. High strength concrete has the added advantage of a higher elastic modulus that can reduce the deflection. It also has a lower specific creep that makes it possible to use higher stresses associated with high strength concrete.

From ancient times, small and discrete fibers have been used to reduce brittle behavior. Reduced micro cracking, localized macro cracking and improvement in post-cracking strength and ductility of High strength concrete matrix can be achieved by adding discrete fibers. [7,8,9]. The Egyptians used straws to improve the cracking resistance of sun-dried mud bricks used for constructing huts. The feasibility of using fibers to improve the ductility and tensile strength of concrete, however, was not fully realized until the publication of classic reports from [10] followed by [11]. Soon after, the modern era of research and development of fiber reinforcement technologies began [12].

Brittle materials, such as concrete, are considered to have no significant post-cracking strength and ductility. When the principal tensile stress for plain concrete exceeds the tensile strength, cracks will develop. However, the addition of fibers has little effect on the behavior of concrete before cracking [13].

The fibers used in the concrete can be made from a variety of materials such as carbon, steel, plastic, glass, polypropylene, nylon, and cotton.

Steel fiber is one of the few fiber types that can satisfy all the above problems of HSC reasonably well. As a result, steel fiber reinforced concrete (SFRC) technology has grown into a mature industry over the last three decades and steel fibers have become the most used fiber in both research and industry [14]. Therefore, the focus of this work is on confined SFRC.

Recently in design codes, Steel Fibers Reinforced Concrete has gained acceptance for shear-critical members [15] but in earthquake active regions its usage is rare [16].

CFRP is highly strong and light plastic which is used where high strength-to-weight ratio and rigidity are required. It has its applications in aerospace, civil engineering, sports goods, automotive and numerous other industries. It is basically a composite material consisting of reinforcement and matrix. In construction industry CFRP is increasingly being used in retrofitting as it enhances the strength of concrete sections. The wrapping of concrete with CFRP sheets provides sufficient confinement to concrete which significantly increase its Flexural capacity, flexibility, and seismic behavior of beams.

Concrete Filled Tubular (CFT) members are being used as an important structural element in buildings specifically to provide resistance to earthquake [17]. Steel sheet confined members are used in civil engineering structures due to its good plasticity, increased capacity, good earthquake resistance and properties of ease in construction [18]. The CFT dissipates more energy as compared to RC elements and conventional steel elements [19]. In these members steel tube confines the inner concrete and the concrete delays the local buckling of the steel tube.

1.2 Problem Statement

1.2.1 Artificial Neural Network model for Dhajji Dewari

There is a need to develop a machine learning model on lateral load performance of timber walls (Dhajji Dewari) by considering different parameters which can be adopted for future study. There is a need to correlate the experimental data with the machine model for development of a data base for long term study.

1.2.2 Strengthening of Steel Fibre reinforced High Strength Concrete Beams

HSC although is being extensively used in construction industry due to its higher stiffness and modulus but is inherently brittle in nature. To reduce its brittle behavior steel fibers are being used so that its application in construction industry may be increased. To study its applicability in earthquake active regions steel sheet confinements are used. CFRP strip is also applied on beams to study improved flexural behavior.

1.3 Research Objectives

1.3.1 Artificial Neural Network model for Dhajji Dewari

- To develop a Machine learning model for prediction of lateral load performance of Dhajji Dewari walls.
- To carry out a comparison between machine learning model data and experimental data.

Research Significance

The purpose of this research is to make a machine learning model of Dhajji dewari (DD) by considering some parameters so that in future prediction can be made that can reduce project cost, physical efforts, and time.

1.3.2 Strengthening of Steel Fibre reinforced High Strength Concrete Beams

- To carry out experimental analysis on CFRP and steel sheet Confined steel fiber reinforced concrete beams for studying the flexural behavior of High strength concrete beams.
- To make a comparison of Steel Fiber Reinforced Concrete beams with variable confinements.

Research Significance

In this research, the behaviour of high strength concrete beams is studied with constant steel fibre content as 1.5 % and variable confinements. The research aims to improve the flexural strength, ductility, and stiffness of High Strength Concrete. A comparative behaviour is also studied in this research.

1.4 Thesis Overview

➢ Chapter 1

This chapter include the Background, introduction. Problem statement, objectives of research. Research significance of Dhajji construction and High strength concrete.

➢ Chapter 2

In this chapter literature review on Dhajji Dewari, other similar type of structures and application of Neural Network in Civil Engineering has been discussed.

This chapter also includes literature review on Steel fiber reinforced HSC members with different confinements.

➢ Chapter 3

This Chapter include research methodology, Experimental data, and developed Neural Network model.

This chapter also includes the discussion on experimental and machine learning model results of individual panels and their comparison with one another.

➢ Chapter 4

In this Chapter Research methodology and Experimental work on Steel Fiber Reinforced High Strength Concrete Beams is included.

It also includes discussion on experimental results of HSC beams and their comparisons.

➢ Chapter 5

This chapter contain conclusions of present research based on test results.

CHAPTER 2

LITERATURE REVIEW

2.1 Traditional Timber Construction

Traditional timber construction is found across the globe. For example, in France (Colombage), Germany (Fachwerk), Turkey (Bagdadi, Himis and Dizeme), Greece, Italy and Portugal (Gaiola) and in Britain as (Half-Timbered). All these traditional structures are mainly differentiated based on the choice of infill material. The type, nomenclature and seismic behavior of different traditional construction techniques exist in different parts of the world are discussed in detail as under.

2.1.1 Historic Perspective of Dhajji Dewari Construction

Dhajji Dewari is traditional construction found in the Northern areas of Pakistan and Kashmir [2]. Dhajji most commonly consists of a braced timber frame, the spaces left between the bracing or frames are filled with a thin wall (single Layer) of stone traditionally laid and plastered with mud mortar [3]. The timber frame of Dhajji houses consists of vertical and horizontal posts of relatively bigger cross sections and frame further divided into secondary vertical and horizontal posts of smaller cross sections. And finally, these secondary posts are provided with different types of bracing arrangements those later filled with stone or masonry. These houses are bolted with concrete or stone foundation to provide fixity with ground. Roofing system is quite simple in these houses; often wooden truss with corrugated sheets or simple corrugated sheets on flat wooden planks are used as roof [20].

These types of traditional houses consist of framework filled with burnt clay bricks. The composition of this type of structure is very different from that of a typical brick masonry structure and its superior performance has been proved to be earthquake resistant. In Dhajji Dewari houses the connected timber studs sub divides the infill which helps in detention of loss of masonry and resists the destruction of wall.

Figure2.1: A Typical Dhajji Dewari House [2]

The creation and further propagation of diagonal shear cracks and the possibility of out of plane failure of the masonry is halted by the closed spacing of the studs, even in higher stories and gable portions of the walls. In some structures usually the walls in lower portion are made of traditional brick or stone masonry and the upper stories are made as a Dhajji Dewari system. [2]

Figure2.2: Kashmir Museum [2]

After October 2005 earthquake, focused has been made to construct the damaged houses in faster way by viewing the availability of materials and cost. In this regard Dhajji houses were the best solution and these were acknowledged by many people [20]. However, Dhajji Construction is not the natural type of construction when compared to modern construction methods namely as RCC which consist of column and beam frames with brick work as infill.

Figure2.3: A Good Quality Dhajji House [20]

Pombalino Buildings

Lisbon earthquake 1755, had destroyed many areas including central region known as Baixa, the people of Baixa gathered the cluster of engineers to determine the best solution which retain their houses during earthquakes. The type of construction nominated by engineers was called as Pombalino wall, which was also called as gaiola or cage construction [21]. The gaiola construction or Pombalino system consists of timber frame with horizontal and vertical square cross sections about 10-12 cm size and used in interior parts of buildings, with cross slope which act as internal bracing. The empty space between these walls was then filled with bricks and mixture of stones in different arrangements. The walls after filling with rubble stones were covered with plaster. The front side of Baixa buildings was rebuild with masonry wall having thickness about 60 cm and interior of wall had timber frame as well [22]. The significance of these walls lies in the fact that they can resist the lateral loading of earthquake by enhancing the structural ductility.

Figure2.4: Gaiola in Pombalino Buildings [21]

Casa Baraccata Buildings

Similar to Pombalino buildings another type of traditional timber frame structure was developed in Italian cities of Calabria and Sicily and was called as Casa Baraccata building systems. This type of structure was developed in response to the occurrence of earthquakes in the region. The origin of this type of structure coincides with the Gaiola in Portugal. During the 19th century and the first two decades of the 20th century Casa Baraccata due to manifested applications for the seismic resistance it became the basis of instructions for construction practices in Italy [23]. This traditional building construction type was the only alternative for inhabitants of Europe and other parts of the world against seismic actions. It was the first time that these traditional buildings were adopted as earthquake resistant structures on government level.

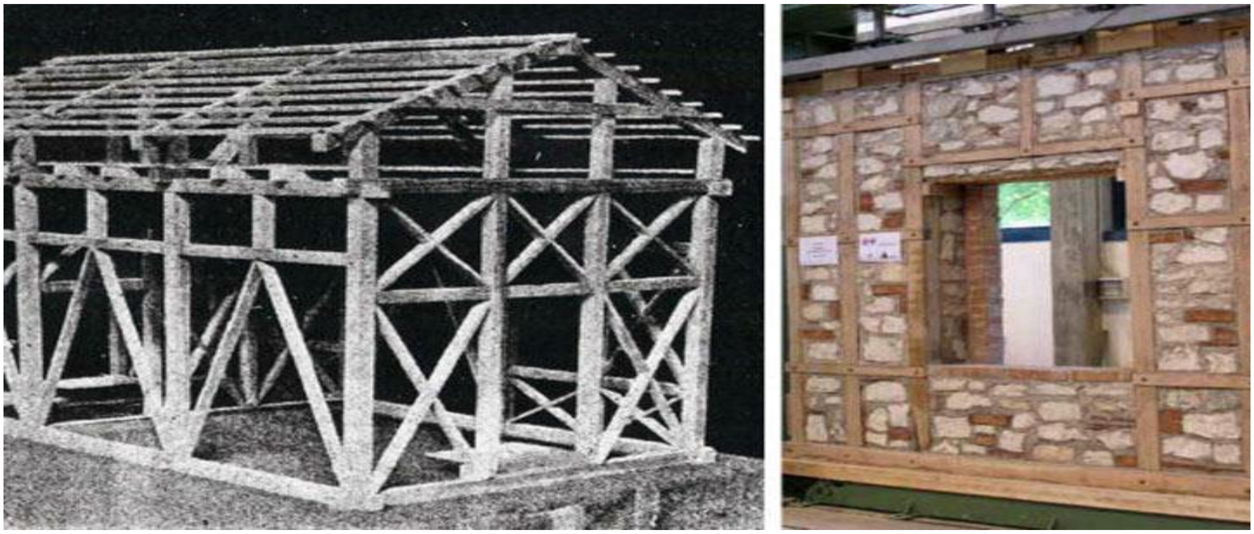

Figure2.5: Casa Baraccata Buildings [23]

Himis Construction

Himis is another type of commonly used traditional buildings that is found in different parts of the world. The masonry pattern in this type of building is different from the traditional bearing wall masonry. The timber frame is essential for providing the framework for the infill masonry. It consists of one layer of brick masonry, or a thin sheath of rubble stone mixed in mud of lime mortar. For decorative and aesthetic purpose bricks are placed at different angles on the front side. The thickness of the wall consisting of both timber and brick is 10 to 12 cm. Himis is in common use in Turkey, but it was overtaken by the reinforced concrete rapidly in the beginning of the 20th century [22].

During the August 17th, 1999 earthquake in Turkey. The epicenter was east of Istanbul at just 100 kilometers away. More than one third of houses were destroyed by the earthquake and most of them RC structures [24]. While on the other hand most of the Himis buildings which were situated in the heart of the city were almost undamaged during earthquake, while some were critically damaged. Turkish researchers conducted surveys and detailed statistical studies in the earthquake affected area of the district. It was found that there was a great difference in the number of RC buildings affected by the earthquake and the unaffected Himis buildings [25].

Figure2.6: Himis House (Turkey) [25]

Bagdadi Construction

Bagdadi construction is another type of construction and fairly found in areas where Himis is common. This type of construction consists of short and rough pieces of timber for infill purpose which cover with plaster and form a solid wall. The significance of Bagdadi houses lie in the fact that they are light in weight, uses scrap wood, easy and economical to build. The main defect of these walls is the attack of insects which causes bigger rots to deteriorate [22].

Half-Timbered Structures

Half-Timbered structure traditionally found in Roman Empire, also referred to as Opus Craticium [26]. Half-timbered structures consist of timber and masonry materials. Mostly timber wall is used in construction with masonry wall. According to Tampone [27] timber elements were used in construction of Knossos and Crete located in Minoan forts to support the masonry work.

Different type of timber configuration was used in half-timbered construction, but the mean and common method was that the timber members can resist tension, and masonry members resist compression thus making a perfect assembly to resist lateral loads. However, using timber in conjunction with masonry not only provide confinement to structure but also improves mechanical properties against lateral loading. [28, 29].

Figure2.7: Half -Timber House (Britain) [27]

Fachwerk Construction

Fackwerk construction is very common in Germany. Different types of timber frames found and are classified by the number of stories and the geometric shapes. This type of construction was introduced in the 7th century, but it took till 16th and 17th century to gain popularity. It has three main types (Alemannic, Lower Saxonian and Franconian), which are differed from each other due to dimensions, spacing of the elements and the nature of the framing [30].

Figure2.8: Fachwerk House (Germany) [30]

Previous Studies on Performance of Dhajji Dewari under Lateral Load

The appropriate literature survey shows that several research studies on the lateral load response of timber-braced frames have been conducted as under.

Graca Vasconcelos et al. [31] conducted experiments on typical Gaiola wall subjected to in-plane cyclic loading to perceive its mechanical behavior and to assess its performance under seismic actions. Cyclic test was performed and for this purpose three types of frames each having different typologies was analyzed. (1) Timber frame unreinforced and having no infill; (2) Glass Fiber Reinforced Polymer sheets (GFRP) used on connections of timber frame having no infill; (3) Brick masonry was used as infill in timber frames. Tests on typical "Gaoila wall" have resulted that structural integrity of all walls in all cases dissipated energy over many cycles. [31].

Figure2.9: Gaiola Wall after Test [31]

Ali et al. [32] conducted Quasistatic cyclic test on typical "Dhajji wall" in Earthquake Engineering Centre, UET Peshawar. It was among the first few full scale Dhajji walls which were tested and was consider very helpful in finding drift limits, hysteretic response, viscous damping and strength envelope of Dhajji Dewari walls. It was observed that Dhajji wall resist numerous load cycles before failure. Thus, confirm that Dhajji Dewari retains remarkable resilience against lateral loading. Further it was proposed that this resistance is essentially offered by the timber frame with very less contribution from infill material and the most critical part of the system are the connections between the vertical and horizontal posts [32].

Ahmad used the experimental data of Ali [32] to formulate numerical model and to assess the seismic performance of Dhajji Dewari walls. He did time history analysis using equivalent frame approach to analyze the 2D Dhajji Dewari walls. He concluded that the Dhajji Dewari structures placed near to the epicenter of a high magnitude earthquake will need retrofitting while those houses positioned away from the epicenter will have a better performance and less damage. He added that more research is required to rectify the numerical model so that the results obtained are distinguished and more accurate. Also, comparative study is required in the region to investigate the relative performance of different regional structural system [33].

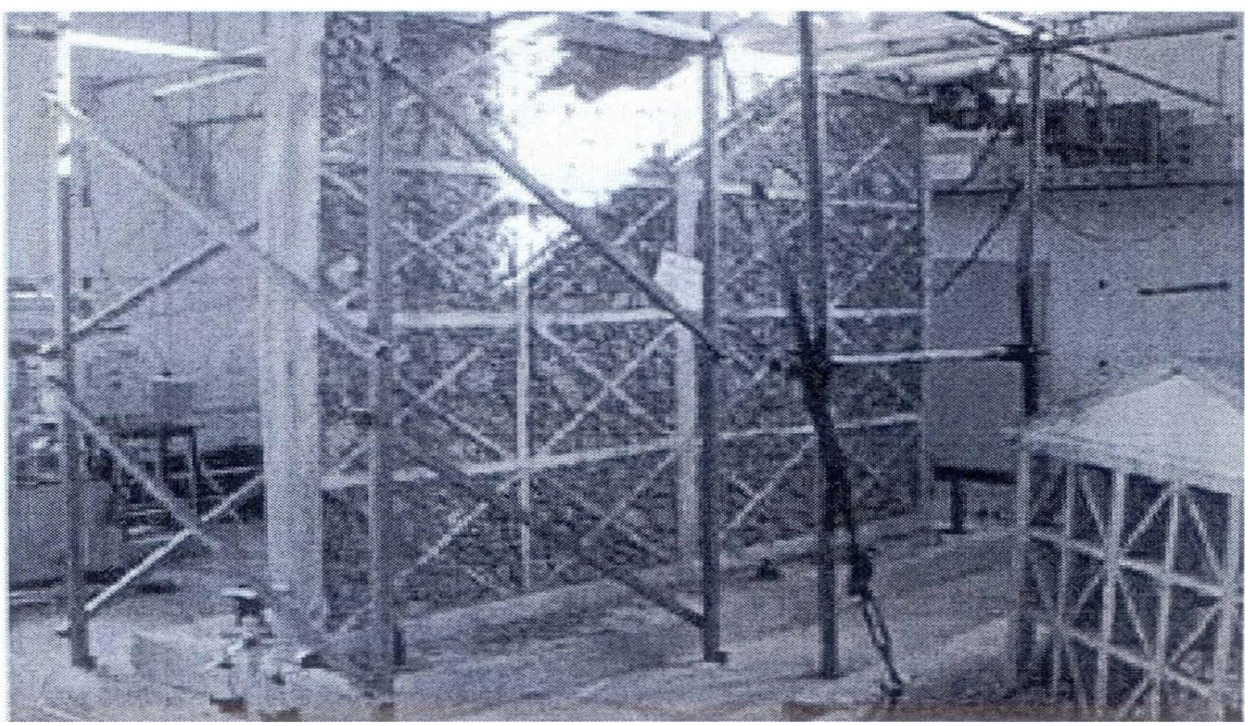

Figure2.10: Full Scale Dhajji Wall Panel Test at University of Peshawar [32]

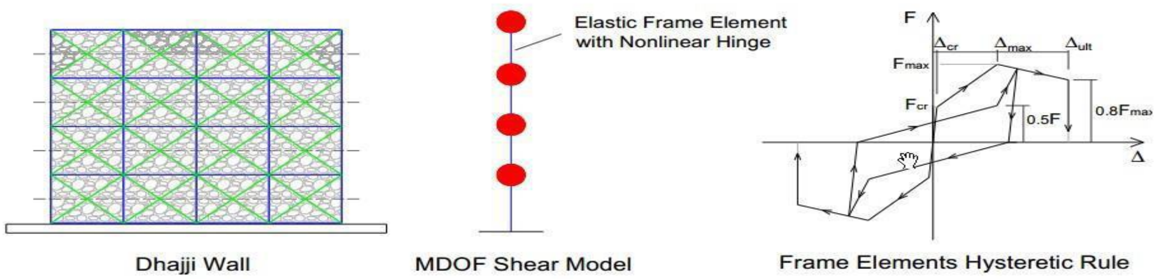

Figure2.11: Numerical Model of Dhajji Wall [33]

Shah et al [34] at NIT Srinagar also performed experiments on Dhajji Dewari frames to check their seismic resistance abilities. Further tests were performed to check that which bracing arrangement gives superior performance. Lateral load was applied to simulate the earthquake loading. It was decided that the joints are the most critical points also by increasing bracing by 1% increases the strength by 3%, while nailing and broad-shouldered the joints gave significant increase in load carrying capacity of the timber frame [34].

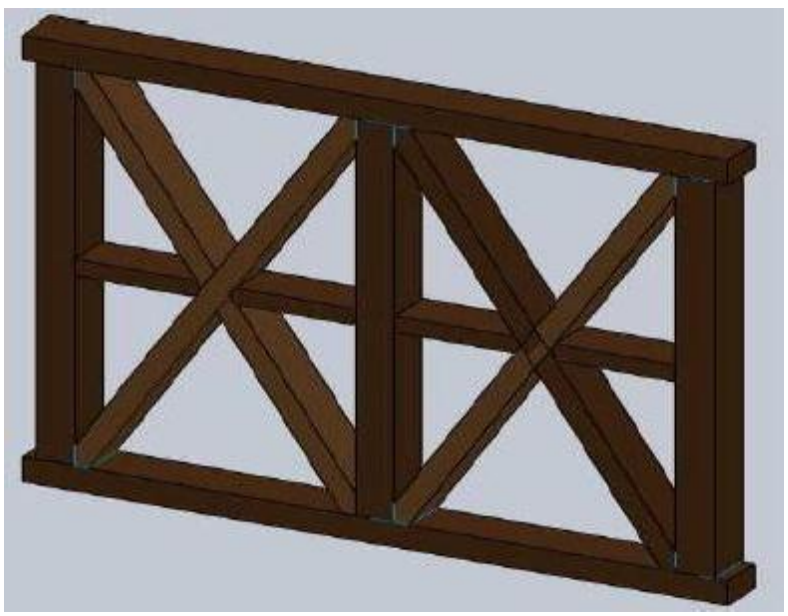

Figure2.12: Best Bracing Type by Shah [34]

2.1.2 Neural Networks

Neural Network is an advanced type of computing, inspired by the biological structure of neuron & internal operation of the human brain. Artificial Neural Network is used to learn pattern & relationship in data. This data may be the monthly electricity bills of past months to forecast the bill of coming months in a multi-story building. The aim of Neural Network is to impersonate the human ability to adapt the changing circumstances & the current environment. Associated terminologies of Biological & Artificial Neural Networks are given in figure 2.13.

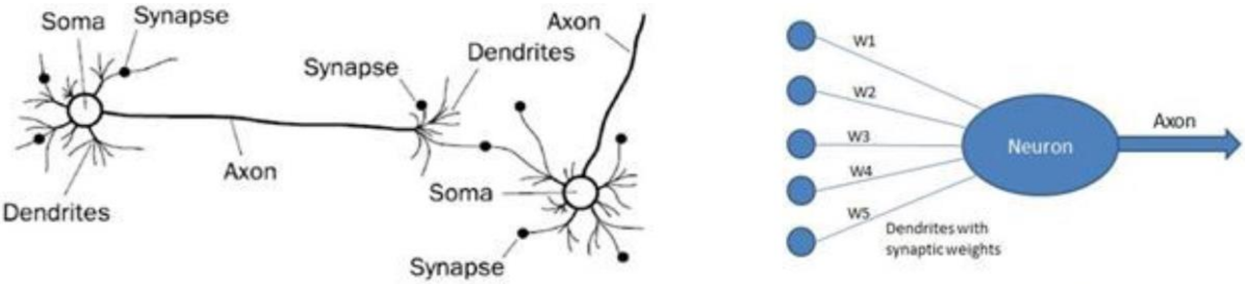

Figure 1.13: Biological and Artificial Neural Network systems

The functions of the main elements are given below.

Dendrite: Acts as signals receiver from other neurons

Soma: Sums all the incoming signals

Synapse: Transfers the signals

Axon: Transmits output signals

Artificial Neural Networks consists of several neurons which act as processing units analogous to neurons in the brain. Each node has a node function, associated with a set of local parameters determines the output of the node, given an input. Modifying the local parameters may alter the node function. Artificial Neural Network is thus an information processing system. In this information processing system, the elements called neurons process the information. The signals are transmitted by means of connection links. The links possess an associated weight which is then multiplied along with the incoming signals (input) for any typical neural network. The output signal is obtained by applying activations to the network input. The neural network can generally be a single layer or a multilayer network. The constitution of a simple neural network is shown in figure 2.14.

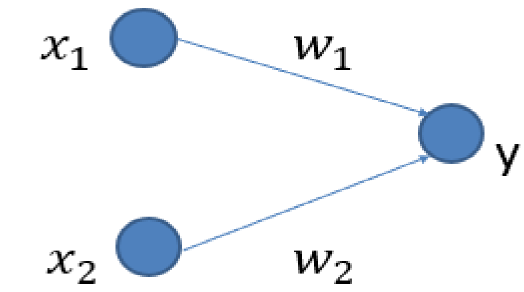

Figure2.14: Single Layer Neural Network

Figure 2.14 is showing an Artificial Neural Network having two input nodes (x_1, x_2) and one output node y. Here w_1 and w_2 are weights interlinked with the input and output nodes. Figure 2.15 is representing composition of a Multi-Layer Neural Network system.

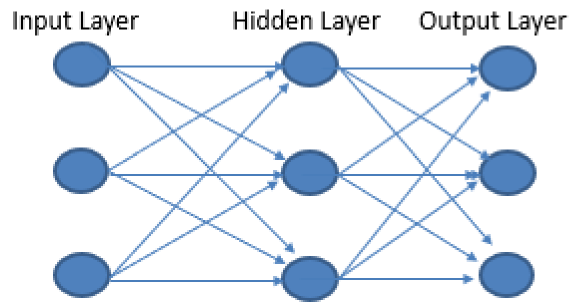

Figure2.15: Multi-Layer Neural Network

This structure consists of an input layer, output layer and a hidden layer of neurons.

Basic Building Blocks of ANN

- Architecture
- Training and Learning
- Activation Function

2.1.3 Application of Artificial Neural Network (ANN) in Civil Engineering

Oztas and Pala worked to represent possible applicability of neural networks (NN) to predict the compressive strength and slump of HSC. For this purpose, 187 various concrete mix designs of High strength concrete were collected from literature and an Artificial Neural Network model was constructed, trained, and tested. The data used in the Artificial Neural model was organized in form of seven input parameters: w/c ratio, water content, fine aggregate ratio, fly ash content, air entraining agent, superplasticizer, and silica fume replacement. An Artificial Neural Network model was produced in MATLAB to predict the slump values and compressive strength of the High strength concrete. They found that percentage errors for compressive strength was less than 1,956,208% and for slump value was less than 5,782,223% and R2 values of both compressive strength and slump for test set were 99.93% and 99.34% respectively. They finally concluded that Neural Networks have capacity to predict compressive strength and slump value [35].

Sarkar worked on Application of ANN for optimum analysis and design of short axially loaded columns and proposed a Feed Forward Network with one hidden layer & with two and a half times the number of nodes in the input layer produces plausible results [36].

Gupta and Sharma reviewed some of the applications of neural network in the field of structural analysis and design. They concluded that the neural network modeling gives a more effective and precise method in comparison of the regular techniques. They also suggested that Neural networks have capability to produce the difficult input/output relationship without using complex and expensive programming [37].

2.2 High Strength Concrete (HSC) Beams

The steel fibers act as crack arrestors not as crack inhibitors. Post-Cracking tensile response of concrete can be improved by adding steel fibers. In the concrete matrix Steel fibers are randomly distributed and are discontinuous. These characteristics permit cracks to spread out in any direction to be bridged by the fibers and allows improved stress transfer across all cracks, improving post-cracking shear and flexural resistance [38].

Use of fibers helps in bridging the cracks and reducing crack width and crack spacing resulting in increased post-cracked ductility and energy absorption capacity [9]. In compression, like the tensile response, the addition of steel fibers primarily improves the post-peak ductility and toughness [8].

The addition of steel fibers only marginally improves the compressive strength, in the range of 0% to 15% [8, 39]. With increased fiber content in concrete, the strain capacity is enhanced which ultimately leads to greater load bearing capacity of concrete in compression.

The shape and aspect ratio of steel fiber used can also be an influential factor in behavior of SFRC. With small length fiber usage, the number of fibers increase greatly keeping the aspect ratio same. This increased number of fibers leads to improved crack bridging and stress transfer across the cracks. This aspect is also shown by [40,41] where short fibers lead to improvement in overall response. Shape of fiber also influences the behavior of SFRC. Fibers may be straight, deformed silt, end-hooked, flattened-end, machined chip and melt extract as shown in Figure 2.16 below. As at failure steel fibers tend to pullout from concrete matrix instead of yielding, it is beneficial to use fibers which are deformed. Their mechanical anchorage plays a role in overall behavior of SFRC [42].

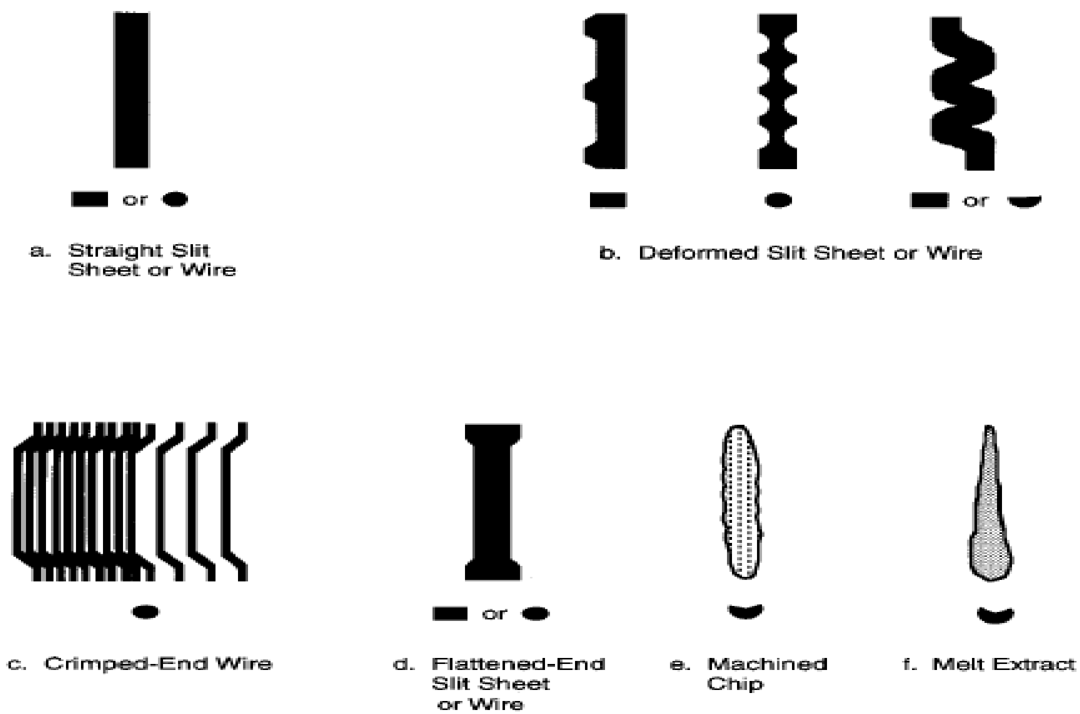

Figure2.16: Different shapes of Fibers [42]

In the concrete matrix the efficiency of steel fibers highly depends upon orientation of fibers in the concrete. It is difficult to determine the exact orientation, but it is known that the maximum benefit would occur with fibers parallel to the direction of tensile stress and minimum benefit would occur with fiber orientation perpendicular to applied tensile load. This is because the number of fibers bridging the crack would increase with parallel orientation to load. In the modeling of SFRC, many accounts for the effects of fiber orientation and its impact on the efficiency of fibers through a fiber orientation factor [43].

A higher compressive strength also increases the bond strength of fiber and matrix. The fibers will also experience a higher interfacial shear stress at the onset of cracking due to the higher first-cracking strength. Due to this more bond slip may occur before fibers can be fully de bonded [40].

[44] undertook a comprehensive experimental program containing 237 samples for deducing Confinement Model for FRP-Confined High-Strength Concrete. This database was augmented with another database of FRP-confined normal-strength concrete (NSC), which consists of 739 test results. Existing stress-strain models were studied; their strengths and weaknesses were identified. A novel design-oriented model for FRP-confined concrete was developed based on the

database summarized in the experimentation program. The model was tested, and it was observed that it performs significantly better than any of the existing stress-strain models of FRP-confined concrete in predicting the ultimate conditions of FRP-confined HSC.

[45] carried out research on Axial and lateral stress–strain model for FRP confined concrete. They developed a theoretical model calculating the lateral strain, axial stress, and confining stress in FRP confined concrete. They applied the model to FRP confined concrete specimens tested by others, of a total of 174 specimens, to verify its correctness. In comparison to the experimental results stated in the literature their model predicted lateral strain and axial stress with absolute error range of 20.2% and 8.4%. Their model can be used to analyze FRP confined concrete.

Experimental Investigations on 33 concrete filled steel tubular (CFT) beam-columns were made with different strengths of concrete (40 MPa and 90 MPa) and steel (400 MPa, 500 MPa and 780 MPa). Variations of width to thickness ratios of steel tubes were also considered. After investigations it was concluded that higher steel thickness and strength improved the behavior, but higher concrete strength led to negative effect on beam-column samples [46].

CHAPTER 3

MACHINE LEARNING APPLICATION IN DHAJJI DEWARI

3.1 Methodology

To develop a Machine learning model for comparison of Dhajji Dewari (DD) panels with experimental results five different variables (strengthening technique, retrofitting, time, load and opening) were considered.

3.1.1 Experimental Data

Table 3.1 is representing experimental data of two research papers followed in this research work.

Table 3.1: Experimental Data [47,48]

Panels type	Panels	Retrofitting Type
Pre-Retrofit Panels	DDW1	-
	DDW2	-
	DDW3	-
	DDW4	-
	DDW5	-
Post Retrofit Panels	RDDW1	Single layer of CFRP wrap
	RDDW2	Double layer of CFRP wrap
	RDDW3	Double layer of CFRP
	RDDW4	Double layer of CFRP wrap
	RDDW5	Double layer of CFRP wrap

3.1.2 Types of Panels used

Pre-Retrofit Panels

Pre-retrofit Dhajji Dewari (DD) panels are shown in figure 3.1.

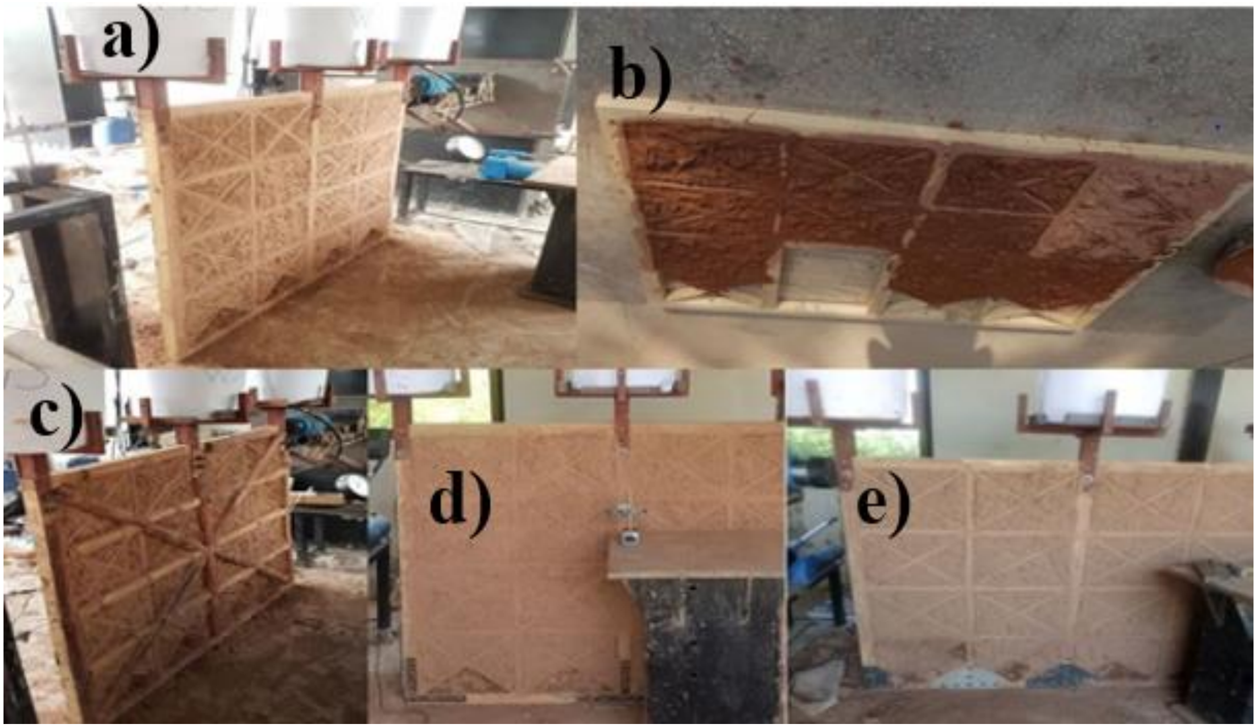

Figure3.1: Pre-Retrofit wall panels [47]

Here Panel (a) is representing reference wall panel. This wall was taken without any strengthening technique as shown in figure 3.1 and is considered as reference wall to check the behavior of conventional wall and for comparison purposes with strengthened walls. Panel (b) is representing an opening a mimic to door or window in a wall. Panel (c) was strengthened with Bamboos on periphery of wall and along tension and compression struts for purpose of enhancing lateral load performance by strengthening the joints. Detail is shown in figure 3.1. Panel (d) was strengthened on the connection of main horizontal and vertical posts of wall with metal strips. Two types of strips were used and bolted on both sides of wall. The shape of strips used was depending on the location of connection i.e., L-shaped strips were used on bottom corner joint and T-shaped strips were used on bottom middle joint as shown in figure 3.1. The size of strips was random, length was taken according to spacing of bolts and by keeping in mind the timber strength, thickness was 2 mm and width was selected according to width of post. Panel (e) was strengthened with metal gusset plates on connection of main horizontal and main vertical and secondary vertical posts. Two types of plates were used and bolted on both sides of wall. The shape of plates used was depending on the location of connection i.e., L-shaped plates were used on bottom corner joint and triangular shaped plates were used on bottom intermediate joints as shown in figure 3.1. The size of plates was random, length was taken according to spacing of bolts and it was less than metal strips length reason behind was the large surface area of plates, thickness of plates was 2 mm and width was selected according to width of posts.

Post Retrofit Panels

Post-Retrofit panels are shown below in figure 3.2.

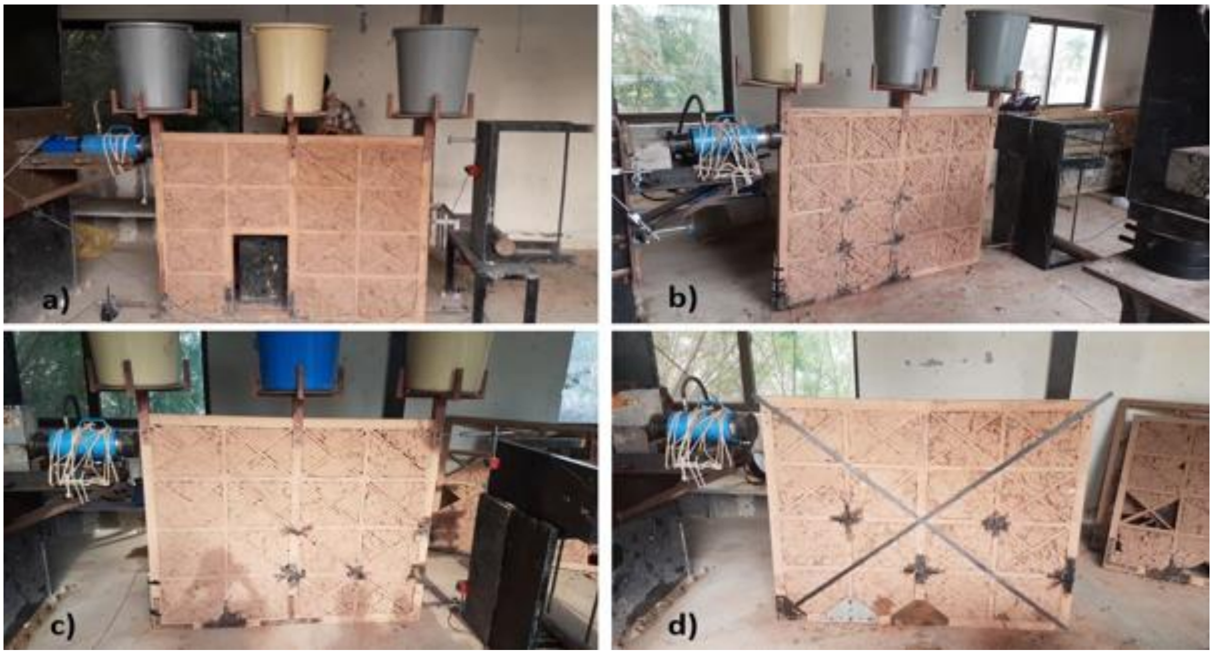

Figure3.2: Post-Retrofit wall Panels [48]

In Figure 3.2 Panel (a) is representing Retrofitted wall panel RDDW 2. It can be seen that three joints were wrapped with CFRP for retesting. Panel (b) is representing RDDW 3, CFRP wrapping can be seen at the joints that were failed during testing. Another change was made that Bamboo bracing was removed during retrofitting. Panel (c) is representing RDDW 4. CFRP wrapping was provided for retrofitting. Bracing of metal strips were kept same. RDDW 5 can be seen in figure 3.2 (d) part. Here gusset plate bracing remained as it is and CFRP wrapping was also provided at failed joints additional CFRP strip was provided in diagonals.

3.1.3 ANN Model Making

Figure 3.3 is showing the Flow Diagram of the process of ANN modeling through MATLAB. It includes the collection of data of all ten pre- and post-retrofitted panels then it comes modeling phase after that selection of Network type, selection of number of hidden layers, selection of number of output layer nodes and then selection of input parameters. After selecting all these parameters training and validation of input data starts.

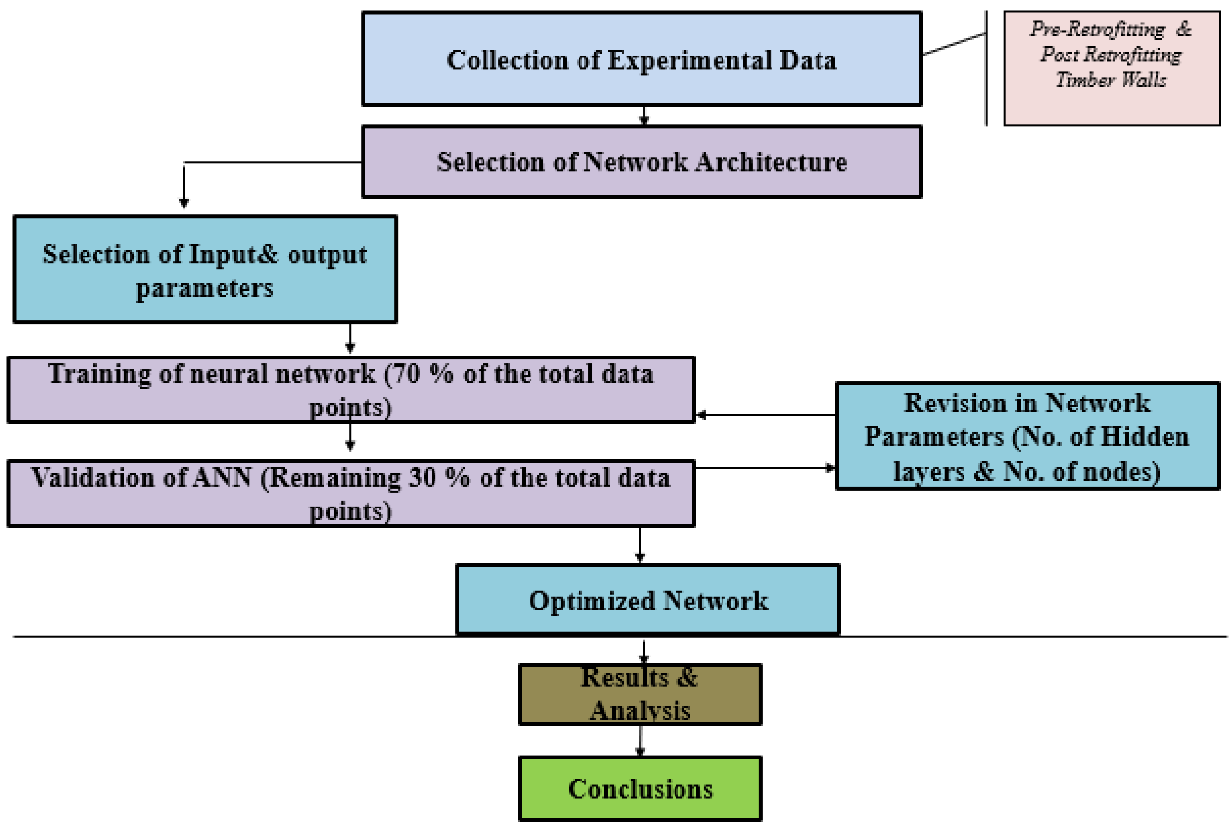

Figure3.3: Methodology for ANN model

3.1.4 Input Variables

All the input variables considered for modeling and training of input data are given in Table 3.2. The reason for selection of these variables was because these were the variables selected by the authors of the papers followed.

In Table 3.2 some random constant values are selected for some variables like strengthening, retrofitting, and opening. These values do not have any numerical significance but only to differentiate the panels properties i.e., 0 is representing to no strengthening 1.5 number is for bamboo, 2 is for metal strip and for gusset plate 2.5 is used.

Table3.2: input parameters [47,48]

Nomenclature of panels	Strengthening	Retrofitting	Opening	Time (Sec)	Load (KN)
DDW 1	NONE [0]	NONE [0]	NO [0]	Actual Values	Actual Values
DDW 2	NONE [0]	NONE [0]	YES [1]		
DDW 3	Bamboo [1.5]	NONE [0]	NO [0]		
DDW 4	Metal Strip [2]	NONE [0]	NO [0]		

DDW 5	Gusset plate [2.5]	NONE [0]	NO [0]		
RDDW 1	NONE [0]	SINGLE LAYER FRP FABRIC [1]	NO [0]		
RDDW 2	NONE [0]	DOUBLE LAYER FRP FABRIC [2]	YES [1]		
RDDW 3	NONE [0]	DOUBLE LAYER FRP FABRIC [2]	NO [0]		
RDDW 4	Metal Strip [2]	DOUBLE LAYER FRP FABRIC [2]	NO [0]		
RDDW 5	Gusset plate [2.5]	DOUBLE LAYER FRP FABRIC +FRP STRIP [3]	NO [0]		

3.1.5 Developed Neural Network model

Developed Neural Network model is shown in figure 3.4. In this model Layer Recurrent Type network was selected. Five input variables (Strengthening, Opening, Retrofitting, Time, and Load) were selected on the basis that author of the paper followed selected the same parameters. Two Hidden layers were selected with twenty nodes in each Hidden layer. All these were selected through trial-and-error method where model data and experimental data matched. This model also includes an output layer with one node in the form of displacement.

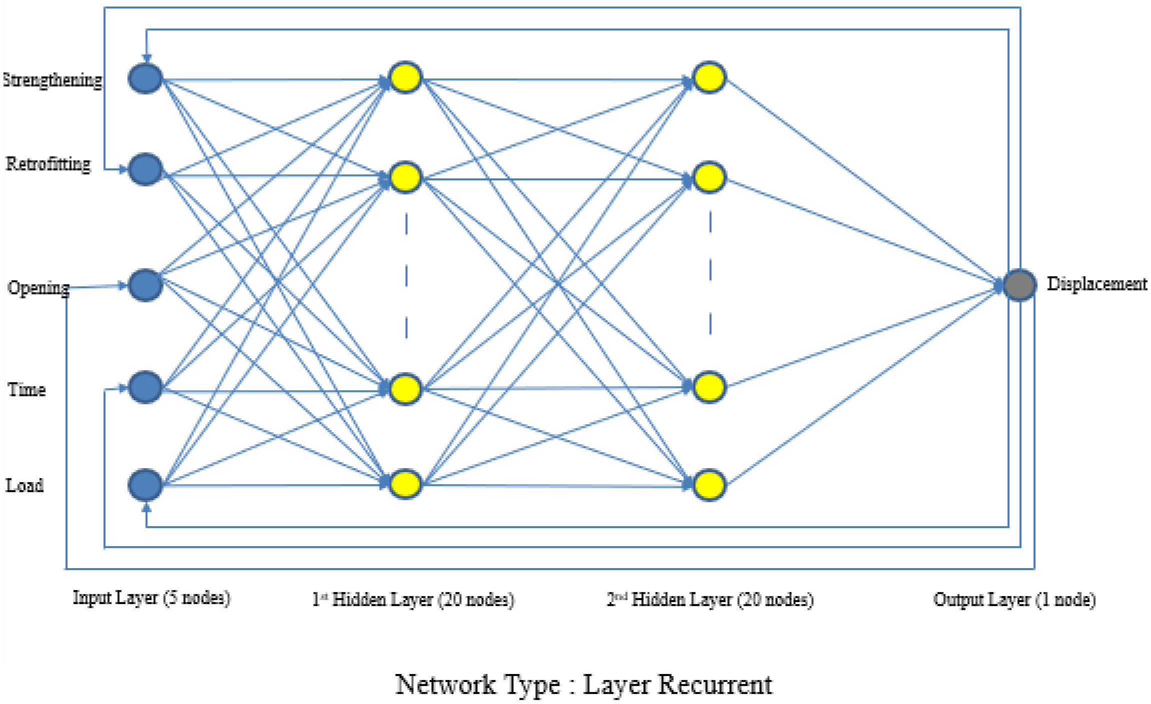

Figure3.4: Developed ANN model

3.1.6 Parameters studied for comparison of model data and experimental data

Load-Deflection relationship

When we apply load on wall panel, it will deflect, increasing load will increase deformation. A load–displacement curve measures the extrinsic properties of the test sample. The main parameters assessed through a load-displacement curve are stiffness, fatigue, and ultimate load and displacement.

The concept of equivalent-elastic perfectly plastic system proposed by (Muguruma, 1991) was used to locate the yield point on load displacement curve. Similarly, ultimate load is located on the load displacement curve as well.

Energy Dissipation

The energy dissipated by the sample walls is an important parameter of a structure. From the load-displacement or stress-strain curves energy dissipation can be calculated.

Higher value of energy dissipation represents ductile failure, on the other hand higher values of energy dissipated by the specimens can also be due to the strengthening techniques used and their effectiveness.

Energy dissipation of a structure depends upon a few factors, some of the factors involved are.

- crushing of wood
- base-lift and rocking
- the friction along joints
- crack propagation
- formation of new cracks

The type of failure of dhajji dewari wall samples also affect the energy dissipated.

Energy dissipation is calculated as the area enclosed by the load displacement curve up to ultimate point. Whole area is divided into different segment and calculating sum of all the segment gives the energy dissipated by system. Area under each segment is calculated using trapezoidal formula as given in equation 3.1. The detail is shown in figure 3.5.

$$\text{Area Under Curve} = (\frac{a+b}{2}) * (d-c)\Big) \qquad (3.1)$$

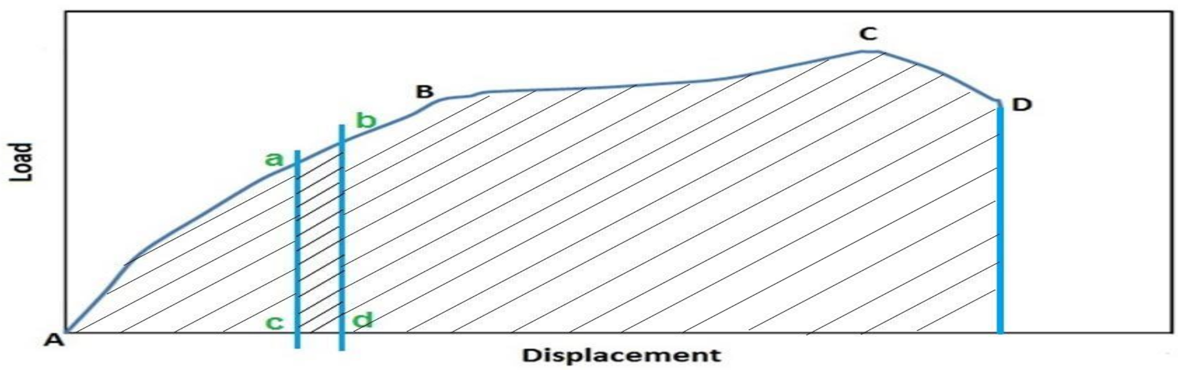

Figure3.5: General Load vs Displacement Diagram

Ductility and Response Factor

Main factors taken into consideration while designing a structure in seismically active zones are Ductility and Response factor. Ductility is the property of a structure which lets the structure deform and bend beyond its yield strength without toppling and collapsing the whole structure.

The response factor represents the seismic capability of a structure. Seismic forces are dealt with more effectively in ductile structures. The response factor of the test samples in both cases was calculated using the relationship given by Paulay and Priestley (Paulay and Priestley 1992).

Ductility is calculated as the ratio of ultimate over yield displacement as given in equation 3.2. Structural properties and type of material control affect the ductility.

$$\mu d = \frac{\Delta u}{\Delta y} \qquad (3.2)$$

Where, μd is ductility factor, Δu and Δy are ultimate and yield displacements, respectively. Muguruma et al. concept of equivalent elastic-perfectly plastic system was used to determine the yield and ultimate displacement on load displacement curve shown in figure 3.6.

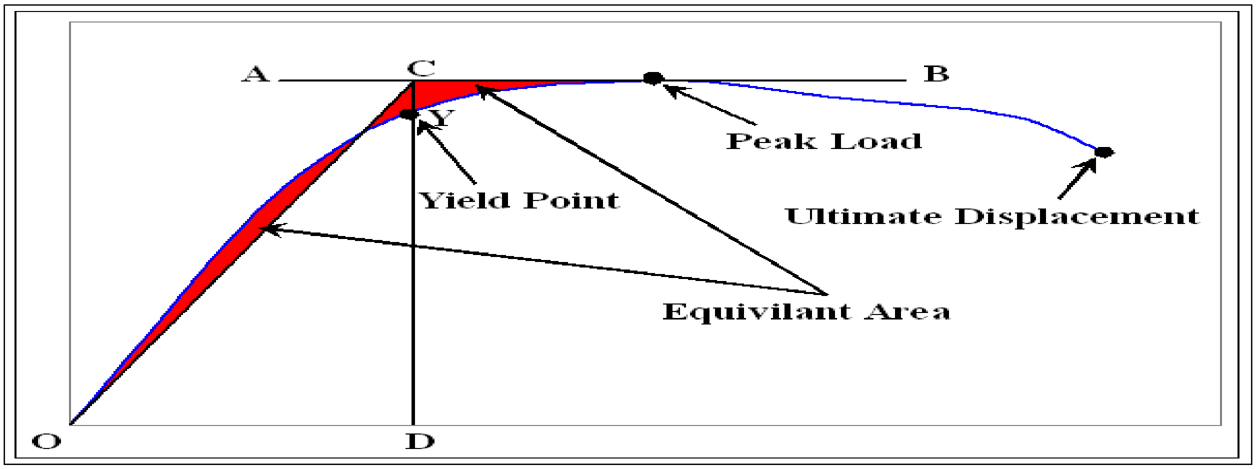

Figure3.6: Muguruma model

Relationship to calculate the Response Factor is given in equation 3.3.

$$R_f = \sqrt{(2\mu_d - 1)} \qquad (3.3)$$

3.2 Results and Discussions

Comparison of results of Dhajji Dewari walls with Artificial Neural Network model is also carried out in this chapter. The different performance parameters of Dhajji wall like load displacement behavior, energy dissipation; ductility and response factor are determined and compared in detail. The group wise discussion on experimental results is as under:

3.2.1 Load Displacement Behavior

Load vs Displacement comparison of model data and experimental data for Pre-Retrofit Dhajji Dewari Wall 1 (DDW 1) can be seen in Figure 3.7. Figure shows that in the start there is small deviation of model data also small deviation can be seen nearer to end.

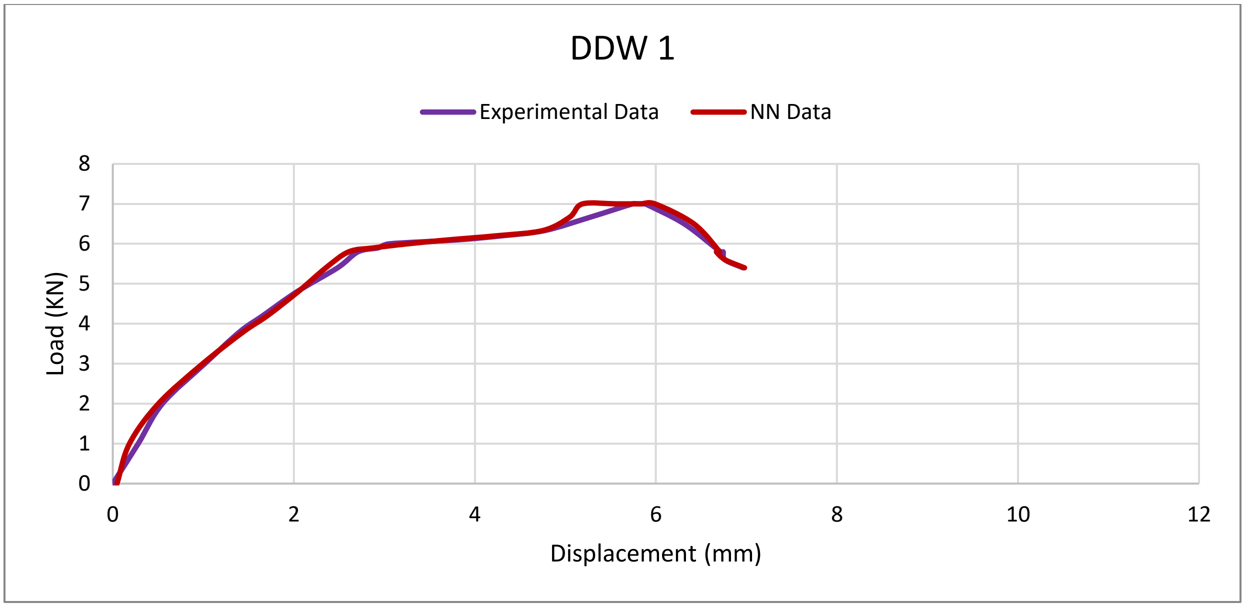

Figure3.7: Load-Displacement of DDW 1

Figure 3.8 is representing load vs displacement comparison between model data and experimental data of Dhajji Dewari Wall 2 (DDW 2). Start of both data is not from the same point. Much more deviation of curve from DDW 1 can be seen.

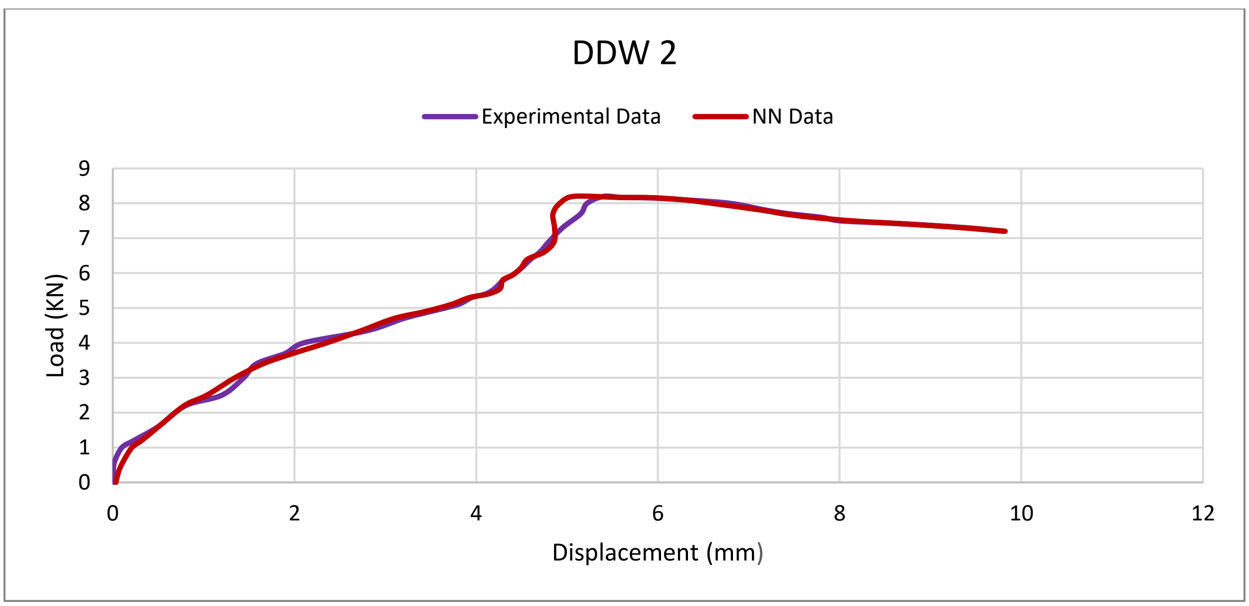

Figure3.8: Load-Displacement of DDW 2

Figure 3.9 and figure 3.10 are representing load vs displacement curves for comparison of model data and experimental data od Dhajji Dewari wall 3 (DDW 3) and Dhajji Dewari wall 4 (DDW 4) respectively. Almost fully matched model data with experimental data can be seen in these two figures.

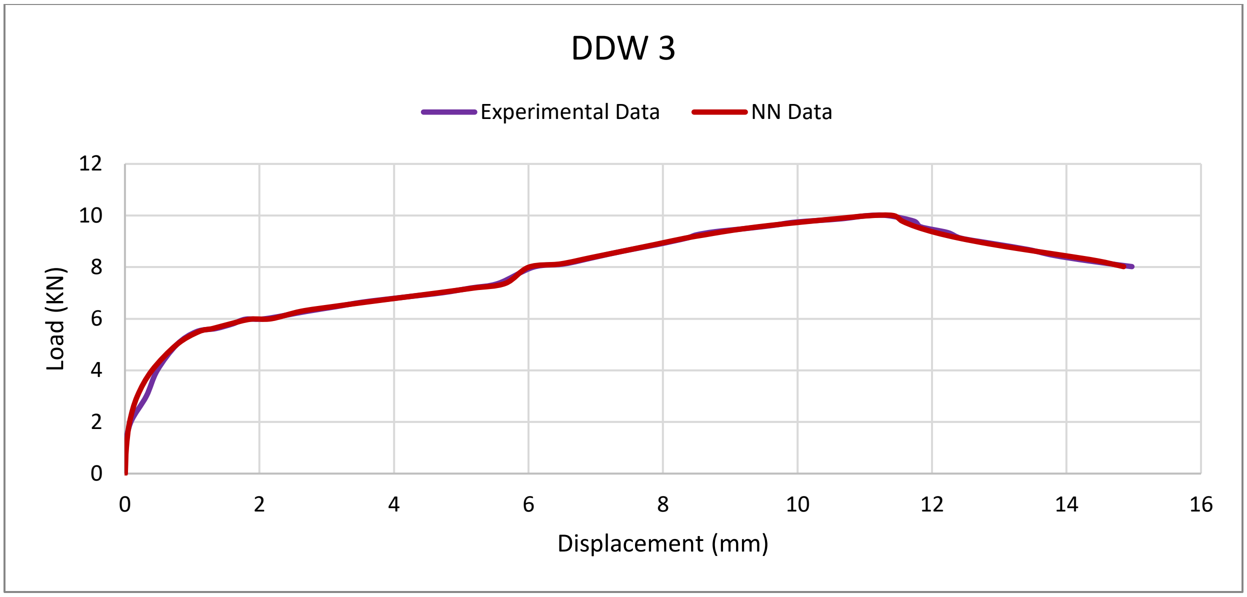

Figure3.9: Load-Displacement of DDW 3

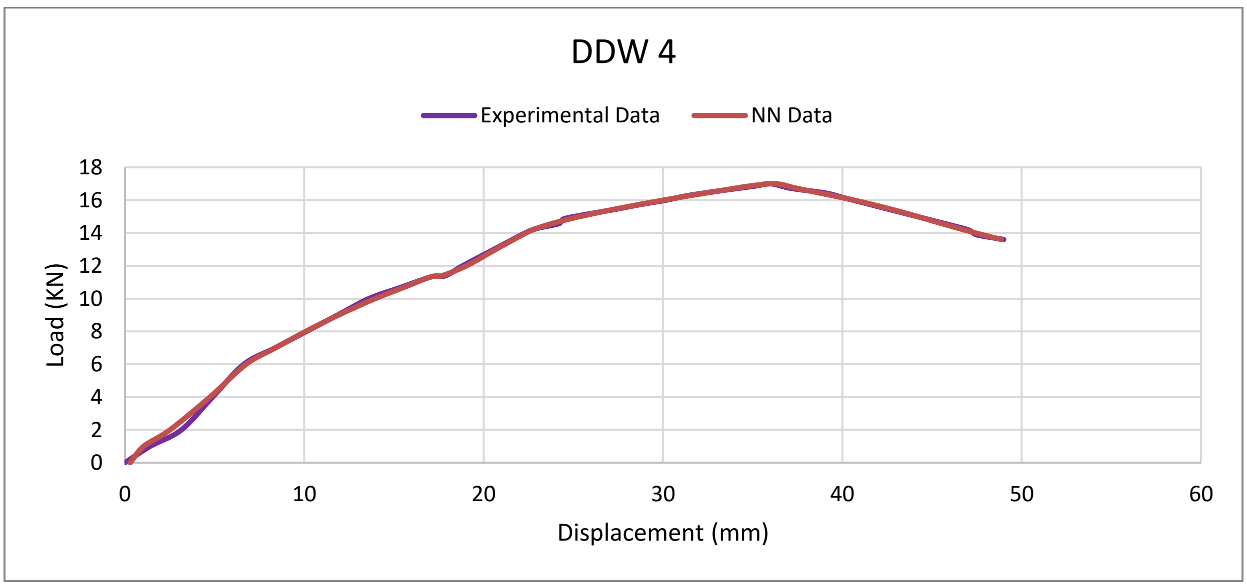

Figure3.10: Load-Displacement of DDW 4

Figure 3.11 is representing load vs displacement comparison between model data and experimental data of Pre-Retrofit Dhajji Dewari Wall 5 (DDW 5). Curve of model data stops before the curve of experimental data which can be clearly seen in the figure.

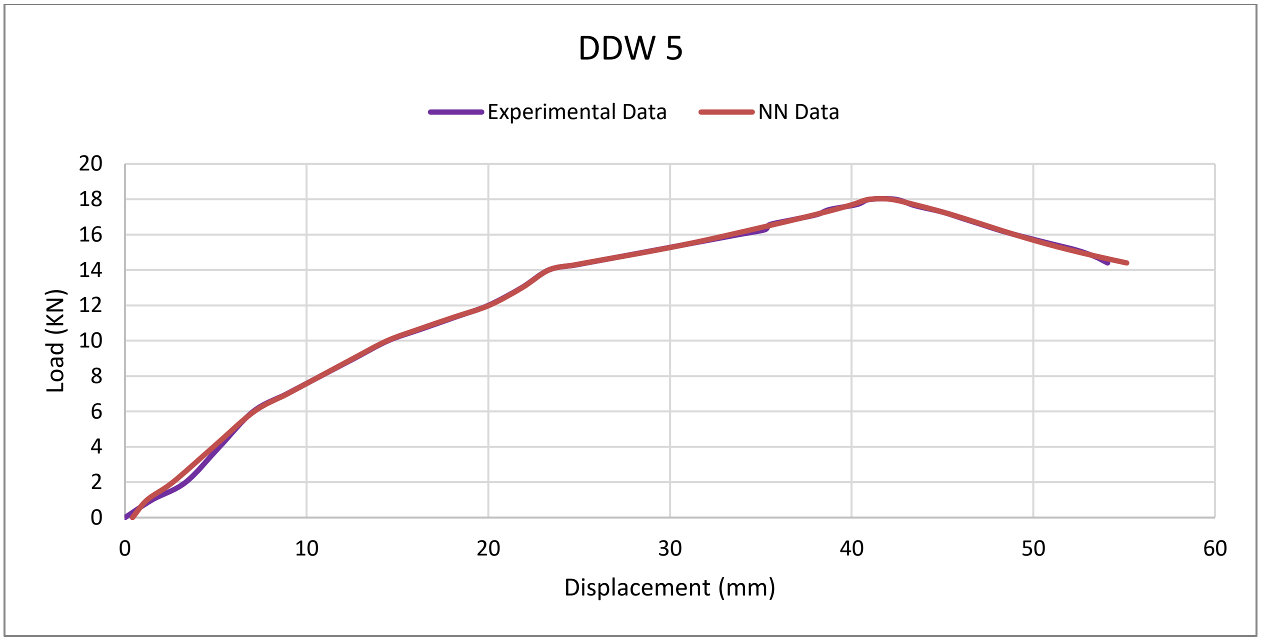

Figure3.11: Load-Displacement of DDW 5

Figure 3.12, Figure 3.13, Figure 3.14, and Figure 3.15 are representing load vs displacement comparison between model data and experimental data of Retrofitted Dhajji Dewari Wall 1 (RDDW 1), Retrofitted Dhajji Dewari Wall 2 (RDDW 2), Retrofitted Dhajji Dewari Wall 3 (RDDW 3) and Retrofitted Dhajji Dewari Wall 4 (RDDW 4) respectively. In all these four figures all the model data is matching to the experimental data.

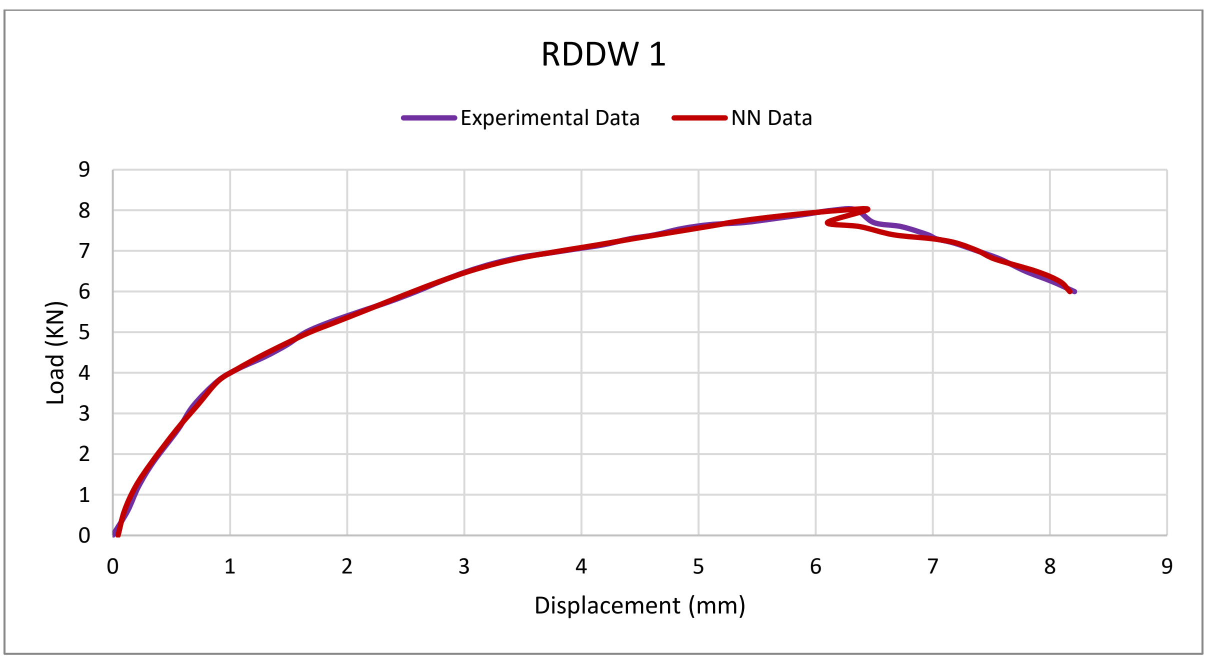

Figure3.12: Load-Displacement of RDDW 1

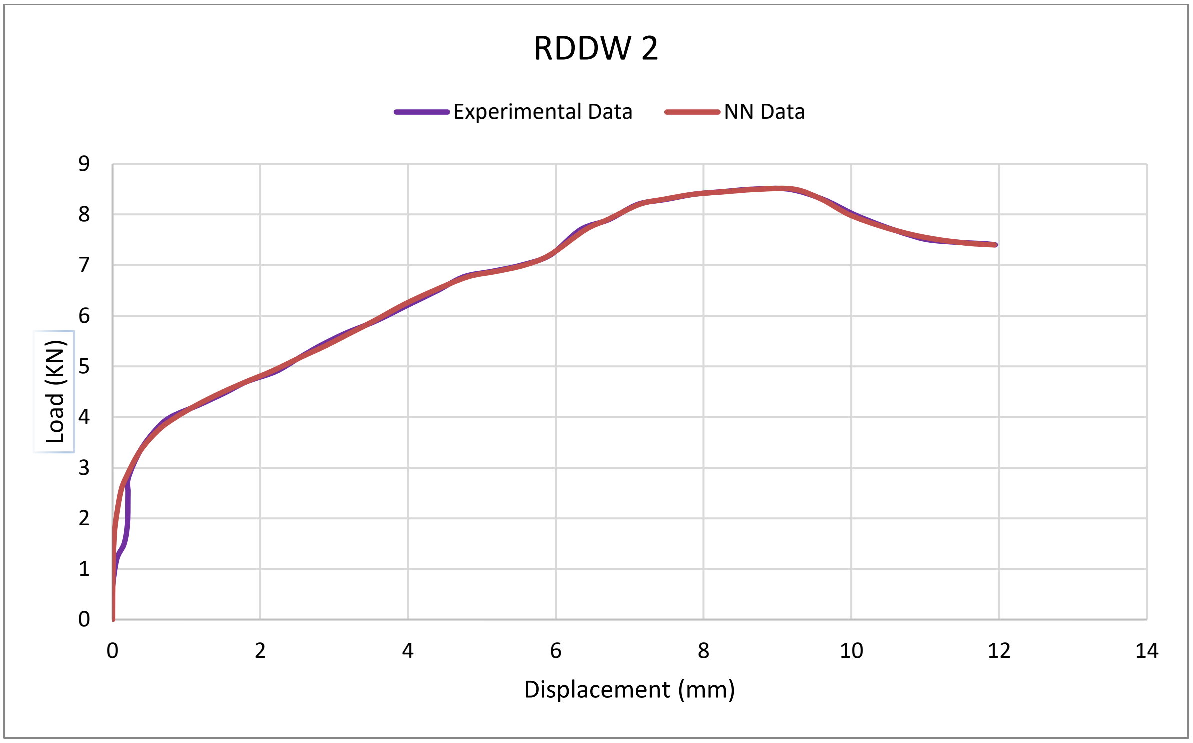

Figure3.13: Load-Displacement of RDDW 2

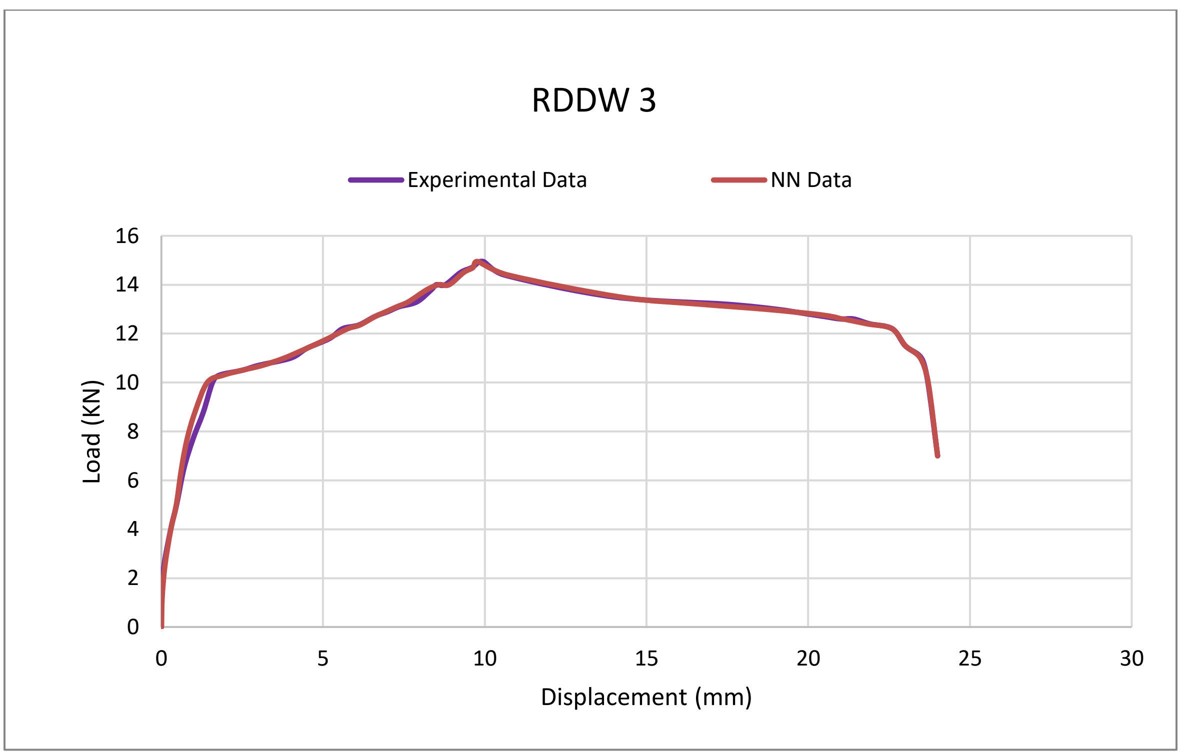

Figure3.14: Load-Displacement of RDDW 3

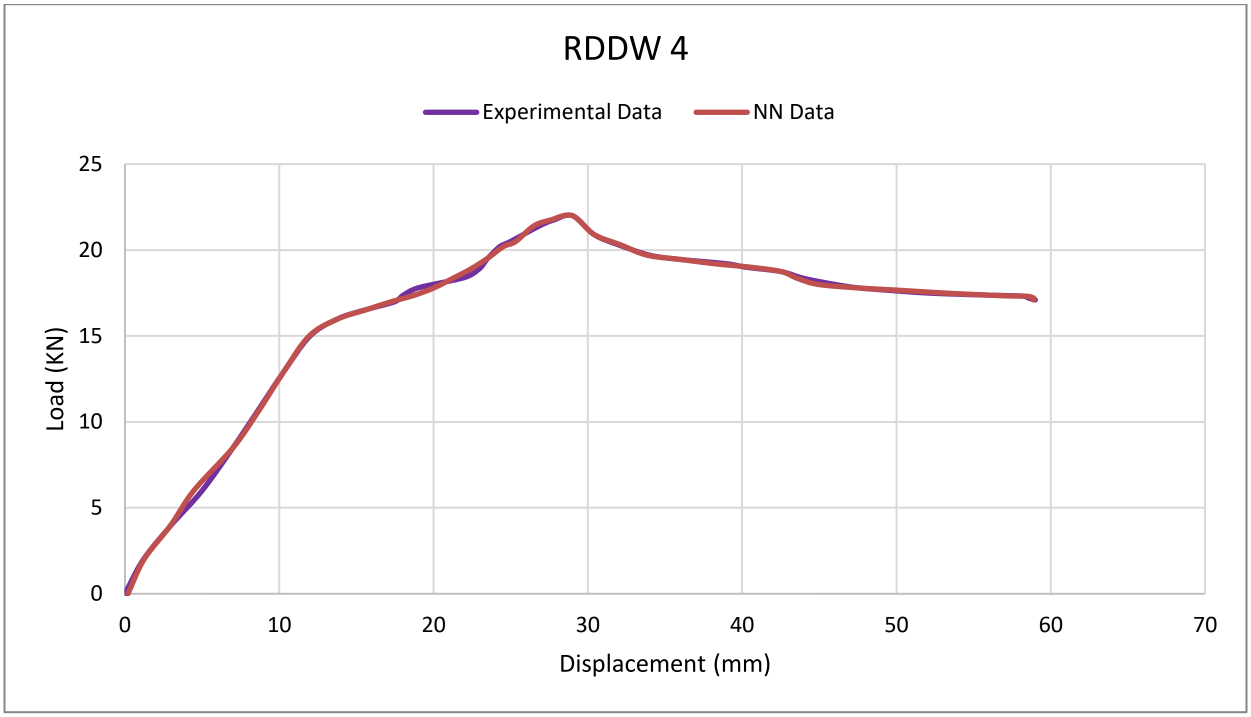

Figure3.15: Load-Displacement of RDDW 4

Figure 3.16 is representing load vs displacement comparison between model data and experimental data of Post-Retrofit Dhajji Dewari Wall 5 (RDDW 5). Curve of model data stops before the curve of experimental data which is clearly visible in the figure.

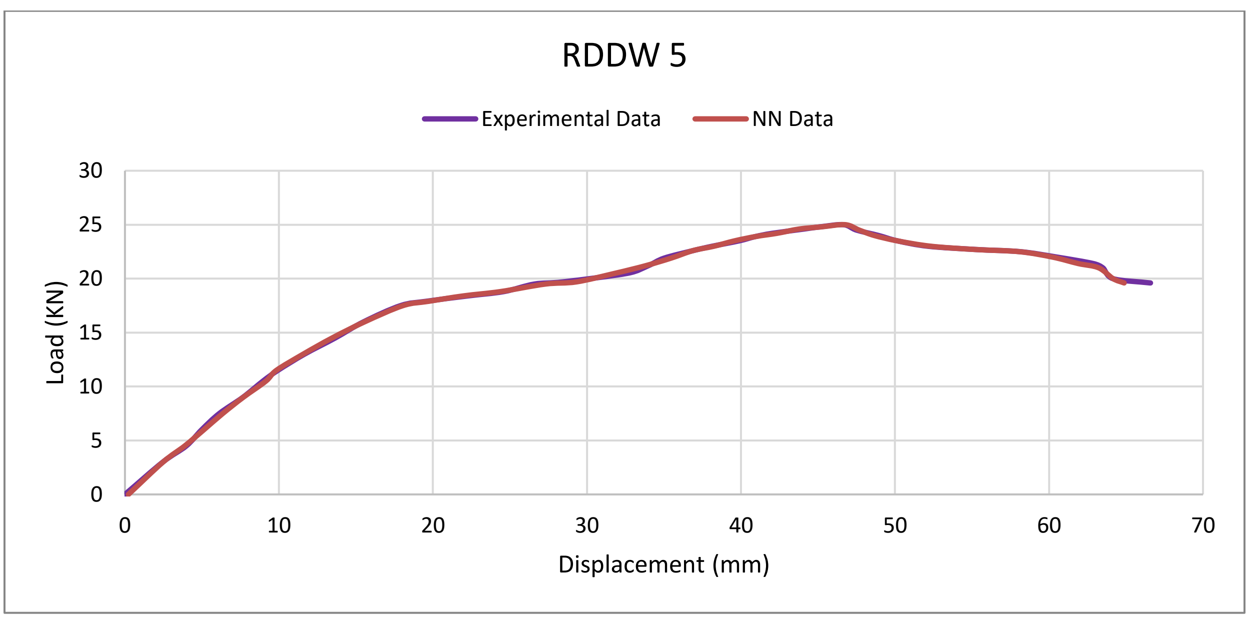

Figure3.16: Load-Displacement of RDDW 5

Percent Increase in Displacement (Post Retrofit walls)

Figure 3.17 is showing percent increase in displacement between model data and experimental data. Percent increase for first four panels is same while for panel 5 it is not same reason behind is that curve of model data of panel 5 in load displacement diagram was not ending at the same point. So that difference can be seen here.

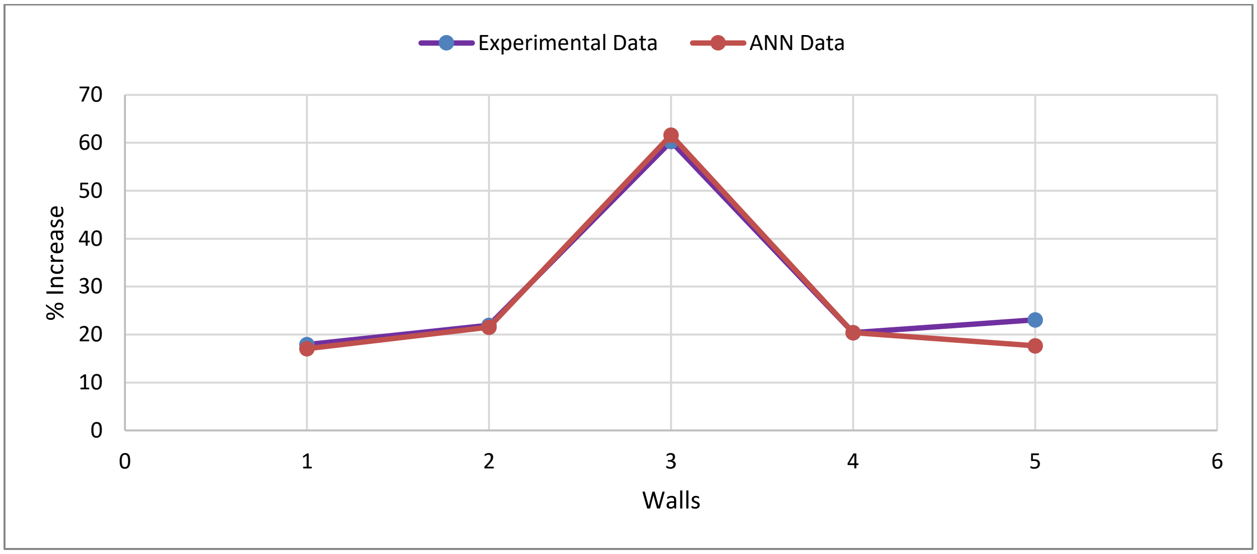

Figure3.17: Percent Increase in Displacement

Performance of NN Model

R values are closer to 1 for all cases validation, training, and test, which shows that performance of model is good given in Figure 3.18.

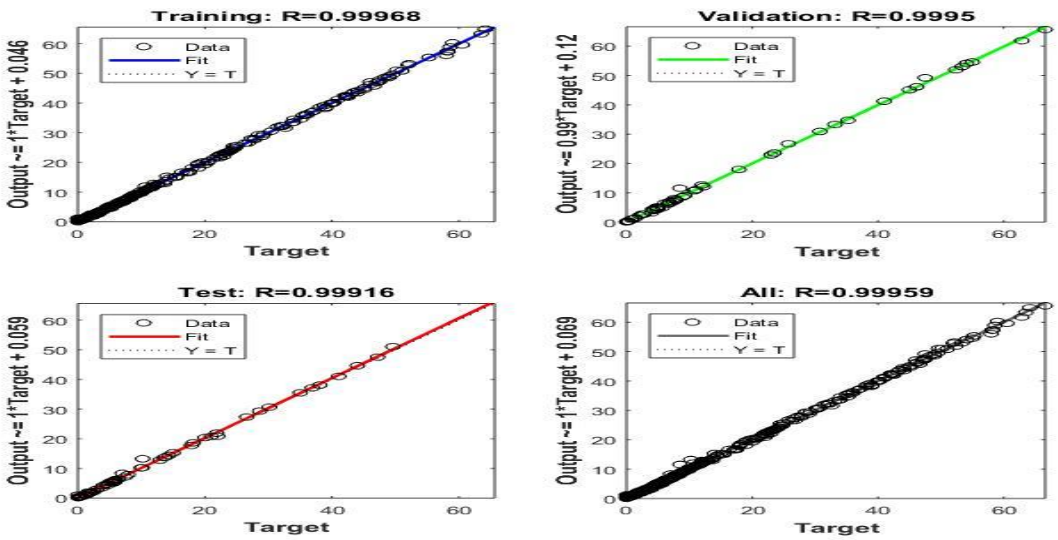

Figure3.18: Regression data

3.2.2 Energy Dissipation (Pre & Post Retrofit Walls)

Energy Dissipation for Pre-Retrofit panels can be seen in Figure 3.19. It shows that Energy Dissipation for first four panels is matching to the experimental data but for panel 5 it is not matching reason is perhaps that curve for model data in load-displacement diagram does not end at the same point where curve of experimental data ends. Because of this reason energy dissipation by experimental data is less.

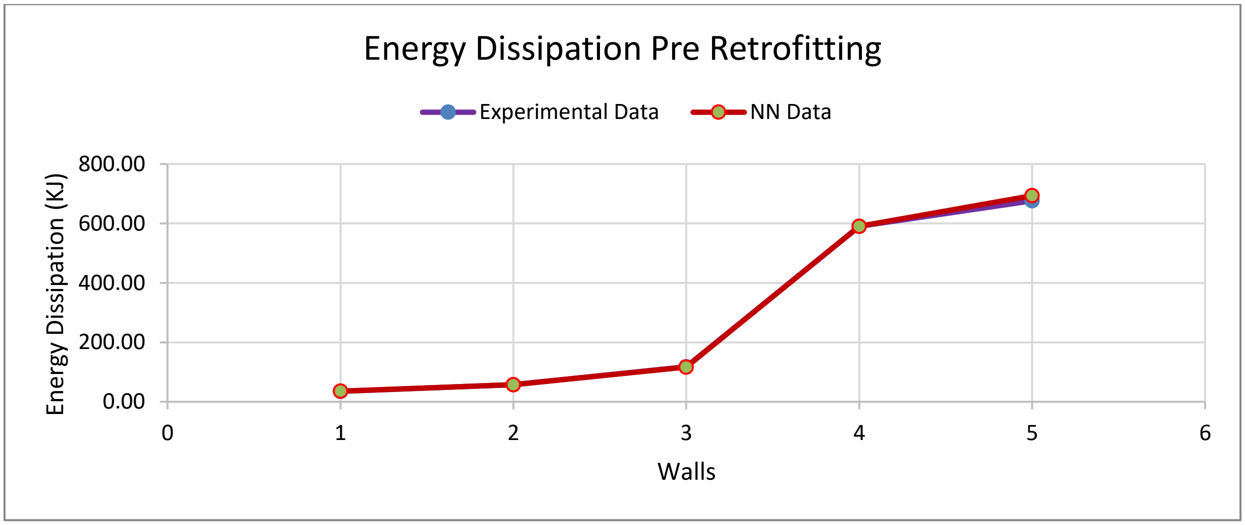

Figure3.19: Energy Dissipation of Pre-Retrofit Panels

Energy Dissipation for Post-Retrofit panels can be seen in Figure 3.20. It shows that Energy Dissipation for first four panels is matching to the experimental data but for panel 5 it is not matching reason is perhaps that curve for model data in load-displacement diagram stops before the curve of experimental data. Because of this reason energy dissipation by model data is less.

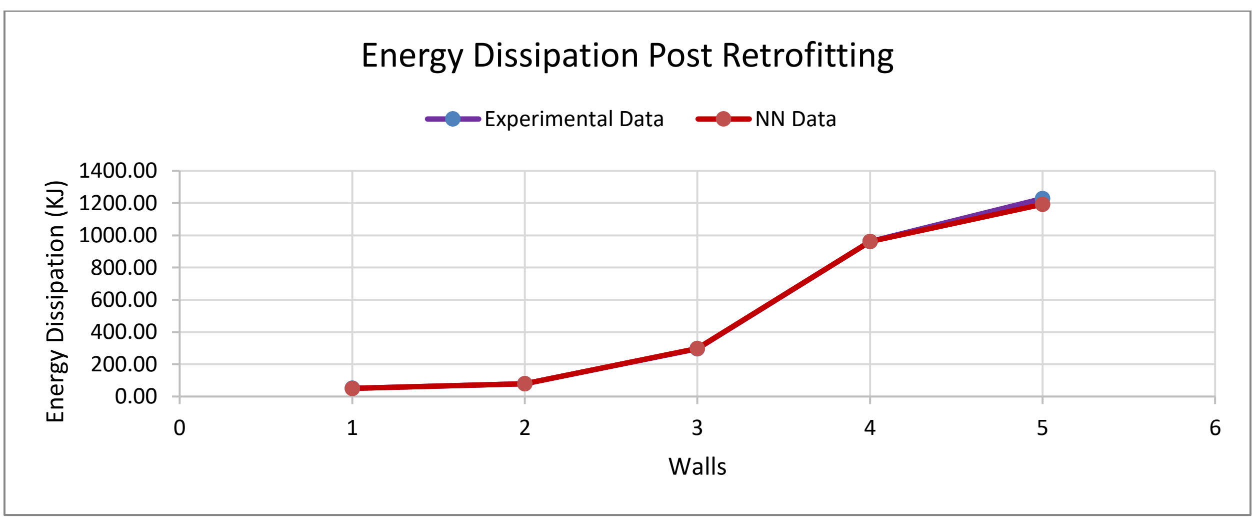

Figure3.20: Energy Dissipation of Post-Retrofit Panels

3.2.3 Ductility (Pre & Post Retrofit walls)

Ductility for Pre-Retrofit and Post-Retrofit panels can be seen in Figure 3.21 and Figure 3.22 respectively. Only small deviation in panel 2 of Pre-Retrofit and panel 3 of Post-Retrofit can be seen.

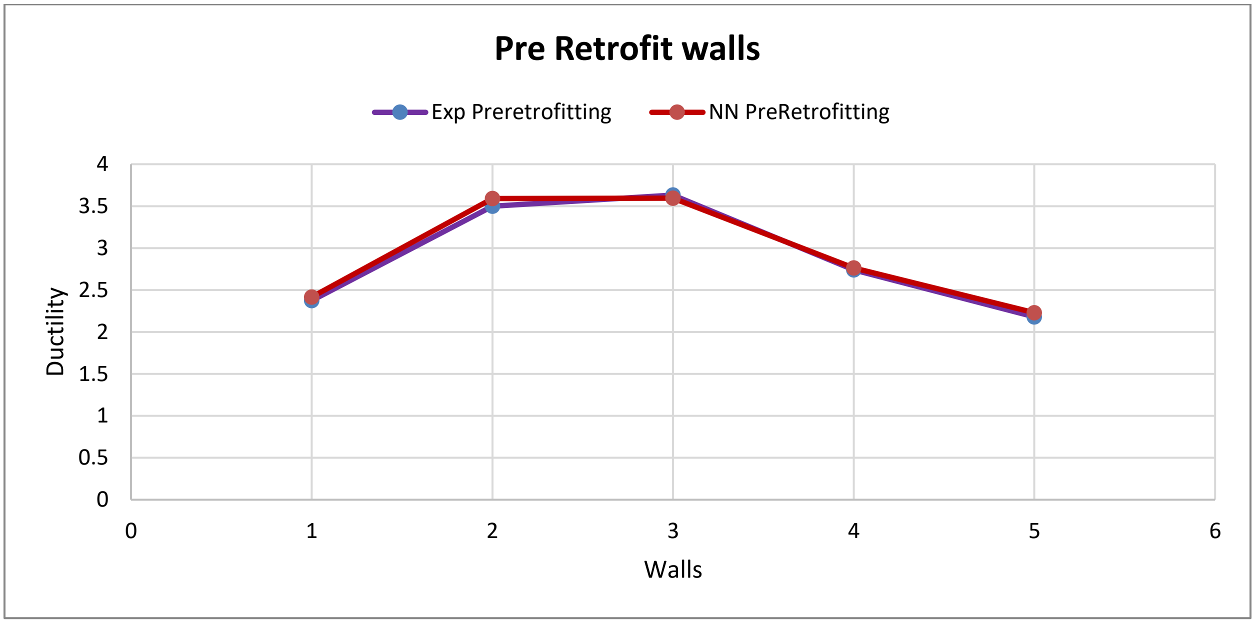

Figure3.21: Ductility of Pre-Retrofit Panels

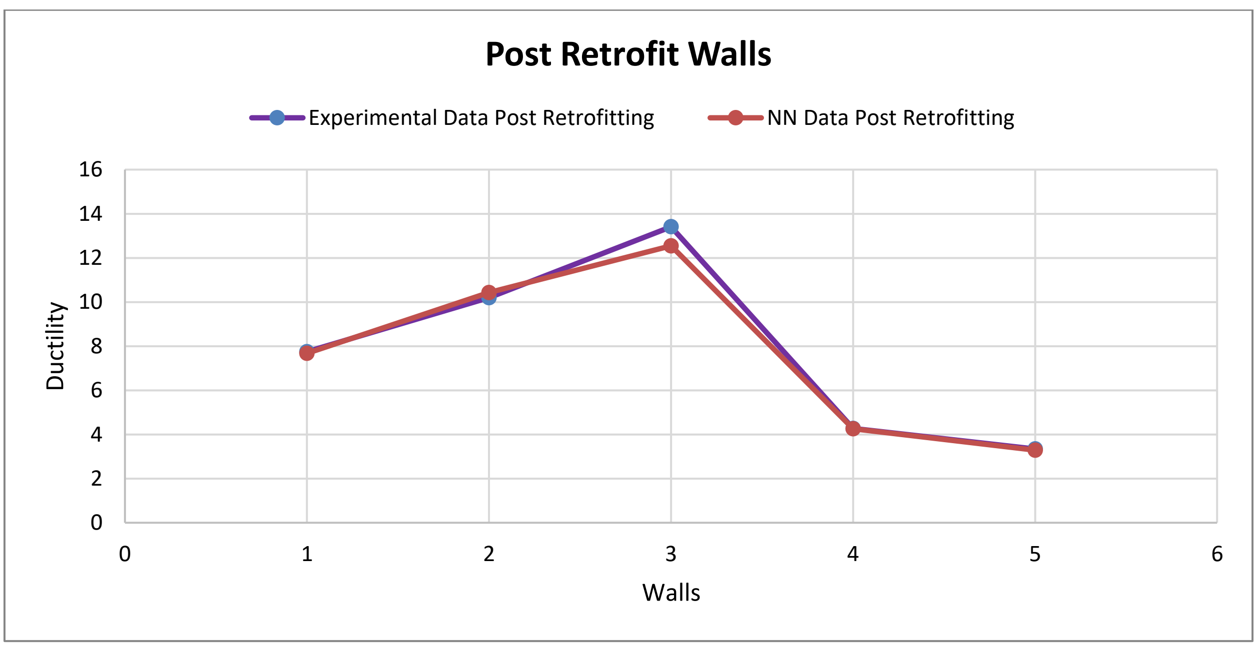

Figure3.22: Ductility of Post-Retrofit Panels

3.2.4 Response Factor Pre & Post Retrofit Walls

Response Factor for Pre-Retrofit and Post-Retrofit panels can be seen in Figure 3.23 and Figure 3.24 respectively. There is no significant variation in model data and experimental data for both Pre-Retrofit as well as Post-Retrofit panels.

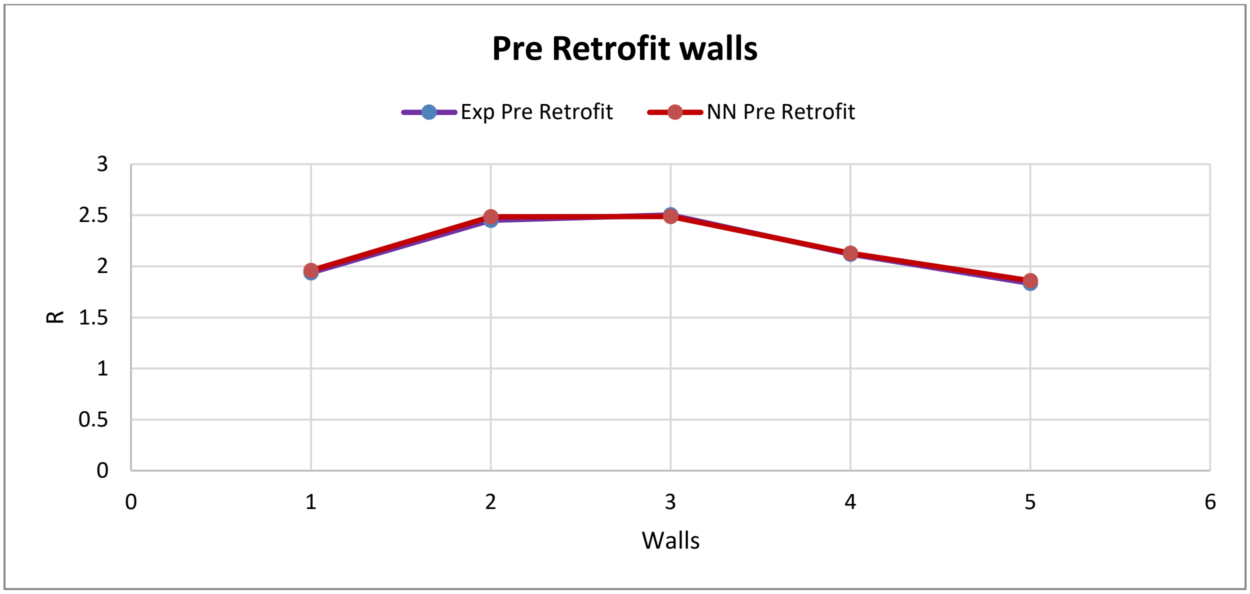

Figure3.23: Response Factor of Pre-Retrofit Panels

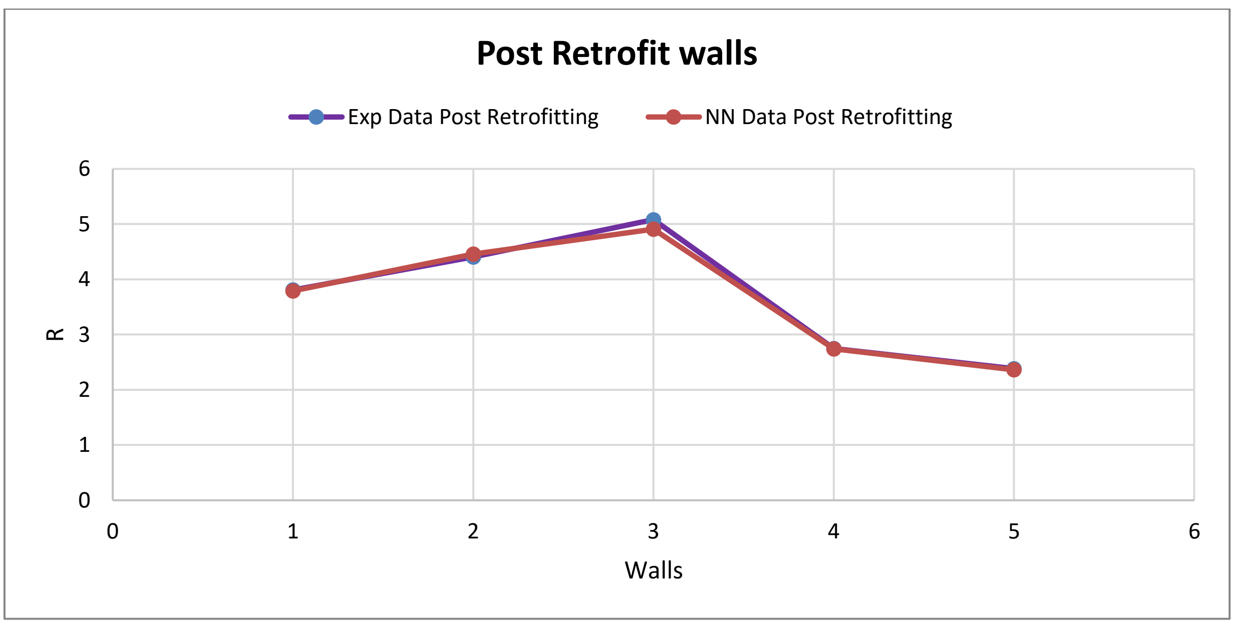

Figure3.24: Response Factor of Post-Retrofit Panels

CHAPTER 4

HIGH STRENGTH CONCRETE BEAMS

4.1 Methodology

In this research work five high strength steel fiber reinforcement concrete beams with two different confinements (CFRP & Steel sheet) were made. Out of these five beams 1st beam was taken as reference beam having conventional flexural reinforcement. 2nd beam was made with flexural reinforcement as well as steel fibers reinforcement. 3rd beam was made with steel fibers only and was strengthened with steel sheet. 4th and 5th beams were made by using flexural reinforcement and steel fiber reinforcement and were strengthened with CFRP and steel sheet. Monotonic load was applied at the beams to compare their flexural behavior.

4.1.1 Experimental Program

Description of Specimens

Testing program consists of testing five specimens as shown in Table 4.1. Nomenclature adopted as; Ref is symbol for conventional HSC beam which is reference beam for other beams. First two alphabets SF stand for steel fiber. In second set alphabet R stands for Flexural Steel Reinforcement and in third set the alphabet C shows CFRP confinement and alphabet S is for Steel sheet confinement.

Table 4.1: Nomenclature of Beams

Specimen	Dimensions (mm)	Flexural Reinforcement	Steel Fibers (1.5 %)	Steel Sheet Confinement	CFRP Confinement
Ref	152.5 × 152.5	Yes	--	--	--
SF-R	152.5 × 152.5	Yes	Yes	--	--
SF-R-C	152.5 × 152.5	Yes	Yes	--	Yes
SF-0-S	152.5 × 152.5	--	Yes	Yes	--
SF-R-S	152.5 × 152.5	Yes	Yes	Yes	--

4.1.2 Test Setup

Test setup was same for all five specimens. Four-point loading test was used for all the samples, in compliance to ASTM D790-17 [48]. Beams are tested as simply supported with one end pin and the other end as roller support, whereas Load P is applied at two loading points (Figure 4.1) Instruments used were three LVDTs. Three LVDTs were fixed at sections 1,2 and 3 as shown in Figure 4.1. To record data, NI 9363 Module was used. The LVDTs can measure the displacement up to 100 mm.

The hydraulic system used in the reaction floor has capacity of 400 KN. The jack can apply both cyclic and monotonic loadings. The jack was supported by a reaction floor made from two C sections welded together back-to-back. To record data, NI digital data acquisition system was used. The system has three slots for LVDTs, that is why 3 LVDTs can be connected at same time. The jack is operated by Laptop, having Lab view software. The software controls the movement of fluids with pressure and thereby movement of jack is controlled. To adjust the jack to save time the test setup is changed to manual in Lab view software which lead to the quick movement of jack. To adjust the optimum height of the jack supports are lifted upward and a cylinder is provided below the jack. The cylinder has diameter of 100 mm and length of 380 mm. Supports were lifted upward by using steel plates and fixing two cylinders in the plates. To provide the simply supported behavior, one support is arranged as roller and the other is arranged as pin (Figure 4.1). Hinge support is provided by placing a concrete filled steel block over the steel plates. Pin support is formed by placing a cylinder horizontally over the frame.

The specimens were very heavy, to lift the specimens, a manually controlled crane was used. First a steel reinforced plastic rope was tied over beam, then the rope was hanged in the hook of a crane. The crane was also supported by reaction floor.

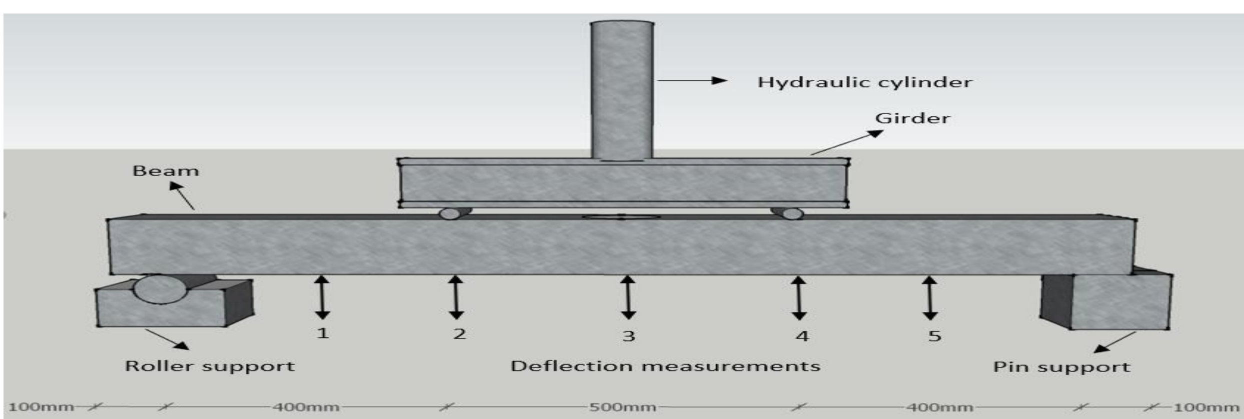

Figure 4.1: Test Setup

Loading Protocols for Monotonic Tests

Monotonic tests are performed to study the mechanical behavior of concrete beams. In this test load is applied at a constant increase to the specimen to obtain its properties such as yield strength and ultimate strength etc. Five samples were casted for monotonic loading. Test was conducted at a slow rate of loading, the rate of loading used was 14.71 KN/min.

The major difference between the three and four-point bending tests is the distribution of the bending moments. Four-point loading method is used for high modulus composites. The four-point loading give uniform stress distribution between the loading points, as shown in the Figure 4.2. The three-point bending test gives stress distributions below the central point where the load is applied. In our case, a steel girder was used to distribute the load at two points from girder to the supports. To record the deflection at points of loading, at mid span of beam, the midspan between the point of load applied and support and under the point of load applied LVDTs were used.

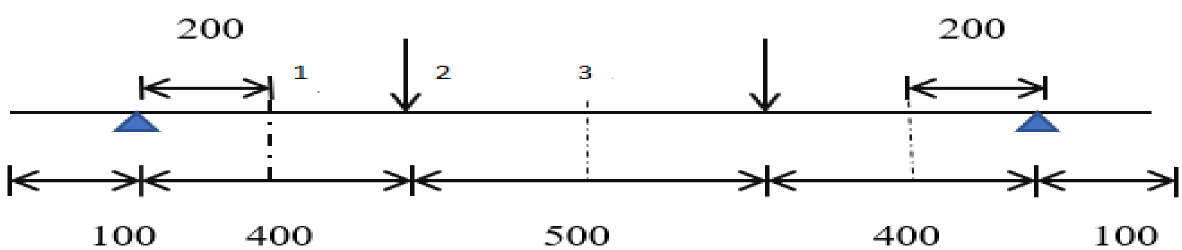

Figure 4.2: Test Setup

4.1.3 Material Properties

Steel fibers and CFRP strips were imported from Belgium and China, all other materials used in this research were obtained locally. Special attention was paid to select material identical in properties that are being used in the local construction industry. Details regarding each material used during the research are provided as under.

Concrete Materials and Steel Fibers

Ordinary Portland cement manufacturing by “Best way Cement Limited” of grade 53 type 1 by Pakistan standard PS-232-2008 and conformity to ASTM C150-04, EN 116 was used in casting samples. Viscocrete 3110 (1st generation) superplasticizer was used for preparation of high strength concrete samples. Sand used in testing samples of HSC was obtained from the sand deposits of Lawrencepur. The Sand test for Fineness Modules (FM) was carried out and found to be 2.01, as per ASTM C-136 sieve analysis was carried out. The gradation and aggregate size were chosen as per mix design of High Strength Concrete. The maximum size taken was 20mm. Two sieve sizes were used, 12mm and 20mm. The ratio was set to 1:0.88, respectively. Graded aggregate was used to achieve higher packing density. Angular and rough particles of coarse aggregates were used so to increase the mechanical properties of concrete. Gradation of coarse aggregate was done as per ASTM C136. Master Steel Fiber S 65 was used in this research, obtained from BASF Company. It contained hooked end fiber with length 35mm and aspect ratio 64. Properties of Steel Fibers are given below in Table 4.2.

Table 4.2: Steel Fiber Properties

Length	35mm
Diameter	0.55mm
Aspect Ratio	64
Tensile Strength	1345 MPa

Mix Design

Mix design of concrete to achieve the target strength was found by making some trials. The cylinders of size 150*300 mm were tested, in compliance with ASTM C31, for each trial. Finally, mix design for the compressive strength of 65 MPa was obtained. Mix Design for 65 MPa is shown in the Table 4.3.

Table 4.3: Concrete Mix Design for Compressive strength of 65 MPa

Coarse Aggregate <10 mm	Margalla
Coarse Aggregate (10-18) mm	Margalla
Fine Aggregate	Lawrancepur
W/B ratio used	0.384
Superplasticizer Viscocrete 3110(1st generation)	1.82
Silica Fume	11 %
Steel Fibers	1.5 %
Steel Fibers Bought from	BASF Rawalpindi
Chemicals Bought from	Sika Chemicals Rawalpindi

CFRP Strips, Primer and Epoxy

Carbon fiber reinforced polymer strip was obtained from BASF Company. The brand name for the CFRP strip is "Master Brace Laminate®". The strip had to be imported from China as it is not being produced locally. This strip can be used in a variety of sizes. The strip needs to be pasted on concrete surface using suitable adhesive. The CFRP was pasted on concrete samples using epoxy obtained from BASF Company. The solution used was imported from Australia and had a tested excellent adhesion to concrete surfaces. The epoxy chosen were according to the working temperature and environment to ensure sufficient pot life while preparation of samples. Adhesion strength was another important factor in the selection of chemicals. For this research "Master Brace P3500 ® and Master Brace P4500® was used in the lamination of concrete samples with CFRP strip. Master Brace P3500 is a two component Solvent-Less epoxy system which when mixed yields a penetrating medium viscosity primer. This substance was used as primer prior to use of Master Brace P4500. It ensures smoothness of the surface and is applied by brush to concrete samples. The table below shows the properties of P3500. Master Brace P4500® SAT4500 is epoxy based, solvent free, light strength adhesive development for FRP bonds on concrete and other surfaces. This has two component "A" and "B" which need to be mixed at a ratio **1:0.5** for obtaining the final epoxy. It is easy to apply and has low viscosity.

The properties of Master Brace Laminate®, Master Brace P3500 and Master Brace 4500 are shown in Table 4.4 below.

Table 4.4: Properties of CFRP strip, Primer and Epoxy

Master Brace Laminate®	
Width (mm)	230 gsm
Thickness (mm)	230 KN/mm^2
Section (mm2)	4900 N/mm^2
Weight (gm/mm)	200 g/m^2

Master Brace P3500	
Adhesion strength on carbon MPa (ASTM D4541:95e1)	2.87
Tensile strength, MPa (ASTM D638:00)	35
Flexural strength, MPa (ASTM D790:01)	55
Compressive strength, MPa (ASTM D695:96)	73
Mixing ratio: by Volume	1.67 to 1
Pot-Life (at 25^0C)	70min
Master Brace P4500	
Mixed Density	1.02 Kg/ liter
Viscosity	1500-2500 MPa
Compressive Strength TS EN 196 (7 days)	>60 N/mm^2
Flexural Strength TS EN 196 (7 days)	>50 N/mm^2
Bonding Strength to concrete (7 days)	>3.0 N/mm^2
Application Temperature	+5ºC - +30ºC
Pot Life	30 minutes
Fully Cured at 20oC	7 days

Steel Tubes & Flexural Steel

Material selected for fabrication of Steel tubes was made up of steel sheets having thickness of 2 mm. Material properties such as modulus of elasticity E, yield strength fy and ultimate strength fu of steel sheet was found using ASTME8/E8M-13a. Fabrication process consisted upon a steel sheet of 2mm thickness that was bended to a square shape. That bended steel sheet was welded longitudinally to make exact square cross sections of 152.5 mm. Tensile properties of steel bars were obtained using testing standard ACI 440.3R (2004). Flexural steel was purchased from Taiba steel, I-9 Industrial Estate, Islamabad. Flexural and shear steel reinforcing bars were tied with binding wire to maintain spacing for bonding and to make a cage. Mechanical properties of steel tubes & steel bars are shown in the Table 4.5.

Table 4.5: Properties of Steel Tube and steel Bars

Thickness of steel tubes(t)	2 mm
Outer size of steel tubes(do)	152.5 mm
Elastic modulus(E) of steel tubes	200 GPa
Yield strength of steel tubes (fy)	305 MPa
Ultimate strength of steel tube (fu)	368 MPa
Length of flexural steel bars	1450 mm
Length of Anchor steel bars	1450 mm
Outer Size of stirrup steel bars	100 mm
Diameter of flexural steel bars	20 mm
Diameter of Anchor steel bars	10 mm
Diameter of stirrups steel bars	10 mm
Elastic modulus of steel bars (E)	200 GPa
Yield strength of steel bars (fy)	416 MPa

Four beams were provided with balanced longitudinal steel of 567.74 mm2, according to the ACI requirements. Two 20 mm bars of 416 MPa were provided to each sample in tension zone and two 10 mm bars of 416 MPa were provided in compression zone of concrete. The tensile properties of steel bars were obtained using testing standards ACI 440.3R(2004).

4.1.4 Samples preparation

Four no of samples were prepared with addition of constant percentage of steel fibers (V_f). The percentage of steel fiber was added as 1.5% of volume of concrete. The mixing of concrete was done in drum mixer with mixing at the rate of 45 rpm. The steel fiber and super plasticizer was added in the mixture uniformly during the mixing for workability. Three samples were casted in 152.5 * 152.5 size molds made from Usman Khattar Taxila confirming to ASTM standards.

Figure 4.3: Preparation of Beam samples

Steel tubes were fabricated in two steps. First, a sheet of 2 mm thickness was bent to make square-shaped section. Then, the square-shaped sections were welded longitudinally to make a complete square section. During casting tubes were poured with concrete by standing them vertically. Compaction was done with the help of long steel rod. Binding of reinforcement in the reinforced beam was done through binding wire to maintain proper gap for bonding. Prepared beams are shown in the Figure 4.4. The Beams were prepared in Government college of Technology (GCT) Taxila Materials Testing laboratory at standard room temperature of 25^{o}C.

Figure 4.4: Fabrication & Pouring of Steel Tubes

The samples were cured for 28 days in curing tank at GCT Taxila. After curing all five beams were shifted to NUST Structural Engineering Laboratory.

For application of CFRP strip one of the samples having steel fiber and flexural reinforcement was Air dried. The sample was cleaned on the surface with brush and Master Brace ® 3500 was applied as a primer before the application of epoxy. MB 3500 consisted of past “A” and “B” which were mixed in ratio **1.67:1.0** before application. The primer was allowed to get absorbed in concrete surface for 6 hours before the application of epoxy Master Brace ® 4500. MB 4500 consisted of part “A” and “B” which were mixed in ratio **1:0.5** before application. It consisted of blue color thick liquid and was applied to concrete sample after that CFRP strip was pasted.

After CFRP strip was pasted, MB®4500 was applied the strip as well as recommended by the supplier/ importer. The sample was air dried for 05 days. The Figure 4.5 below shows the final confined sample.

Figure 4.5: Application of CFRP strip

4.1.5 Parameters studied in Experiments

Load-Deflection relationship

When we apply load on beam, it will deflect, increasing load will increase deformation. To study the response, three points are taken into consideration i.e., yield point, Peak point, and ultimate load. The yield displacement of an elastoplastic system is taken at 75 percent of the ultimate load. The method used to calculate yield point is shown in Figure 4.6.

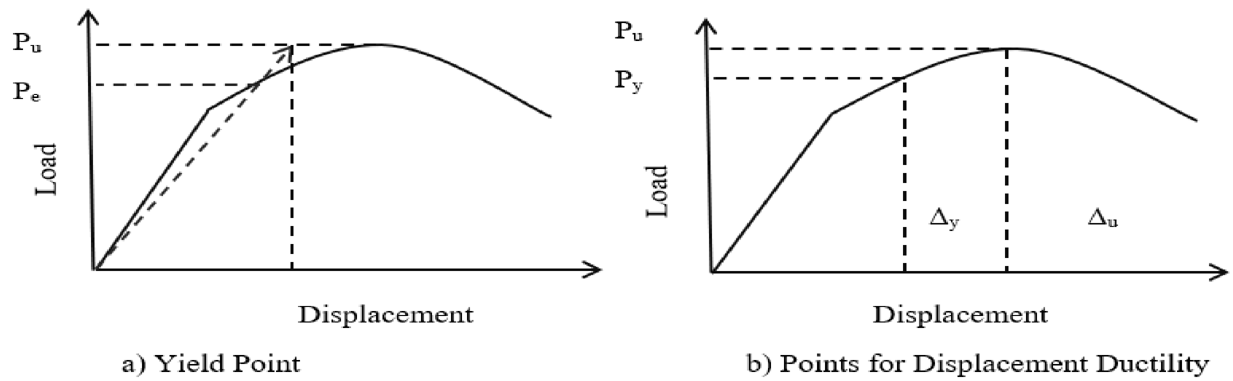

Figure 4.6: Determination of Yield point & Ductility

Energy Dissipation

The energy dissipated by the sample beams is an important parameter of a structure, it is also calculated from the load-displacement or stress-strain curves.

Higher the value of energy dissipated represent ductile failure, on the other hand higher values of energy dissipated by the specimens can also be due to the addition of steel fibers and confinements used and their effectiveness.

The failure mode of the sample beams also affects the energy dissipated.

Energy dissipation is calculated as the area enclosed by the load displacement curve up to ultimate point. Whole area is divided into different segment and calculating sum of all the segment gives the energy dissipated by system. Area under each segment is calculated using trapezoidal formula as given in equation 4.1. The detail is shown in figure 4.7.

$$\text{Area Under Curve} = (\frac{a+b}{2}) * (d - c)\Big) \qquad (4.1)$$

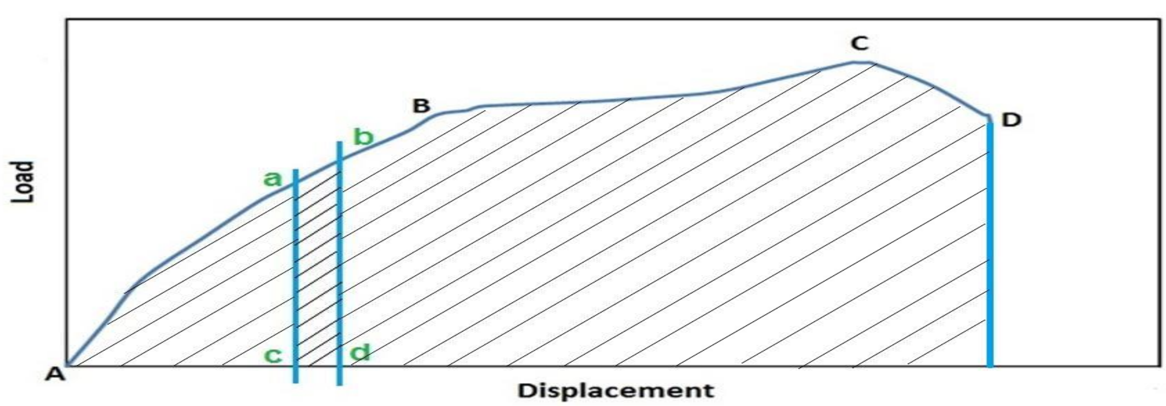

Figure4.7: General Load vs Displacement Diagram

Ductility and Response Factor

The ability of a structure to deflect beyond yield point, without significant loss in strength is called ductility. The method used for calculation of ductility is from displacements, therefore it is called displacement ductility. Displacement ductility is ultimate deformation divided by yield deformation, given by equation 4.2.

$$\mu = \Delta u / \Delta y \qquad (4.2)$$

Response factor is an important factor of seismic design. Here we will determine the response factor using Pauly and Priestley (1992) which is given by equation 4.3.

$$Rf = \sqrt{(2\mu - 1)} \qquad (4.3)$$

Where μ is the displacement ductility of the specimen.

Stiffness degradation

To define the degradation in stiffness three points considered are yield, peak and ultimate points. The force per unit deformation at that points is called stiffness at that points.

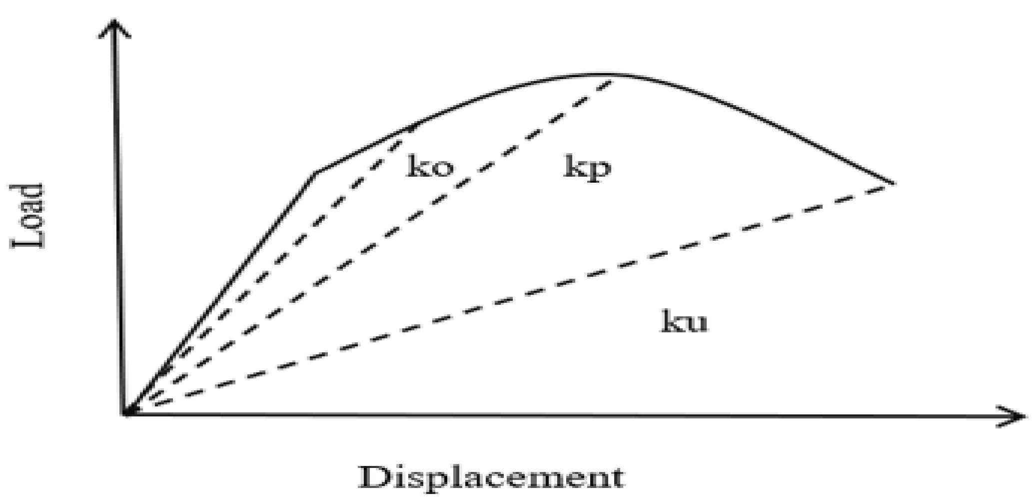

Figure 4.8: Determination of stiffness at different points

Stiffness degradation ratio Ck is the ratio of secant modulus K at specified displacement to the secant modulus Ko at yield which can be seen in equation 4.4.

$$Ck = K / Ko \qquad (4.4)$$

Here we will use peak of load displacement curve stage.

4.2 Results and Discussion

4.2.1 Load-Deflection Relationship

Load deflection relationship gives us mechanical behavior of specimens under loading. In figure 4.9 it is visible that all beam specimens having steel fibers reinforcement are more ductile and are showing good post peak behavior as compared to Ref beam that is without steel fiber reinforcement.

From Load Deflection curves in Figure 4.9 it can be seen that SF-R-C specimen having CFRP confinement managed to get peak up to 166.63 KN but there is sudden fall in curve due to flexure-shear cracks which lead to debonding of CFRP strip. In SF-R-C specimen steel fiber could not play very good role because of dominating role of CFRP strip. Only role of steel fiber in this specimen is that it made the beam specimen ductile up to some extent as compared to Ref beam.

Beam specimens with steel sheet confinement got almost double flexural load capacity as compared to Ref beam. When we compare steel sheet confined beams there is a sudden fall at much early stage in curve of SF-0-S that is due to full rupture of concrete in the steel tube as well as rupture of steel tube in tension zone. SF-R-S attains maximum ultimate displacement that is 94.36 mm in this load-deflection curve.

SF-R specimen justifies the literature that addition of steel fibers increases the shear strength as it helped this specimen in this load-deflection diagram to achieve its full flexural strength by providing it more shear strength and making its post peak behavior better as compared to the Ref beam specimen that failed due to flexure-shear cracks due to bending.

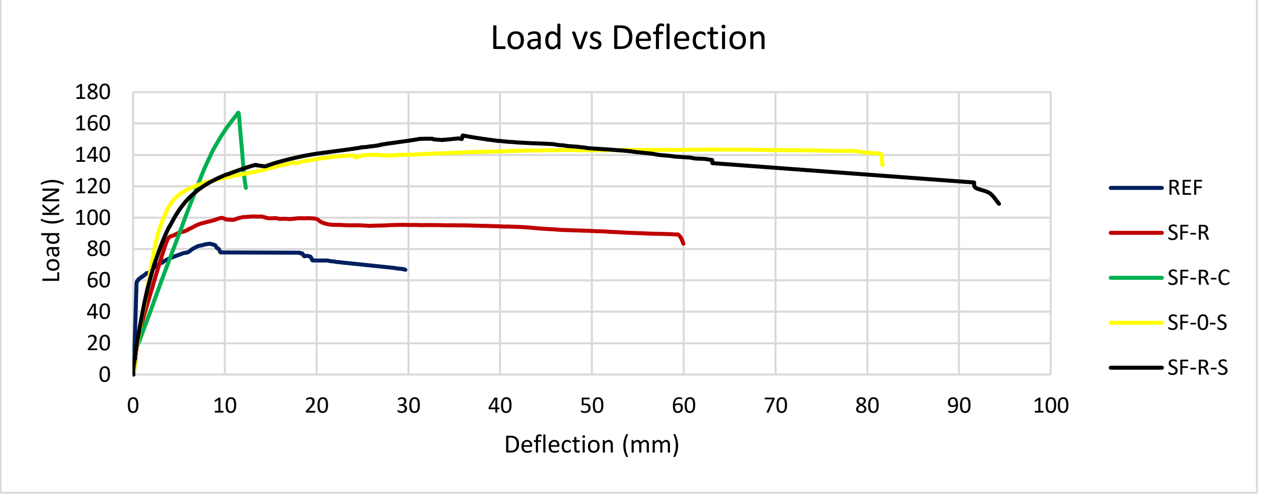

Figure 4.9: Load vs Deflection

Table 4.6: Results after Monotonic Loading

Specimen	Deflection at yield point Δy (mm)	Deflection at peak point Δp (mm)	Ductility μ	Stiffness values			Ck	Rf
				Ko	Kp	Ku		
Ref	2.67	7.966	2.98	12.91	5.89	1.88	45.68	2.23
SF-R	3.07	13.22	4.31	24.24	7.62	1.39	31.44	2.76
SF-R-C	7.30	11.49	1.57	17.06	14.50	9.68	85.01	1.47
SF-S	3.94	63.82	16.19	27.26	2.25	1.64	23.81	5.60
SF-R-S	2.03	35.92	17.69	17.81	4.24	1.15	8.24	5.86

4.2.2 Energy Dissipation

Energy dissipated by monotonic loading beams is the area under load deflection curve. From Figure 4.10, it can be seen that energy absorbed by SF-R-S beam is greater than all other beams. The energy dissipation is greater due to the availability of steel fibers, flexural steel and steel sheet confinement. On the other side, the overall energy of steel sheet confined beams is greater than all other beams due to presence of steel sheet confinement. In steel sheet confined samples energy is dissipated in cracking concrete, breaking of bond between two steel and concrete, damaging of steel sheet and steel bars. In these members SF-R-S has greatest energy dissipation capacity due to the presence of more steel fibers to absorb energy (Figure 4.10). Energy dissipation value of steel sheet confined beams is almost double the Ref beam. SF-R have additional reinforcement of steel fibers, thereby it has absorbed more energy as compared to Ref beam as steel fibers improved its post peak behavior. SF-R-C absorbed less energy even steel fibers were added in this sample, but they could not play their role as CFRP confinement was dominating and after debonding of CFRP strip there was a sudden reduction in strength of the beam.

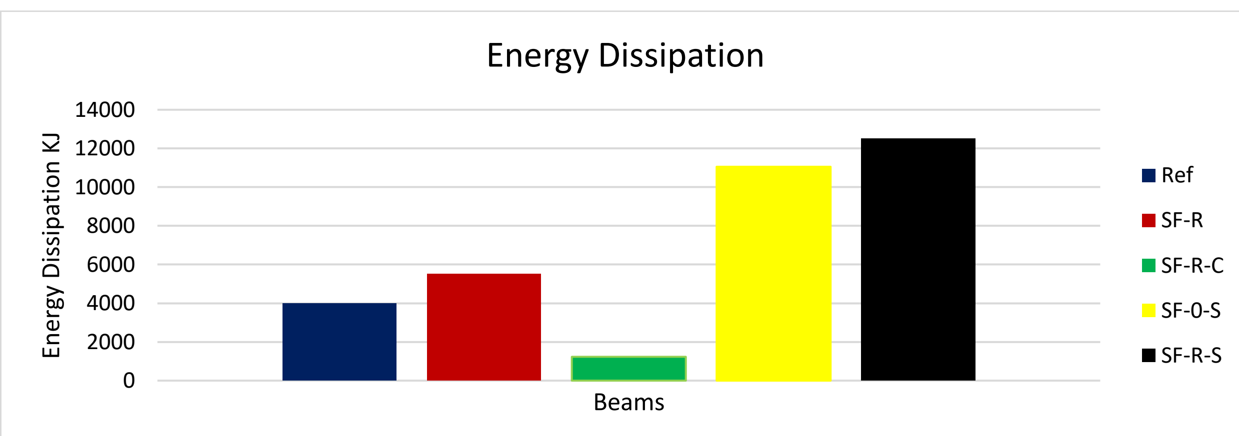

Figure 4.10: Energy Dissipation

4.2.3 Ductility

Displacement ductility μ studied is the ratio of displacement at peak Δp to the displacement at yield Δy. In Figure 4.11, in SF-R-S beam, the deflection difference at the peak and yield is greatest which resulted in the higher value of ductility. Beams having steel sheet confinement gave an almost four times value of displacement ductility. SF-R-S shows highest ductility. Here it can be seen that steel sheet confinement makes beams more ductile. When we compare SF-R-S with SF-0-S, first beam is showing more ductile behavior due to flexural reinforcement.

SF-R-C has lowest ductility value because of the CFRP confinement which makes it stronger in flexural but reduces its ductility. In Figure 4.11 it can be seen that SF-R is more ductile than Ref beam due to addition of steel fibers.

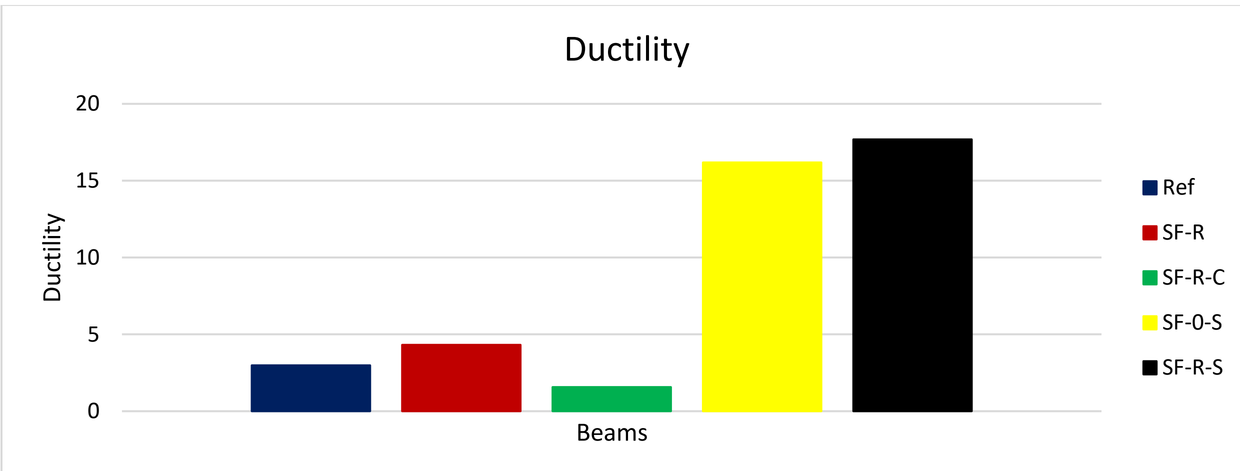

Figure 4.11: Ductility

4.2.4 Response Factor Rf

Response factor is an important factor of seismic design. Here we will determine the response factor Rf for Displacement ductility μ using Paulay and Priestley (1992) which is given by equation 4.3.

Greater value of ductility leads to greater response factor. Here in Figure 4.12, it can be seen that SF-R-C has lowest Response factor because this beam shows very low ductile behavior due to it type of confinement (CFRP) which leads to higher strength but reduced ductility. Reference beam stands second last because all other beams have steel fibers which makes them more ductile even after reaching peak. Steel Sheet confined beams have almost double response factor values when compared with Ref beam and SF-R-S has more response factor value as compared to SF-0-S due to additional longitudinal reinforcement.

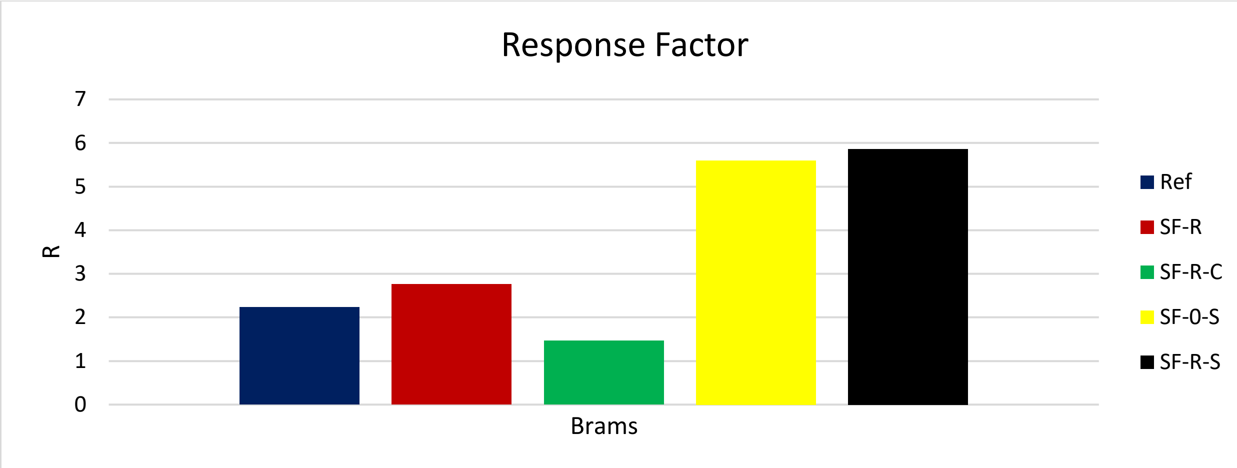

Figure 4.12: Response Factor

4.2.5 Stiffness degradation

Stiffness degradation ratio Ck is the ratio of secant modulus K at specified displacement to the secant modulus Ko at yield. Here we will use peak displacement of load-displacement curve as our specified displacement as shown in equation 4.4.

In Figure 4.13, it can be seen that there is great difference between all beams in stiffness degradation. Beam SF-R-C is showing maximum stiffness degradation because of the type of confinement (CFRP) used in it. When we compare Ref beam with SF-R it is clearly visible that Ref beam has more stiffness degradation value perhaps reason is addition of steel fibers in SF-R beam. Similarly, steel sheet confined beams have lowest values of stiffness degradation as compared to all other beams due to steel sheet confinement. But when we compare stiffness degradation of steel confined beams then it can be seen that SF-R-S has less stiffness degradation because its stiffness at peak is much higher as compared to SF-0-S.

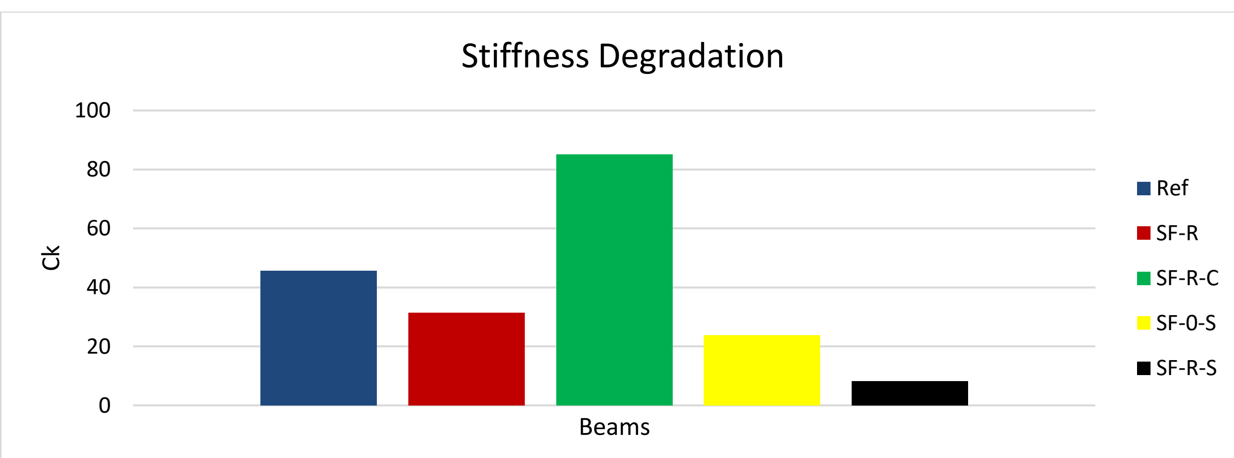

Figure 4.13: Stiffness Degradation

Failure mechanism of Beam Specimen

Figures 4.14, 4.15, 4.16, 4.17 and 4.18 are showing some specimens at the end of testing. Loading was stopped after decreasing of load to 20 percent of the Peak load or if beam was failed.

Figure 4.14 shows failure of Ref beam specimen. In this beam initially minor flexural cracks started under points of loading and in center of the specimen, but it started showing major flexure-shear cracks and finally failed due of those cracks.

Figure 4.14: Failure pattern of Reference Beam (Ref)

Figure 4.15 shows that SF-R beam failed in flexural conventional manner exactly under the points where load was applied. There are large cracks under the points of loading and small cracks in between the points of loading in tension zone of the beam due to excessive deflection.

Figure 4.15: Failure pattern of beam with flexural and steel fibers reinforcement (SF-R)

Figure 4.16 is representing SF-R-C. In this figure it is clearly visible that this beam failed due to debonding (Pre mature failure) of CFRP strip. Flexure-Shear cracks that initiated at ends of the beam due to high shear stresses, forced this debonding.

Figure 4.16: Failure pattern of beam with flexural and steel fibers reinforcement along with CFRP (SF-R-C)

Figures 4.17 & 4.18 show SF-0-S and SF-R-S beam specimens with and without longitudinal steel reinforcement which failed due to buckling of steel tube in compression zone and based on excessive deflection of the beam we can assume that concrete in the steel tube is also cracked. In SF-0-S in Figure 4.17 beam specimen without longitudinal reinforcement, failure occurred due to cracking of concrete, buckling at the top and rupture of steel tube at bottom. In Figure 4.18 in SF-R-S Buckling of steel tube at point of loading can be seen in the samples. Rupture and buckle of steel tube is at point of loadings in both cases. The specimen having flexural reinforcement showed delayed rupture.

Figure 4.17: Failure pattern of beam with steel fibers reinforcement strengthened with steel sheet (SF-0-S)

Figure 4.18: Failure pattern of beam with flexural and steel fibers reinforcement along with steel sheet (SF-R-S)

CHAPTER 5

CONCLUSIONS

The research on application of Artificial Intelligence was conducted to develop an Artificial Neural Network model for prediction of lateral load performance of Dhajji Dewari walls and to carry out a comparison that can be made between model data and experimental data. Different findings of this research work are discussed below.

- Neural Network have strong potential to carry out predictive study on lateral load performance of dhajji dewari.
- The developed model can help in parametric analysis of timber walls by considering five variables.
- Ultimately this research shows that Neural Network model having Layer Recurrent network with two hidden layers and twenty neurons in each hidden layer can be used in prediction of behavior of timber walls.
- Developed model is only applicable to experimental data covered by the papers followed however its exposure can be later extended on availability of data.

The focus of the second part of research was placed on enhancing flexural properties of High strength Concrete beams. For the said purpose steel fiber content was added by 1.5 % in replacement of concrete. Two different types of confinements (CFRP strip & Steel sheet) were used. Findings are given below.

- Ultimate Displacement of beam can be increased by addition of steel fibers and steel sheet confinement.
- Steel fibers reduce the crack widths and number of cracks under monotonic load.
- Steel fibers increase the shear strength of the beams.
- As in literature CFRP increases flexural strength and stiffness of the beams.
- CFRP confinement can increase flexural strength than steel sheet confinement.
- Flexure-Shear failure of control specimen was changed to flexure failure by adding steel fibers.
- Steel sheet confinement increases ductility and energy dissipation than control specimen.

- Inclusion of steel fibers in high strength concrete not only increase shear capacity but also increase bending strength also.
- Steel Fibers improve ductile behavior of the beams.

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