

**STRUCTURAL HYBRIDIZATION AND ECONOMICAL
OPTIMIZATION OF STRENGTHENING SYSTEMS USED
FOR CONCRETE BEAMS**

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**FACULTY OF ENGINEERING
UNIVERSITY OF MALAYA
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ECONOMICAL OPTIMIZATION OF
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CONCRETE BEAMS**

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ABSTRACT

Strengthening of an existing structure is often necessary to increase its load carrying capacity to meet new strength and serviceability requirements. However, strengthening can lead premature failure and efficient usage of the strengthening materials should be emphasized. Therefore, an efficient strengthening method along with the preparation of relevant design guidelines is urgently required. To address this issue a combination of external bonding reinforcement (EBR) and Near Surface Mounting (NSM) technique was developed and tested in this study. The proposed technique is called the hybrid strengthening method (HSM). In this study, efficient approach of strengthening reinforced concrete beam were also studied along with introduction of HSM. To prevent premature failure the use of end anchorage, shear strengthening and side HSM were employed. In order to make strengthening method efficient, steel bar with cement mortar was also used to replace the fibre reinforced polymer (FRP) and epoxy. Semi-numerical and finite element models were developed and validated with the experimental results to be used in the preparation of design guidelines. To help the designer reduce the strengthening cost further, mathematical design optimization techniques are also presented.

For this study, thirty-three reinforced concrete beams were cast and tested. These were designed to address the objectives described above. The strengthening materials used comprised of steel bars, steel plates and CFRP composites with different dimensions were used for strengthening. The beams were extensively instrumented to monitor loads, deflections, and strains. The beams were subjected to static and fatigue loadings.

Semi-numerical models were formulated to initiate the preparation of the design procedure of the HSM beam. In these models, an analytical approach was made with the help of the genetic algorithm optimization procedure to avoid time-consuming trial and

error. In addition, finite element models (FEM) from the ABAQUS package to predict flexural strength and deflection were used to do the parametric study. In the mathematical design optimization method, the strengthening cost was minimized using non-linear programming and genetic algorithms where flexural strength and serviceability requirements were used as the major constraints.

From the experimental results, the HSM beam, in general, gave about 65% higher flexural capacities as compared to the control beam at best. In terms of the efficiency, the HSM beams showed a 36% increase in flexural capacities as compared to the EBR beam. The partial replacement of epoxy adhesive with cement mortar in NSM strengthening reduced costs without significantly affecting the flexure performance. The fatigue performance of the HSM strengthened beam was found to be at least 6.5% higher than that of the NSM strengthened beam. The semi-numerical and finite element models were shown to be able to give consistent results as compared to the experimental results. The application of the optimization method led to savings of up to 8% in the amount of strengthening materials used as compared to classical design solutions.

ABSTRAK

Pengukuhan sesuatu struktur sedia ada selalunya diperlukan untuk meningkatkan kapasiti beban bagi memenuhi keperluan kekuatan dan servis baru. Namun, pengukuhan boleh menyebabkan kegagalan pra-matang dan penggunaan bahan pengukuhan yang efisien patut diutamakan. Oleh itu, teknik pengukuhan yang efisien berserta dengan penyediaan garis panduan rekabentuk yang berkaitan adalah sangat diperlukan. Untuk menyelesaikan isu ini, satu kombinasi pemasangan pengukuhan luaran (EBR) dan juga teknik pemasangan berhampiran permukaan (NSM) telah dihasilkan dan diuji dalam kajian ini. Teknik ini dinamakan sebagai teknik pengukuhan hibrid (HSM). Dalam kajian ini, kaedah yang efisien untuk pengukuhan rasuk bertetulang konkrit juga telah dikaji berserta dengan pengenalan kepada HSM. Untuk mengelakkan kegagalan pra-matang penggunaan labuh pada hujung, pengukuhan ricih dan HSM jenis sisi telah digunakan. Untuk menghasilkan teknik pengukuhan yang efisien, batang besi dengan mortar simen telah digunakan untuk menggantikan serat polimer bertetulang (FRP) dan epoxy. Model-model separa numerikal dan unsur terhingga telah dihasilkan dan disahkan dengan keputusan eksperimental untuk digunakan dalam penyediaan garis panduan rekabentuk. Untuk membantu mengurangkan kos pengukuhan, teknik rekabentuk optima jenis matematik juga telah disediakan.

Dalam kajian ini, tiga puluh rasuk bertetulang konkrit telah dihasilkan dan dikaji. Semuanya direkabentuk untuk menepati objektif yang diberikan di atas. Bahan teknik pengukuhan yang digunakan adalah batang besi, papan besi dan juga komposit CFRP dengan dimensi yang berbeza telah digunakan untuk pengukuhan. Rasuk-rasuk tersebut telah dipasang instrumen untuk memonitor beban, pesongan dan keterikan. Rasuk-rasuk tersebut kemudian dikaji di bawah beban statik dan beban letih.

Model-model semi-numerikal telah dihasilkan bagi memulakan penyediaan garis panduan rekabentuk rasuk HSM. Dalam model-model ini, satu kaedah analitikal telah dihasilkan dengan bantuan prosedur pengoptimuman algoritma genetik untuk mengelakkan teknik cuba dan gagal yang memakan masa. Di samping itu, model-model unsur terhingga (FEM) daripada ABAQUS untuk mengira kekuatan lenturan dan pesongan telah digunakan untuk melakukan kajian parametric. Dalam kaedah rekabentuk pengoptimuman matematik, kos pengukuhan telah dikurangkan dengan menggunakan program tidak linear dan algoritma genetik di mana kekuatan lenturan dan keperluan servis telah digunakan sebagai kekangan utama.

Daripada keputusan eksperimen, rasuk HSM memberikan kapasiti lenturan yang 65% lebih tinggi berbanding rasuk kawalan. Dari segi kecekapan, rasuk HSM memberikan 36% peningkatan kekuatan lenturan berbanding rasuk EBR. Penggantian sebahagian daripada gam epoxy dengan mortar simen dalam teknik pengukuhan NSM telah mengurangkan kos tanpa mengubah prestasi lenturan dengan ketara. Prestasi keletihan untuk rasuk HSM didapati adalah sekurang-kurangnya 6.5% lebih tinggi berbanting rasuk NSM. Model-model semi-numerikal dan model unsur terhingga ditunjukkan mampu memberi keputusan yang konsisten dengan keputusan eksperimental. Aplikasi teknik pengoptimuman memberikan penjimatan sebanyak 8% dari segi jumlah bahan pengukuhan yang digunakan berbanding teknik rekabentuk klasik.

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LIST OF SYMBOLS

a	:	The depth of stress block
a_e	:	Edge clearance
a_g	:	Clear spacing of NSM grooves
A_s	:	Cross sectional area of steel bar
A_f	:	Cross sectional area of plate
b	:	Width of the concrete beam specimen
b_f	:	Width of strengthening plate
C	:	Total cost of strengthening system
C_F	:	Cost of carbon fiber reinforced polymer plate
C_a	:	Cost of adhesive
d	:	Effective depth of concrete beam specimen
d_c	:	Depth of concrete cover
d_x	:	Depth of compressive force carried out by concrete
E_c	:	Modulus elasticity of concrete
E_p	:	Modulus elasticity of strengthening plate
E_s	:	Modulus elasticity of steel bar
ε	:	Strain
ε_c	:	Strain of concrete
ε_{cu}	:	Ultimate strain of concrete
ε_{nsm}	:	Strain of NSM bar
ε_p	:	Strain of strengthening plate
ε_s	:	Strain of main tensile steel
F_{cc}	:	The force carried by the concrete
E_{fu}	:	The ultimate strain of FRP

F_{nsm}	:	The force carried by the NSM steel bar
F_p	:	The force carried by the strengthening plate
F_s	:	The force carried by the steel bar
f_y	:	Yield strength of steel bar
h	:	Height of the concrete beam specimen
l	:	Span length
L_f	:	Length of the strengthening plate
M	:	Moment
m	:	Meter
M_r	:	Resisting bending moment
$M_{r,b}$	:	Balance moment of resistance
t_f	:	Thickness of the strengthening plate
V_c	:	Shear capacity of the concrete
V_{cap}	:	Shear capacity of the beam
V_s	:	Shear capacity of the steel bar
w	:	Uniformly distributed load
c	:	Depth of neutral axis
z	:	Lever arm

LIST OF ABBREVIATIONS

ACI	:	American Concrete Institute
ACO	:	Anti-Colony Optimization
CFRP	:	Carbon Fiber Reinforced Polymer
EMPA	:	Swiss Federal Laboratories For Materials Science And Technology
EBR	:	External Bonded Reinforcement
FEA	:	Finite Element Analysis
FEM	:	Finite Element Modeling
FHB	:	Friction Hybrid Bonded
FRP	:	Fiber Reinforced Polymer
GA	:	Genetic Algorithm
HF	:	Beam Strengthened with CFRP using HSM
HS	:	Beam Strengthened with Steel using HSM
HSM	:	Hybrid Strengthening Method
JSCE	:	Japan Society of Civil Engineering
LVDT	:	Linear Variable Displacement Transducer
NLP	:	Non Linear Programming
NSM	:	Near Surface Mounting
PF	:	Beam Strengthened with CFRP using EBR
PS	:	Beam Strengthened with Steel using EBR
PSO	:	Particle Swarm Optimization
RC	:	Reinforced Concrete

CHAPTER 1: INTRODUCTION

1.1 Research Background

Rehabilitation and strengthening of reinforced concrete (RC) structures are some of the major challenges for structural engineers today. The strengthening of RC structures is a dynamically growing division of structural engineering and in recent years, there has been an increase in the application of new repair and strengthening systems for RC load-carrying structures. In most cases, it is an increase in dead and live loading that has to be safely carried by the structures, as well as their poor technical performance that necessitates the use of strengthening procedures.

The main reasons why structural strengthening is done are to:

- i. Safely accommodate increases in dead and live loading,
- ii. Counter material aging and corrosion,
- iii. Offset mechanical damage,
- iv. Reduce strain limits in order to maintain composite action,
- v. Decrease stress in steel reinforcement for fatigue consideration,
- vi. Decrease crack widths to maintain serviceability,
- vii. Modify a structure's static scheme to adapt to a changed situation, and
- viii. Overcome construction failures.

Structures that have been built more than several decades ago often require strengthening and upgrading to meet current service load demands. Thus, the use of strengthening techniques is expected to grow rapidly over the next few years. Several methods of strengthening RC structures using various materials have been studied and applied in the rehabilitation field (Eberline et al., 1988; Juozapaitis et al., 2013; Macdonald & Calder, 1982). However, no solution can be applied to all cases as each specific structure must be approached on an individual basis (Kamiński & Trapko, 2006).

Selection of the proper strengthening method requires careful consideration of many factors including the following engineering issues:

- i. Amount of the required increase in strength.
- ii. Effect of variations in relative stiffness.
- iii. Size of the works.
- iv. Environmental situations (adhesives might not be suitable for use in high-temperature environments; external steel may not be suitable in corrosive environments).
- v. In-situ concrete strength and substrate integrity.
- vi. Constraints dimension or clearance.
- vii. Accessibility.
- viii. Operational limitations.
- ix. Availability of local materials, equipment, and experienced contractors.
- x. Construction, maintenance and life cycle costs.

One technique commonly used to enhance the strength or serviceability of RC structures is the gluing of steel or CFRP plates to the outer surfaces of the structures. This method has been employed universally since the late 1960s (Hermite & Bresson, 1967). However, the use of this technique usually suffers from premature failure like plate end separation, intermediate crack induced debonding or shear failure. This debonding can cause serious brittle and catastrophic failure before the strengthened beam reaches its ultimate capacity.

Many studies have been conducted to find solutions to this brittle debonding and to reduce the interfacial stresses between the RC substrate and the strengthening plate (Fitton & Broughton, 2005; Hildebrand, 1994). One remedy was to change the thickness of the steel or FRP plate or the joint geometry by tapering the plate (Tsai & Morton,

1995). Although the use of geometrical variations to the plate ends by tapering the form is a useful tool to reduce the stresses in adhesive joints, it is a complex, time consuming and costly process. This solution aims to control the allowable strain in FRP plates to a threshold value to prevent debonding but the results of this approach are not efficient (Radfar et al., 2012).

More recently, near surface mounted (NSM) reinforcement has been the subject of fascination in an increasing amount of research as well as realistic application because it is less prone to premature debonding (De Lorenzis & Teng, 2007). However, there are some limitations to its application. Sometimes, the width of the beam may not be sufficiently wide to provide necessary edge clearance and clear spacing between two adjacent NSM grooves. ACI 440 recommends that the minimum edge clearance and clear spacing for the NSM grooves should be four and two times the groove depth, respectively. However, this recommendation has also been proven to be inadequate by Lorenzis (2002). Additionally, the thickness of the concrete cover should be high enough to provide sufficient groove depth.

The use of NSM steel bars to strengthen RC structures started in Europe in the early fifties (Lorenzis & Nanni, 2002). The earliest reference to this technique dates back to 1949 (Asplund, 1949), where steel bar with cement grout was used to strengthen a concrete slab in field construction work. More recent use of NSM steel bars for the strengthening of masonry structures and arch bridges have also been documented (Garrity, 2001). Most experimental studies on this strengthening technique investigate the flexural behaviour of concrete beams strengthened using NSM FRP bars or strips (Al-Mahmoud et al., 2009; Badawi & Soudki, 2009; El-Hacha & Rizkalla, 2004a; Lorenzis et al., 2000; Soliman et al., 2010). The test results confirm that NSM FRP bars can be applied to increase the flexural capacity of RC elements. However, little or no

experimental investigation has been done on the flexural behaviour of concrete beams strengthened with NSM steel bars.

The present research work would like to present a hybridization of the above two strengthening methods in order to address the shortcomings of both methods. This hybridization combines the externally bonded reinforcement (EBR) with the NSM technique so that they complement each other and mutually reduce their limitations. This method is called the hybrid strengthening method (HSM). Previous research has shown that decrease in plate thickness reduces the magnitude of stress concentration at the plate extremities. Instead of tapering the plate, this new bonding method could make it possible to reduce the plate thickness by transferring a portion of the required amount of strengthening material from the EBR to the bars in the NSM technique. Consequently, the size or number of NSM bars required can also be reduced by sharing the amount of strengthening material needed with the plate in the EBR. This can then ensure enough space for edge clearance and clear spacing of the NSM groove.

The main purpose of the HSM is to increase bond performance against plate end debonding failure between the concrete substrate and the strengthening plates and bars. Plate end debonding can probably be prevented or delayed through the reduction of interfacial stress. The HSM method can reduce interfacial shear and normal stress in two ways. The first way is by decreasing the thickness of the plate by transferring some of the required material from the plate to the NSM system. After transferring, the total amount of strengthening material used on the structure will be the same. However, the magnitude of interfacial stress will be decreased due to reduced plate thickness. Plate thickness is one of the most important parameters in reducing interfacial stress (Lousdad et al., 2010).

Most codes of practice (Fib, 2001; JSCE, 2001) also recommend limiting design strain on the plate to eliminate debonding. Other studies (Maruyama & Ueda, 2001; Teng et al.,

2003) have confirmed similar limits. In most cases, the design debonding strains are inversely proportional to plate thickness. For a fixed FRP ratio, the debonding potential has been shown to increase significantly with increasing FRP thickness (Garden et al., 1997). Although the above studies used FRP plates for strengthening, the findings are also applicable to strengthening with steel plates.

Several studies have focused on steel plate end debonding. Swamy et al. (1987b) showed that premature end debonding of steel plates can be avoided by increasing the aspect ratio of the plate by more than fifty. Swamy and Mukopadhyaya (1995) have demonstrated that this criteria holds true for FRP plates when glass, glass-carbon, and aramid fibers are applied. Oehlers (1992) proposed a formula based on the interaction between flexural and shear capacities of the beam where the debonding failure moment is inversely proportional to plate thickness. Zibra et al. (1994) presented a model based on the shear capacity of the beam where debonding shear force decreases with steel plate thickness. Hassanen and Raoof (2001) proposed that design strain on the plate is inversely proportional to plate thickness. Therefore, reducing plate thickness is an effective way to prevent plate debonding.

The second way, the HSM method can reduce interfacial stress by increasing the surface area in contact between the strengthening plate and the concrete face. The HSM involves cutting grooves along the beam for NSM strengthening. The grooves increase the bonding surface area between the plate and the concrete substrate. As stress is equal to load divided by the corresponding surface area, an increase in surface area will decrease interfacial shear stress. Moreover, the addition of adhesive in the NSM grooves in the hybrid strengthening system further improves bond performance between the strengthening plate and the concrete substrate.

In order to confirm the advantages of the HSM mentioned above, the structural performance of the RC beam strengthened with the new method needs to be fully characterized. Even though there are some test data on the structural behaviour of strengthened beams using the above two existing methods, it is difficult to find any test data on the experimental behaviour of strengthened beams using hybridization of the EBR and the NSM technique. The HSM has the potential to take advantage of both methods and complementarily eliminate their respective shortcomings. Therefore, the HSM could become an effective and efficient method to strengthen structural members through proper utilization of materials.

Proper utilization of materials is an important parameter in the constructing and strengthening of structures. There is an increasing demand to reduce construction costs of structures to cope with universal competition and this has encouraged structural engineers to find more efficient structural strengthening systems. A number of design guidelines have been published in different countries for the design of RC structures strengthened with FRP. However, the design of RC beams and their structural strengthening systems involves performing preliminary elastic analyses based on assumed dimensions and then examining the member for its adequacy against strength, serviceability and other requirements as imposed by the design codes. If the requirements are not satisfied, then the cross-sections are modified repeatedly until they satisfy the requirements of the codes. This process is carried out repetitively without consideration of the relative costs of the structural strengthening system's component materials. As a result, a situation in which excessive material is used usually occurs, this results in higher costs than necessary. A guideline is therefore required to determine the minimum amount of materials needed to adequately strengthen a structure to maintain its functionality and thus optimize the total cost of the structural strengthening system.

Material cost is an important factor in designing and implementing external strengthening systems for RC structures. The main factors influencing the cost are the amount of steel, FRP and adhesive to be used. Labor and formwork costs are also significant. It is therefore necessary to make RC strengthening structures less expensive, while still satisfying serviceability and strength criteria. Many researchers have used several optimization techniques for the design of RC structures. Kanagasundaram and Karihaloo (1991) formulated cost optimization as a non-linear programming problem. Adamu et al. (1994) developed a method based on a continuum-type optimality criteria, while Han et al. (1996) used discretized continuum-type optimality criteria. Leps and Sejnoha (2003) used genetic algorithms to optimize RC beams while Camp et al. (2003) used them for structures. However, no research has been found to optimize structural strengthening except Perera and Varona (2009), where genetic algorithms were used for discrete optimization.

Based on the discussion of the research background above, a number of research gaps have been found and are summarized below:

- i. Strengthening of RC beams using NSM steel bars lacks investigation.
- ii. Effect of replacing epoxy adhesive with cement mortar on the behaviour of NSM strengthened RC beams has not yet been studied.
- iii. Hybridization of EBR with NSM technique is a potential research area.
- iv. A design methodology for the HSM has yet to be devised.
- v. Optimum design methods to strengthen RC beams are rare and limited.

The present study explores the use of steel bars and cements mortar in the NSM system and develops a new structural strengthening method that combines the conventional EBR with the NSM technique into a hybrid strengthening system. A number of RC beam specimens are strengthened using different configurations of steel bars and steel or CFRP

plates, which are then subjected to static and fatigue loading. With extensive use of instruments, the beams are constantly monitored for loading, deflections and strains over the entire spectrum of loading to failure. The effects of different parameters on the performance of the RC beams strengthened using the HSM are investigated implicitly. The present research also utilizes optimization techniques coupled with advanced computer aided tools in the process of creating conceptual and detailed designs of the structural strengthening system. Therefore, the present study also describes the development of an easy and efficient model for optimizing the design of FRP strengthened RC beams. The model uses two mathematical methods, which are non-linear programming and genetic algorithms. The use of the hybrid strengthening technique and the optimum design method may lead to significant savings in the amount of component materials used in strengthening as compared to classical solutions.

1.2 Goal and objectives of the Study

The ultimate goal of this study is to make a more efficient structural strengthening system using newly proposed hybrid strengthening method (HSM). The efficiency of a system can be defined as the performance divided by the corresponding cost of the system. The performance of strengthening will be increased using two means: method and material hybridization. The material hybridization will be achieved by the replacement of epoxy adhesive with cement. The HSM will be a combination of the EBR and NSM method. To reduce the cost of strengthening, the design will be optimized using a mixture of genetic algorithm with non-linear programming. Therefore, the goal of this research is supported by a number of objectives.

The objectives of this research work can be summarized as follows to:

- i. Develop a strategy for eliminating premature failures of strengthened beams including the introduction of the hybrid strengthening method (HSM).
- ii. Study the effectiveness of using cement mortar to replace epoxy and steel bar to replace FRP in the NSM strengthening method.
- iii. Conveyance the fatigue performance of RC beams strengthened with HSM, EBR, and NSM.
- iv. Develop a semi-numerical model and finite element model (FEM) to predict flexural strength and deflection of RC beams strengthened using the HSM.
- v. Propose an economical approach for flexural strengthening of RC beams with CFRP plate based on non-linear and genetic algorithms.

1.3 Research Methodology

Three methods are used to achieve the objectives of this research work. The three methods are: conducting an experimental programme, developing a semi-numerical and finite element model and using mathematical optimization. Extensive experimental investigations were done to achieve the first to fourth objectives of this study as listed above. To achieve the fifth objective, a semi-numerical model was formulated of the strengthened beams and the original un-strengthened control beam. Similarly, a finite element model was developed to achieve objective six. Mathematical programming and an evolutionary algorithm-based optimization technique were applied to achieve the seventh objective.

1.4 Chapter Outline

The thesis comprises of five chapters dealing with various aspects of strengthening RC beams for static and fatigue loading. A brief outline follows:

Chapter 1 gives a general introduction of the research to be dealt with. A short research background on recent advancement of strengthening RC beams is presented. Accordingly, the goal, purposes, research objectives and brief methodology are discussed. The chapter concludes with an outline of the thesis.

The second chapter presents a thorough review of relevant literature. A concise survey is given of recent literature on the use of external bonding and the NSM technique to strengthen RC beams under monotonic and fatigue loadings and on the application of design optimization techniques.

The third chapter presents the experimental program, specimen fabrication, test instrumentation and loading test set-up. The choices for the different parameters is explained and justified. A methodology for the design optimization of RC beams strengthened with FRP composites is also presented.

The fourth chapter presents the results and discussion. A description of the performance of the strengthened beams under test conditions is qualitatively compared to the behaviour of an un-strengthened control beam. The results are compared with the findings of previous related studies. Observations and possible solutions for design optimization are also discussed in this chapter.

Finally, the fifth chapter summarizes the main findings of the research work and highlights conclusions. Recommendations for further research are also given in this chapter.

The appendices present selected results in more detail, as well as necessary hand computations for the research.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

This chapter discusses existing works that are relevant to the objectives of the present study, and identifies the gaps in the existing research that will be addressed by this study. Furthermore, it presents the research questions of this study. This chapter has the following six sections. Section 2.2 reviews experimental investigations on structural strengthening, specifically the NSM reinforcement technique and the EBR method. The limitations of each strengthening method are also discussed. In addition, this section outlines various studies that have investigated the fatigue performance of RC beams strengthened with steel or FRP, using either the NSM technique or the EBR method. Section 2.3 reviews research on the numerical modelling of RC beams strengthened with steel and FRP in order to predict flexural behaviour. Section 2.4 discusses works on the optimization of the structural design of strengthened RC structures. Section 2.5 identifies the gaps in research and points out the significance of the present study. Section 2.6 presents the research questions for this study.

2.2 Experimental Investigations on Structural Strengthening

Several materials and methods have been used for structural strengthening. The common material used in strengthening includes spray concrete, ferro-cement, steel plate and FRP. Diab (1998) reported the use of spray concrete. Romualdi (1987) and Iorns (1987) introduced the use of ferro-cement, which was later utilized by Paramasivam et al. (1998). However, the most frequently used materials for structural strengthening are steel plate and FRP, of which FRP is especially promising. There are different types of FRP, including carbon, glass and aramid. FRP can also be found in various forms such as pultrusion plates, sheets and fabrics.

Common structural strengthening methods include section enlargement, external pre-stressing, external bonding and near surface mounting (NSM). The technique of bonding steel plates or carbon fibre reinforced polymer (CFRP) plates to the external surfaces of RC structures to enhance their strength or serviceability has been employed worldwide since the late 1960s (Hermite & Bresson, 1967). More recently, the NSM technique using FRP has become the subject of fascination in a large amount of research and has many practical uses.

2.2.1 External Bonding Reinforcement (EBR)

Research study into the behaviour of structural members strengthened with steel plates was started concurrently in South Africa and France in the 1960s (Fleming & King, 1967; Gilibert et al., 1976; Hermite & Bresson, 1967; Lerchenthal, 1967). The first application of epoxy bonded steel plates for strengthening concrete beams was reported in 1964 in Durban, South Africa. Further development of proper adhesives encouraged more research work.

Preliminary research works were made by Irwin (1975), Macdonald (1978) and Macdonald and Calder (1982). Macdonald and Calder (1982) made four-point loading tests on RC beams strengthened with steel plates, 4900 mm in length. Strengthening with steel plates of existing structures has also been studied in Switzerland at the Swiss Federal Laboratories for Material Testing and Research (EMPA) (Ladner & Weder, 1981). Bending tests were conducted on RC beams 3700 mm in length, and the effect of the plate aspect ratio was studied while the plate area was kept constant. Summary studies on EBR are given in Table 2.1

Table 2.1: Summary literature review on EBR

Sl. no	Authors	Parameters/ Variables	Findings
1	Jones et al. (1988)	Type and size of end anchorage, length	The different anchorage systems caused no apparent variations on the deflection performance of the beams. The use of bolts did not prevent debonding
2	Hussain et al. (1995)	Effectiveness of anchor bolt	Bolts were found to improve the ductility of the plated beams considerably, but to only marginally effect the ultimate load capacity, agreed with Jones et al. (1988)
3	Saadatmanesh and Ehsani (1989)	Size of glass FRP (GFRP)	Flexural strength increased with increasing area of the GFRP sheets
4	Meier and Kaiser (1991)	Effect of strengthening	The beams doubled in strength, but were less ductile as the reduced deflections at failure, The CFRP laminates also caused a more distributed cracking pattern with reduced crack widths. Other researchers have subsequently found similar results (Beber et al., 1999; Heffernan & Erki, 1996; Jonaitis et al., 1999; Kachlakev, 1999; Naaman, 1999; Swamy et al., 1996a; Swamy et al., 1996b)
5	An et al. (1991)	Internal steel ratio, stiffness of plate	beams with high internal steel ratios were more effectively strengthened using a stiffer plate with high strength concrete than with a plate of lower stiffness with low strength concrete. Cha et al. (1999) found similar results
6	Triantafillou and Plevris (1992)	Number of FRP layer	The debonding of FRP limited the number of FRP layers that could be used.
7	Hutchinson and Rahimi (1993)	FRP type and thickness	Using either GFRP or CFRP remarkably increased the flexural capacity of RC beams.
8	Triantafillou and Plevris (1995)	Reliability, reduction factors	They proposed a general strength reduction factor of 0.85 and a partial reduction factor of 0.95 for FRP strength.
9	White et al. (1998)	Loading rates	Service and ultimate flexural capacity increased as the rate of loading increased. Cracking and

			failure modes were not affected by the rate of loading
10	Toutanji et al. (2001)	Types of Matrices, no. of layers	The inorganic matrix system was effective in increasing the strength and stiffness of the RC beams, but the failure mechanism of the inorganic system seemed more brittle
11	Kurtz and Balaguru (2001)	Types of Matrices	The inorganic matrix and the organic matrix were equally effective at increasing the strength and stiffness of the beams, although the inorganic matrix slightly reduced ductility
	Spadea et al. (2001)	CFRP layouts	Externally bonding a CFRP plate to strengthen the RC beams increased the flexural strength but reduced ductility
	Rasheed and Pervaiz (2003) and Spadea et al. (1998)	Shear modulus and thickness of Adhesive	The FRP tension force cannot be fully developed when the adhesive shear modulus is below 65 MPa/mm (239 ksi/in.) of the adhesive layer thickness regardless of the length of the plate
	Brena et al. (2003)	Layout of CFRP	The use of CFRP U-wraps delayed or prevented the CFRP composite sheets from debonding
	Alagusundaramoorthy et al. (2003)	Type of FRP, anchorage	The increase in strength was 49% and 40% for beams strengthened with CFRP sheet and fabric, respectively. A 58% increase was achieved when anchorages were used.
12	Akbarzadeh and Maghsoudi (2011)	Effect of hybrid FRP	Using HCG to strengthen the continuous RC beams led to considerable increases in the bearing capacity
13	Attari et al. (2012)	Effect of hybrid FRP	The cost-effectiveness of using twin layers of glass-carbon FRP fabric as a strengthening configuration for RC structures
	Rami et al. (2014)	Effect of hybrid FRP	The ductility at failure loads of the beams strengthened with glass and hybrid sheets is higher than that with a single carbon sheet

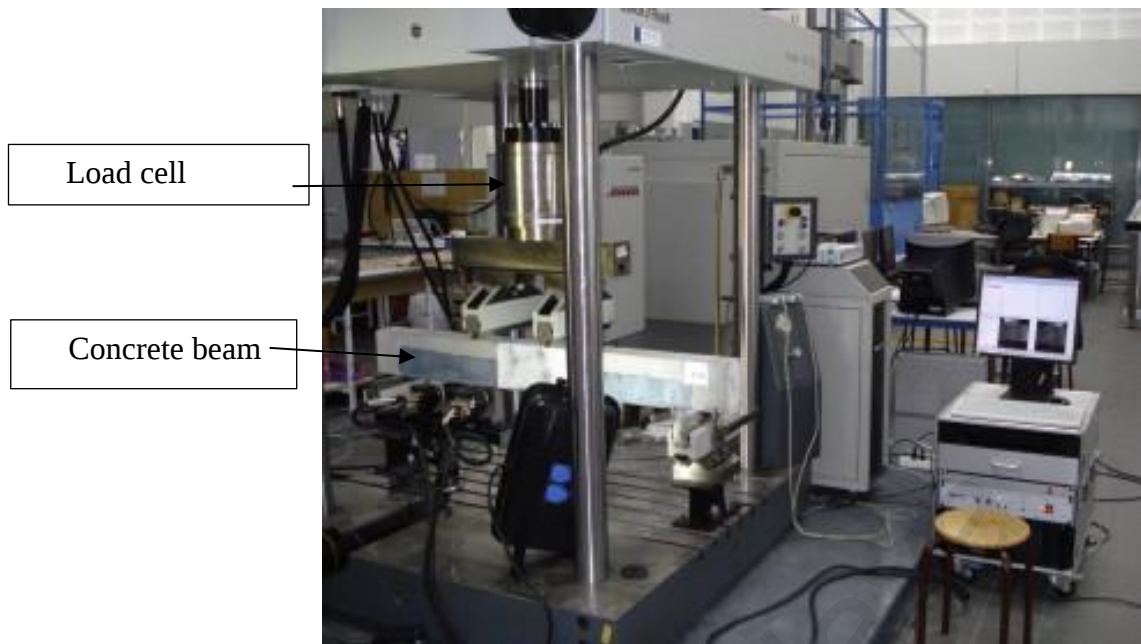


Figure 2.1 : Strengthened RC beams tested by Attari et al. (2012)

2.2.2 Limitations of EBR System

The use of CFRP sheets or strips without appropriate anchorage severely decreases structural ductility and causes early debonding of the CFRP laminate. The strengthened member cannot reach the theoretical ultimate strength calculated by assuming a perfect bond between the laminate and concrete. By anchoring CFRP laminates using bolts and steel plates for end anchorage, and steel or FRP straps along the beam, the composite action of the strengthened beam can be maintained up to its ultimate load. However, for a span to depth ratio greater than 4.0, anchorage had no effect on the peeling-off of the laminate.

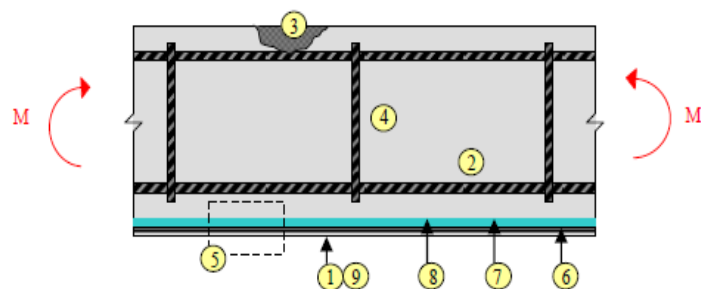


Figure 2.2. Different failure modes of EBR system

Ten failure modes are theoretically possible in RC beams externally strengthened with either steel or FRP materials. The first nine failure modes are as shown in Figure 2.2 (the number in parenthesis indicate type of failure mode in Figure 2.2) and names of ten failure modes are given below:

- i. Rupture of the strengthening plate (1),
- ii. Rupture of the internal reinforcement (2),
- iii. Crushing of concrete in the compression zone (3),
- iv. Shear failure (4),
- v. Failure caused by debonding (peeling-off) (5),
- vi. Rupture of the strengthening-adhesive interface (6),
- vii. Rupture of the concrete-adhesive interface (7),
- viii. Cohesive failure within the adhesive (8),
- ix. Interlaminar shear failure within the CFRP material (observed as a secondary failure) (9), and
- x. Concrete cover peeling off.

For steel plated RC beams, the ultimate failure mode appears to be closely related to the geometry of the plated cross-section. Thin plates usually fail in flexure. However, when the plate aspect ratio falls below a certain value, separation of the plate from the beam can occur. This usually starts from the plate end and results in the concrete cover being ripped off. These observations are consistent with the fact that simple elastic longitudinal shear stresses are inversely proportional to plate width. Therefore, as the steel plate width decreases, the longitudinal shear stresses increase. The bending stiffness of the plate also increases, and thereby increases the peeling stresses normal to the beam.

Shear and normal stresses become concentrated at the plate ends of strengthened beams subjected to flexure. This is caused by incompatibility between the plate stiffness and the concrete stiffness. This incompatibility can only be overcome by severe distortion of the adhesive layer. The transition area from the basic member to the plate reinforcement is a region of high shear and low bending moment. The changing bending moment and distortion in the adhesive layer causes a build-up of axial forces at the ends of the external plate. This leads to high bond stresses in the adhesive to plate and adhesive to concrete interfaces, which may reach critical levels and thereby cause failure. The magnitude of these plate end stresses depends upon a number of factors. These factors are: the geometry of the plate reinforcement, the engineering properties of the adhesive and the shear strength of the original concrete beam (Swamy & Mukhopadhyaya, 1995). The peak peeling and shear stresses at the plate ends, in addition to bending stresses, result in a biaxial tensile stress state. This causes the cracks initiated at the plate ends to extend horizontally at the level of the internal steel reinforcement.

When failure occurs in delamination, the use of a more flexible adhesive is advantageous, since the area over which the tensile strain builds up in the external steel plate is increased. This results in a lower peak stress. Jones et al. (1988) verified this procedure experimentally. Beams strengthened using an adhesive with an elastic modulus of around $1.0 \times 10^3 \text{ N/mm}^2$ gave slightly improved strengths when failure occurred by plate separation than strengths given by an adhesive with a modulus of around $10 \times 10^3 \text{ N/mm}^2$.

Many models have been developed and proposed in the past to predict plate end debonding failure load. However, no existing model can accurately predict the failure load when cover failure occurs (Smith and Teng, 2002). In addition, they could not consider and distinguish end delamination and concrete cover separation.

2.2.3 Eliminating Premature Debonding in EBR

To avoid peeling failures several approaches have been investigated and developed. Some of these include mechanical anchorages at the ends of the sheet, wrapped sheets around the web of the beam over the longitudinal FRP sheet, and changes in the geometry of the sheets in the anchorage zones as suggested by Karam (1992).

Swamy et al. (1987a) showed that premature debonding of steel plates can effectively be avoided by ensuring that the width to thickness ratio of the plate is not less than 50. Swamy and Mukopadhyaya (1995) have shown that this recommendation holds true for FRP plates when glass, glass-carbon, and aramid fibers are used. In the case of CFRP, the plates are generally so thin that this criterion is automatically satisfied. Another technique that can be used if multiple layers are being prestressed is to end the layers (and transfer the prestress) at different locations along the beam. This technique is effective at reducing the magnitude of shear and normal stress concentrations that occur at the plate ends (Wight, 1998).

By using bonded angle plates or transverse FRP wraps, longitudinal FRP strips can be effectively anchored to the tensile face of the beam. The effectiveness of this technique has been commented on by Adimi et al. (2000), who found that a great deal of ductility could be observed until rupture of the plate, shear failure of the wrap, or angle plates.

Jones et al. (1988) mentioned that the bonded anchor plates were more effective, producing yielding of the tensile plates and allowing the full theoretical strength to be achieved, 36% above that of the unplated control beam. The anchorage detail was also found to affect the ductility of the beams near the ultimate load. Unanchored, the beams failed suddenly with little or no ductility. The beams with bolts or anchor plates all had similar ductility's, at least as high as the unplated control.

Deblois et al. (1992) compared bonded unidirectional and bidirectional GFRP sheets with bolted unidirectional GFRP sheets using 1.0 m and 4.1 m specimens. They found that bolted sheets and bidirectional sheets were more effective than unbolted unidirectional sheets.

Sharif et al. (1994) studied three different anchorage schemes for GFRP plates using small-scale beams. The three schemes were bolting the plates to the tension face, bolting combined with FRP plates bonded to the sides of the beams, and a special one-piece I-jacket plate that was glued along the bottom of the whole span to the sides of the beam in the shear span. The I-jacket was the most effective anchorage scheme as it prevented all types of peeling failures. Bolting the plates was not very effective as the beams failed by shear peeling.

Hussain et al. (1995) tried anchor bolts. The percentage improvement in ductility due to the addition of bolts was found to decrease as plate thickness increased. The end anchorage could not prevent premature failure of the beams, although in this case failure occurred as a result of diagonal shear cracks in the shear spans. Providing anchorage to steel plated beams involves considerable extra site work and this increases the cost of EBR considerably. However, in the case of steel EBR, the use of anchorage is completely necessary.

The University of Surrey (Quantrill et al., 1996) under the ROBUST programme of research, conducted a parametric study on RC beams flexurally strengthened with GFRP bonded plates. The study varied a number of parameters. These were the concrete strength, the pultruded composite plate area and its aspect ratio. As mentioned above, thick narrow plates that have an aspect ratio of less than 50 have been linked with brittle peeling failure modes. Thus, this study tested plates with aspect ratios of 38 and 67. The effect of the width to thickness ratio was isolated in these tests by keeping the plate cross-

sectional area constant. The tests found that plating considerably enhances both the strength and stiffness of RC members, although this is at the expense of ductility at failure. The study also observed that higher strength concrete produced the greatest increase in strength over unplated sections and that the aspect ratio of the plates had little effect on the overall behaviour of the beams.

The ROBUST programme conducted further investigations at the University of Surrey (Quantrill et al., 1996) on the experimental and analytical strengthening of RC beams with FRP plates. They analyzed the effects of different plate parameters on the overall behaviour of the strengthening system. The study showed that relatively small scale 1 m long specimens can be tested to reveal useful information on the behaviour of strengthened beams. Reducing the plate area led to an expected reduction in strengthening and stiffening, which caused the ductility and the plate strains for a given load to increase. The aspect ratio for the values tested had little effect on the overall behaviour of the beams. Using CFRP plates increased the serviceability, yield and ultimate loads and increased the stiffness of the strengthened members after both cracking and yielding. The ductility of the strengthened beams was reduced. For a partially cracked section, the tensile plate strain and compressive concrete strain responses of the beam were accurately predicted by the iterative analytical model.

Garden and Hollaway (1998) tested concrete beams strengthened with CFRP plates to study the effects of three parameters: plate aspect ratio (plate width divided by plate thickness) at constant cross sectional area; shear span-depth ratio of beams; and the form of plate end anchorage. The plates were anchored by extending them under the supports, or attaching them to GFRP angles bonded to the sides of the beams. The ultimate capacities of the strengthened beams decreased with reducing plate width–thickness ratios. Failure was always accompanied by concrete cover separation from internal

reinforcement. Increasing the shear span-depth ratios resulted in improved ultimate capacities. Anchoring the sheets increased the strength of the beams.

Spadea et al. (1998) and Gemert (1999) also found that wrapping the sides of the beams with vertical FRP sheets provided effective anchorage for the flexural sheets. Quantrill et al. (1996) found that GFRP angles attached to the sides of the beams were effective anchors. On the other hand, Naaman (1999) tested 3 m span T-beams and did not find any improvement in the strength of the beams when U-shaped anchors were used.

Teng et al. (1999) conducted an experimental study into strengthening deficient cantilever concrete slabs by bonding GFRP strips on the top surface. Different anchorage systems were used, and the most effective method was to anchor the GFRP strips into the walls through horizontal slots and into the slab with fiber anchors. This method allowed the full strength of the strips to develop, and the strength was almost four times that of the unstrengthened beam.

Bencardino et al. (2007) tested CFRP plated beams and recorded the reduction in member ductility due to plating without end anchorage. The ductility of the strengthened beams was restored when anchorage in the form of externally bonded U-shaped steel stirrups was fitted on to the plated beams. The study then successfully used this method of CFRP plating to strengthen an experimental portal structure.

Xiong et al. (2007) attempted to strengthen RC beams by combining unidirectional CFRP sheets (to bond to the tension faces of the beams) and bi-directional GFRP sheets (to wrap three sides of the beams continuously). The feasibility and potential advantages of this approach were discussed. A comparative test program using ten beams was carried out. The test results showed that the hybrid CFRP and GFRP (H-CF/GF-RP) strengthening not only prevented the tension delamination of the bottom concrete cover,

but also lead to a significant increase in the deformation capacity of the strengthened beams at a very low cost compared to CFRP strengthening alone.

Galal and Mofidi (2009) explored a new hybrid FRP sheet and ductile anchor system for the rehabilitation of RC beams. The study reports that the advantage of this strengthening method is that it overcomes the problem of low ductility that is connected to the brittle failure of beams conventionally strengthened using epoxy bonded FRP sheets. The proposed system leads to a ductile failure mode by triggering yielding to occur in a steel anchor system (steel links) rather than by rupture or debonding of the FRP sheets, which is sudden in nature. Four half-scale RC T-beams were tested under four-point loading. Three retrofitted beams were strengthened using one layer of a CFRP sheet. The behaviour of the two beams that were strengthened with the new hybrid FRP sheet and ductile anchor system were compared with the behaviour of the beam conventionally strengthened with epoxy bonded FRP sheet and the control beam. The results showed that the proposed strengthening system effectively increased flexural capacity and ductility of RC beams.

Zhou et al. (2013) developed and investigated a new FRP bonding system, the friction hybrid bonded FRP (FHB-FRP) technique, in which they use new mechanical fasteners. Debonding of FRP plates can be effectively prevented with the use of the FHB-FRP strengthening system. Compared to the use of U-jackets, strengthening beams using the FHB-FRP technique can increase the utilization of the tensile capacity of FRP. RC beams strengthened with the FHB-FRP technique had higher yielding loads and lower yielding load ratios than beams strengthened using the U-jacketing technique. Thus, the FHB technique can provide strengthened beams with a higher service load-carrying capacity.

2.2.4 Near Surface Mounting (NSM) Technique

The NSM reinforcement technique involves making a groove in the surface of the member, roughening and cleaning the groove, filling the groove halfway with a structural adhesive, installing the reinforcing bar or laminate, filling the groove completely with structural adhesive, and leveling the surface. NSM FRP bars and laminates are increasingly being applied as a substitute to externally bonded FRP laminates. The NSM FRP technique may be especially suitable for cases in which the concrete surface is very rough, weak, or requires significant surface preparation.

Lorenzis et al. (2000) and Lorenzis and Nanni (2002) conducted research on NSM techniques with FRP to strengthen various types of beams. The study investigated both flexural and shear strengthening. The study found that end debonding of FRP bars was the dominant failure mode for T-beams and rectangular beams with low reinforcement ratios. Rectangular beams with greater reinforcement ratios failed by concrete crushing. The researchers proposed a system for post-tensioning the NSM system in cases where the ends of the beam were not accessible. Their test results demonstrated that for the flexurally strengthened RC beams, the ultimate strength increased by 44% as compared to the control beam.

Hassan and Rizkalla (2003) investigated the different strengthening systems as well as different types of FRP for the strengthening of large scale prestressed concrete beams. The test results confirmed that the application of NSM FRP was feasible and cost-effective for strengthening concrete bridge members.

Yost et al. (2004) studied the structural performance of retrofitted concrete flexural members using the NSM CFRP method. They reported an increase of 30% and 78% in the yield load and ultimate strength, respectively when compared to the control beam. They also found that the bonds between the CFRP reinforcement, the epoxy and the

adjacent concrete were strong enough to develop the full tensile capacity of the CFRP reinforcement.

El-Hacha et al. (2004) investigated the feasibility of using NSM CFRP to strengthen RC beams. The study found that complete composite action between the NSM strips and the concrete was achieved. The flexural capacity of the strengthened RC beams likewise increased.

El-Hacha and Rizkalla (2004b) also conducted a study on the flexural strengthening of RC beams using the NSM FRP technique. The variables examined were the number of FRP bars or strips, the form of FRP – (either strips or bars) and the type of FRP – (either glass or carbon). They found that using NSM reinforcement with CFRP strips for flexural strengthening resulted in beams that had a higher load carrying capacity than those strengthened with CFRP bars with the same axial stiffness. The results were explained as possibly being due to debonding occurring earlier between the CFRP bar and the epoxy interface.

Rosenboom et al. (2004) strengthened twelve pre-stressed concrete girders with various CFRP systems and tested them under static and fatigue loading. The girders strengthened with NSM CFRP bars and strips achieved a 20% increase in ultimate flexural capacity compared with the control girder when monotonically loaded to failure. The NSM-strengthened girders also performed well under fatigue loading conditions, surviving over two million cycles of increased service loading with little degradation and reduced crack widths.

Barros and Fortes (2005) and Barros et al. (2006) investigated the effectiveness of using CFRP laminates as NSM reinforcement for structural strengthening. The different variables examined were the number of CFRP laminates, different steel reinforcement

ratios, and different depths of the cross-section. An average improvement of 91% on the ultimate load was obtained. The study also found that high ductility at failure of the strengthened RC beams was assured. A serviceability limit state analysis showed an increase in the rigidity of the beam by 28%.

Jung et al. (2006) performed an experimental investigation on the flexural behaviour of RC beams strengthened with NSM CFRP reinforcement. They compared the NSM CFRP strengthened beams to beams strengthened using externally bonded CFRP. The NSM strengthened specimens utilized the CFRP reinforcement more efficiently than the externally strengthened beams.

An analytical evaluation of RC beams strengthened with NSM strips was presented by Kang et al. (2006). The study focused on the relation between the ultimate strength of the beam and the depth of the NSM groove and the spacing between the CFRP strips. They concluded that the minimum spacing between the NSM groove (for multiple CFRP strips) and the edge of the beam should be more than 40 mm to ensure that each CFRP strip behaved independently.

Aidoo et al. (2006) made a full-scale experimental investigation on the repairing of an RC interstate bridge using CFRP material. The three types of strengthening methods investigated were: externally bonded reinforcement, NSM reinforcement, and powder actuated fasteners. All three methods improved the load-carrying capacity of the girders. In particular, the externally bonded CFRP and NSM CFRP behaved better than the powder actuated fasteners. However, the NSM reinforcement showed a significantly higher ductility and this was explained as being due to the better bond characteristics.

Soliman et al. (2010) investigated the behaviour of twenty RC beams flexurally strengthened with NSM FRP bars. Different variables including internal steel ratio, type

and diameter of FRP bars, contact length and groove dimension were investigated in their research. Test results confirmed that the application of NSM FRP bars was effective in improving the flexural capacity of the concrete beams.

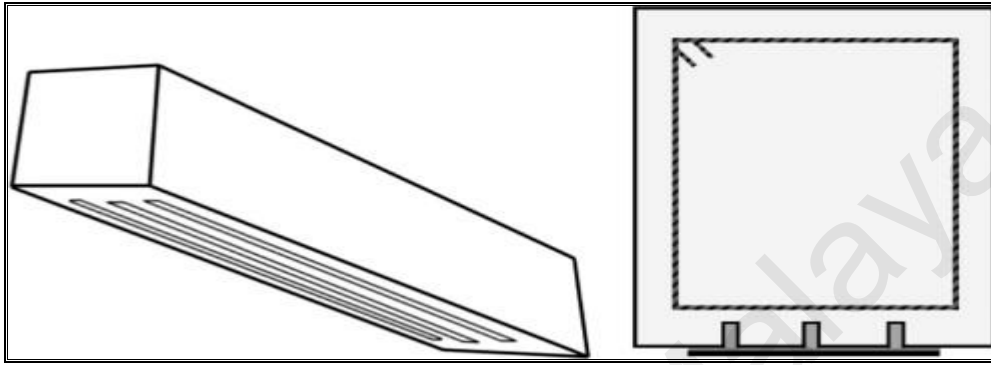


Figure 2.3: EBROG technique (Mostofinejad & Shameli, 2013)

Mostofinejad and Shameli (2013) investigated two new methods named as externally bonded reinforcement on grooves (EBROG) shown in Figure 2.3 and externally bonded reinforcement in grooves (EBRIG) as alternative to conventional externally bonded reinforcement (EBR). Results showed considerable increase in ultimate limits for beams strengthened with EBROG and EBRIG techniques as compared to those strengthened with the EBR method.



**Figure 2.4: Failure modes of beams strengthened with NSM CFRP bars
(Sharaky et al., 2014)**

Sharaky et al. (2014) investigated eight beams to study the behaviour of RC beams strengthened with NSM FRP bars under four-point bending. The effects of material type, epoxy properties, bar size and the number of NSM bars were studied. They found that increasing the number increased the yielding and the maximum loads. However, the small percentage increment in the maximum load was mainly due to the concrete cover separation mode of failure as shown in Figure 2.4.

Bilotta et al. (2015) conducted flexural tests on RC beams strengthened with both NSM and EBR techniques. The results showed that the debonding phenomena for NSM strip strengthened beams are less significant than for EBR plate beams. Moreover, the effect of the loading pattern was analyzed to evaluate the sensitivity of failure modes and loads to different distributions of bending moment and shear along the beam.

2.2.5 Limitations of NSM Technique

Although end debonding failures are less likely in NSM FRP compared to EBR-FRP, they may still notably limit the application of this technology. The debonding failure depends on several factors, like the internal steel reinforcement ratio, the FRP reinforcement ratio, the cross-section and surface condition of the NSM reinforcement, and the strengths of both the epoxy and the concrete. Some researchers (Lorenzis, 2002; Taljsten et al., 2003) extended the NSM FRP over the beam supports to provide anchorage in adjacent members. In spite of this anchorage, debonding can still occur (Lorenzis, 2002). However, Taljsten et al. (2003) reported that one beam failed by FRP rupture where the reinforcement was extended over the supports, as opposed to the failure by debonding observed in an identical beam where the NSM reinforcement did not extend over the supports. Blaschko and Zilch (1999) reported the results of tests on two beams strengthened with NSM FRP. The first beam failed by end debonding from the cut-off section but the second beam with a steel U-jacket bonded to the cut-off section, failed by FRP rupture.

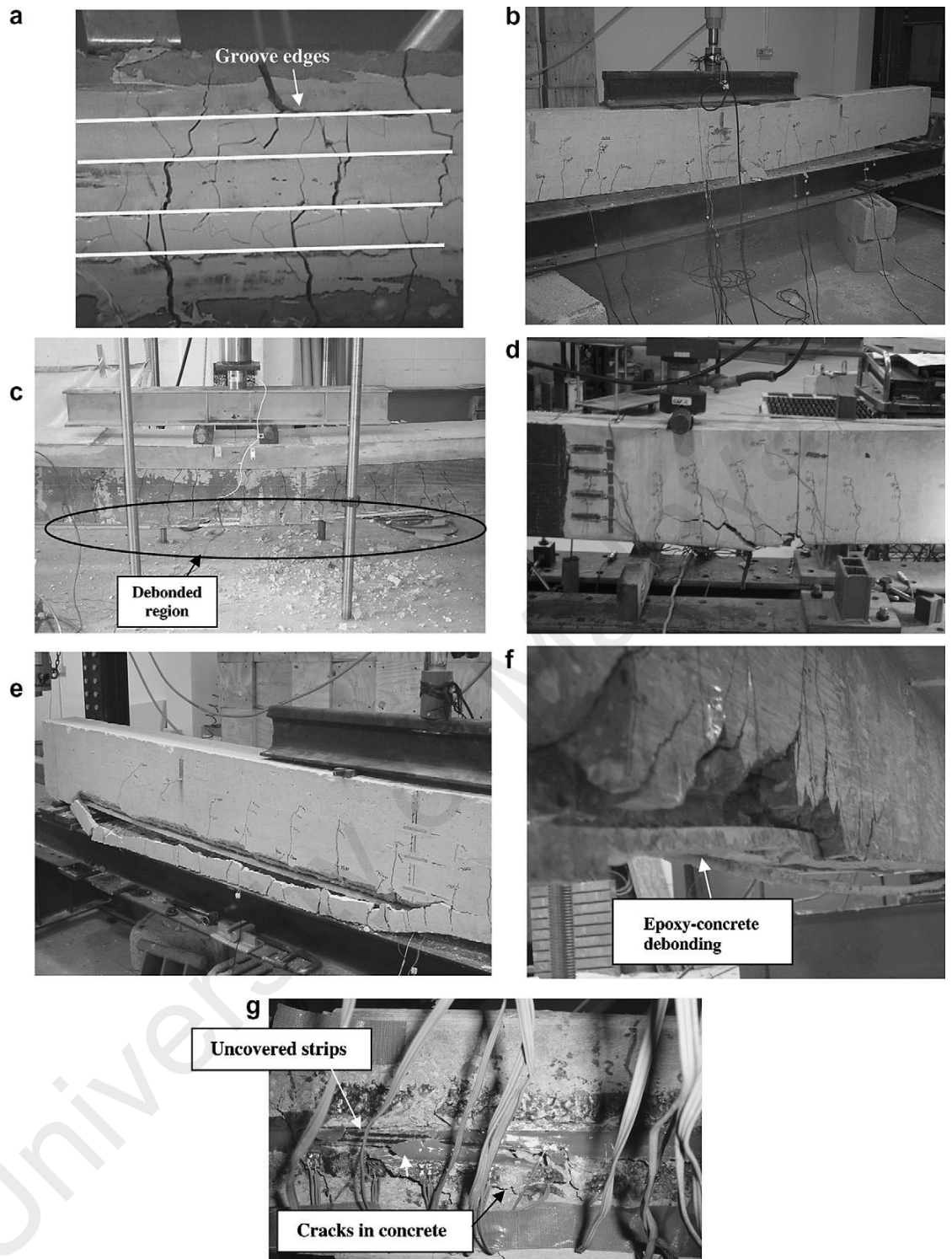


Figure 2.5: Failure mode of the NSM technique (Lorenzis & Teng, 2007)

De Lorenzis and Teng (2007) observed seven debonding failure modes for RC beams flexurally-strengthened with NSM bars and strips. These seven modes are shown in

Figure 2.5 and described (the alphabet in parenthesis indicate the type of failure in Figure 2.5) below:

- i. Separation at the bar-epoxy interface (a),
- ii. Concrete cover separation between two cracks in the maximum moment region (b),
- iii. Concrete cover separation over a large length of the beam (c),
- iv. Concrete cover separation starting from a cutoff section (d),
- v. Concrete cover separation along the edge (e),
- vi. Secondary failure of bond between epoxy and concrete (f), and
- vii. Secondary splitting of the epoxy (g).

The mechanics of end debonding in beams strengthened with the NSM technique is still not fully understood. Descriptions of modes of failure in available literature are often not sufficiently detailed enough to provide an understanding of the progression of the failure. Based on the available experimental data in research works, the probable failure modes of beams strengthened with NSM FRP reinforcement are shown in Figure 2.5. The interaction between the primary failure modes and the secondary failure modes are not still clear and require further investigation.

De Lorenzis and Teng (2007) have pointed out that a large number of factors can affect the flexural behaviour of RC beams with NSM FRP, and thus further experimental and theoretical study is required, particularly to clarify the debonding failure mechanisms in the NSM reinforced beam. Also, the interaction between concrete cover separation and other modes of failure that occur to the NSM FRP concrete interface, such as fracture at the epoxy and concrete interface and separation of the epoxy cover, needs further research. Additionally, investigating the behaviour of pre-damaged beams strengthened

with NSM FRP would be significant especially in the practical field, as cracking and damage to the concrete cover may have a significant influence on the debonding failure process. Lorenzis and Teng (2007) have also recommended that the relationship between bond failure mechanisms and debonding failure mechanisms in flexurally-strengthened beams can be clarified through detailed experimentation and theoretical modeling. In such an investigation, the interaction between flexural or flexural-shear cracking and bond stresses must be clarified for the development of numerical and analytical models to predict debonding failure.

2.2.6 Fatigue Performance of EBR and NSM Technique

2.2.6.1 Strengthened with Steel

A limited number of experimental investigations into the fatigue performance of RC beams rehabilitated with externally bonded steel plates have been reported. Iyer et al. (1989) found that steel plating was not greatly affected by cyclic loading. It can be assumed that the steel plates were able to forestall fatigue failure in the internal reinforcing steel by attracting a portion of the internal tensile stress in the beam and, thus, reduce the stress range applied to the internal reinforcement.

Byung et al. (2003) investigated the static and fatigue behaviour of RC beams strengthened with steel plates. A comprehensive test program was set up and series of strengthened beams were tested. Their study found that the strengthened beams exhibited much higher fatigue resistance than the unstrengthened beams at the same fatigue load level. The increase in deflections of the strengthened beams according to the number of load cycles was much smaller than that of the unstrengthened beams. After applying 43106 cycles under 60% and 70% fatigue load levels, the beams were tested up to failure. The ultimate fatigue loads were found to be similar to the static failure load. This indicates

that a fatigue load below 70% of the static failure load does not decrease the ultimate strength of strengthened beams.

2.2.6.2 Strengthened with FRP

Kaiser (1989) conducted fatigue tests at EMPA on RC beams strengthened with a glass and carbon fiber hybrid composite. The cross-section of the RC beam was 300 mm wide and 250 mm deep, and the span was 2000 mm. The conventional reinforcement consisted of two 8 mm rebars in the tension zone. The composite sheet had a 0.3 mm by 200 mm cross-section and was bonded to the tensile face of the beam. The beam was subjected to two-point loading and cycled from 1 kN to 19 kN (0.2 kips to 4.3 kips) at a frequency of 4 Hz, corresponding to a stress range in the reinforcing bars of 386 N/mm (56 ksi). The first fatigue damage to the rebars occurred after 480,000 cycles. The first damage in the composite appeared after 750,000 cycles in the form of fracturing of individual fibers in the strips. The relatively sharp concrete at the edges of cracks rubbed against the strips at every cycle, and the composite finally failed after 805,000 cycles. These results clearly indicate that FRP laminates can sustain significant loading after failure of the steel reinforcement.

Fatigue tests by Shijie and Ruixian (1993) showed that the fatigue lives of GFRP plated members could be up to three times longer than the life of an unstrengthened RC control specimen. The fatigue strength could be increased from 15% to 30% and mid-span deflection could be reduced to 40%. The bending capabilities of the reinforced beam diminished with the increasing number of cycles. For example, the static loading test for one beam showed that after 2×10^{10} cycles the limiting bending moment of the mid-span location diminished from 244.3 kN-m to 198.4 kN-m. Both the post-cyclic static strength and stiffness diminished as number of cycles increased, but by a smaller magnitude than for the unstrengthened beam.

Meier (1995) performed further tests at EMPA on beams with T-shaped cross-sections under more realistic loading conditions. The cross-section was 900 mm wide and 500 mm deep, and the span was 6000 mm. The beams were tested under cyclic loading ranging from 126 kN to 283 kN, representing 15% to 35% of the static ultimate capacity of the beams. The corresponding stress range in the rebars was 131 N/mm. Crack development was noted after 2 million cycles. After 10.7 million cycles at room temperature, the test temperature was increased to 40°C and the relative humidity to 95%. The first failure in the rebars occurred at 12 million cycles. After 14.09 million cycles, the second bar failed and the CFRP strip sheared from the concrete surface. A third fatigue test similar to the one described above was conducted at EMPA with pre-tensioned strips, and 30 million load cycles were performed without any damage.

Inoue (1996) investigated the strengthening of RC beams by adhesion of CFRP plates. The beams were tested under static and fatigue loading for strength and deformation characteristics. The study compared RC beams strengthened with CFRP plates bonded to the underside of the beam with resin adhesive and RC beams where the CFRP plates were fixed with anchor bolts as well as resin adhesive. The results of the study indicate that the appropriate fatigue life of the CFRP beams can be estimated from the reinforcement stress, which in turn can be determined on the basis of linear elastic theory by assuming a crack section and the S-N equation of the reinforcement in JSCE's specifications. The installation of an anchor bolt increases the fatigue life of strengthened beams under high loads but it exerts little effect on the static strength.

Barnes and Mays (1999) investigated the fatigue performance of RC beams strengthened using CFRP plates for design applications. Five RC beam specimens, 2300 mm long, 130 mm wide, and 230 mm in depth, were tested. Two beams were unplated

control specimens, and three were plated beams. The strengthening plates consisted of 68% volume fraction high-strength unidirectional carbon fibers (Toray T300) embedded in a vinyl ester resin, and bonded using a two-part cold-curing epoxy adhesive (Sikadur 31 PBA). Each specimen was subjected to two-point loading at a frequency of 1 Hz. Three loading options were tested as follows:

- i. Apply the same load to both the plated and unplated beams,
- ii. Apply loads to give the same stress range in the rebars of both beams, and
- iii. Apply the same percentage of ultimate static capacity to each specimen.

The study found that the plated beams demonstrated better stress endurance performance. However, the authors concluded that a criterion for design guidance would be to expect the same fatigue life for plated and unplated beams, with similar ranges of stress in the reinforcing steel (Asplund, 1949).

Shahawy and Beitelman (1999) performed fatigue tests on severely cracked RC beams post-strengthened using different arrangements of CFRP biaxial fabrics applied on the bottom face or fully wrapped on the stem. The objective of the tests was to study the effect of strengthening on the extension of fatigue life of severely damaged members. They tested six beams with T-shaped cross-sections that were 584 mm wide, 445 mm high, and with a 5790 mm span. The beams were loaded at two points. The loads represented 25% to 50% of the ultimate capacity, with the stress range in the rebars being about 103.4 MPa. At this level, the authors expected the steel to have a fatigue life of approximately one million cycles. The unstrengthened control specimen failed after 295,000 cycles. One unstrengthened specimen was previously subjected to fatigue loading for 150,000 cycles and then strengthened using two layers of CFRP biaxial fabric bonded on the full stem of the beam. This specimen failed after two million cycles following rupture of the fabric, after fatigue failure of the internal steel. After

strengthening, the specimen demonstrated a slight increase in stiffness up to the time just before failure. Specimens wrapped with three layers of fabric survived up to 3 million cycles. The researchers concluded that the fatigue life of strengthened specimens was prolonged and that severely damaged members could be effectively rehabilitated using externally bonded CFRP materials.

Benouaich (2000) tested six specimens, strengthened using different configurations of CFRP flexible sheets and pultruded plates. The beams were subjected to fatigue loading under various stress ranges representative of service-load conditions and potential overloading. Test results showed no evidence of damage propagation at the concrete-composite interface when beams were subjected to service-load cycling. Monotonic tests demonstrated no influence of the fatigue loading on the ultimate static capacity. However, post-cyclic ultimate deformations and structural ductility were reduced after cyclic loading. Fatigue performance under high stress ranges appeared to be governed by debonding at the concrete-adhesive interface.

Papakonstantinou et al. (2001) examined the effects of GFRP composite rehabilitation systems on the fatigue performance of RC beams. The results of their study indicated that the fatigue life of RC beams, for a given geometry and subjected to similar cyclic loading, can be significantly extended through the use of externally bonded GFRP composite sheets.

Deng (2002) investigated the static and fatigue behaviour of RC beams strengthened with CFRP sheets bonded with organic and inorganic matrices. The study examined the crack behaviour, failure mode, strength improvement/behaviour, stiffness behaviour, strain behaviour and fatigue life behaviour of the strengthened beams. The results showed that the RC beams bonded with organic matrices and those bonded with inorganic

matrices behaved differently. They study also found that the fatigue lives of RC beams strengthened with CFRP exhibited Weibull probability distribution.

Aidoo et al. (2004) examined the flexural fatigue performance of RC bridge girders strengthened with one-dimensional FRP composites. The study used eight RC T-beams, 508 mm deep and 5.6 m long, with and without bonded FRP reinforcement on their tensile surfaces. The beams were tested with concentrated loads at mid-span under constant amplitude cyclic loading. The results of the study indicated that the fatigue behaviour of beams strengthened with one-dimensional FRP composites is controlled by the fatigue behaviour of the reinforcing steel and that the fatigue life of RC beams can be increased by the application of FRP composites, which relieve some of the stress carried by the steel.

Heffernan and Erki (2004) investigated the fatigue behaviour of RC beams post-strengthened with CFRP laminates. They tested twenty 3 m and six 5 m beams loaded monotonically and cyclically to failure, comparing beams with and without CFRP strengthening. They also examined the effect on fatigue life on increasing the amount of CFRP used to strengthen beams. The study found that the use of CFRP sheets lowered stresses in the tensile steel. Thus, the fatigue life of all the beams, without and with CFRP strengthening, was directly related to the fatigue characteristics of the tensile reinforcing steel and its stress history due to the applied loading. Concrete softening due to repeated loads caused an increase in the stresses in the tensile steel. The CFRP strengthened beams had less severe increases in steel stresses than the beams without CFRP sheets. There was no significant degradation due to cyclic loading in the CFRP sheets or the CFRP to concrete interface. Thus, the basic assumptions for monotonic behaviour remained valid for the beams loaded cyclically.

Gussenhoven and Brena (2005) tested thirteen small-scale beams strengthened using CFRP composites. The beams were tested under repeated loads to investigate their fatigue behaviour. Test results indicated that peak-stress applied to the reinforcing steel in combination with composite laminate configuration were the main parameters that affected the controlling failure mode.

Brena et al. (2005) conducted an experimental program that consisted of fatigue testing of ten RC beams strengthened using two different types of externally bonded CFRP composites. The results indicated that the bond between the composite laminates and the surface of the concrete can degrade at load amplitudes corresponding to extreme load conditions for a bridge. These results showed that an upper limit on stresses generated along the composite-concrete interface might have to be set during the design stage to avoid premature debonding after a limited number of load cycles.

Ekenel et al. (2006) examined the flexural strength of RC beams using two FRP strengthening systems. Two of the RC beams were maintained as unstrengthened control specimens. Three beams were strengthened using CFRP fabrics. The two remaining beams were strengthened with FRP pre-cured laminates. One of the beams strengthened with CFRP fabric also used glass fiber anchor spikes. Of the two beams strengthened with FRP pre-cured laminates, one was bonded using epoxy adhesive and the other one was attached with mechanical fasteners. Five beams were tested under fatigue loading for two million cycles and all five beams survived. The results showed that use of anchor spikes in fabric strengthening increases ultimate strength, and mechanical fasteners can be an alternative to epoxy bonding in pre-cured laminate systems.

Toutanji et al. (2006) studied the fatigue performance of concrete beams strengthened with CFRP sheets bonded with an inorganic matrix. Large scale RC beams were strengthened with three layers of CFRP sheets and tested under fatigue loading. The

relationships between fatigue strength, crack width, and number of cycles were studied and analyzed. The results showed that both the load capacity of the RC beams and the number of cycles the RC beams could withstand were significantly increased with CFRP sheets.

Ferrier et al. (2011) focused on the damage behaviour of FRP strengthened RC structures subjected to fatigue loading in their study. They developed a model calibrated using data from existing literature and from experimental investigations specifically carried out for the study. The model was able to correctly estimate the fatigue behaviour of FRP strengthened beams, as deflection and strain in the different materials could be calculated with a sufficient accuracy.

Al-Rousan and Issa (2011) carried out an experimental and analytical study to study the performance of nine RC beams externally strengthened with various configurations of CFRP sheets. The beams were subjected to static and accelerated fatigue testing. The beams were tested for various stress ranges. After validating a non-linear finite element analysis (NLFEA) with experimental test results, the analysis was extended to provide a better understanding of the effect of: fatigue stress ranges, the number of CFRP layers, and the CFRP to concrete contact area on the performance of RC beams. Stress ranges were found to have a significant effect on the permanent deflection at mid-span especially for higher stress ranges. Cyclic fatigue loading produced a time-dependent redistribution of the stresses, which led to a sudden drop in concrete stresses and a mild increase in steel and CFRP sheet stresses as fatigue life was exhausted.

Regarding the NSM technique, Quattlebaum et al. (2005) evaluated the static and fatigue performance of reinforced concrete beams retrofitted with conventional adhesive applied (CAA), NSM, and powder actuated fastener-applied (PAF) FRP retrofit systems.

The results of this study indicate that the CAA method is outperformed by the other methods under cyclic conditions.

Badawi (2007) studied RC beams with non pre-stressed and pre-stressed CFRP bars to increase the static and fatigue strength of the beams. The test results showed that RC beams strengthened with NSM CFRP bars increased both the static capacity and the fatigue strength.

Yost et al. (2007) studied how fatigue loading for 2,000,000 cycles affected the static performance and stiffness of simply supported steel reinforced beams with NSM FRP bars and strips. Test results showed that all beams strengthened with CFRP plates and CFRP bars survived the 2,000,000 cycles with no significant loss in bond or force transfer. Thus, composite action between concrete and the NSM CFRP appears to be unaffected by fatigue loading

Oudah and El-Hacha (2012) studied the fatigue behaviour of RC beams strengthened using pre-stressed NSM-FRP strips. Experimental test results show that the deflection increase at the end of fatigue loading was almost similar for all beams, which indicates that damage accumulation is not dependent from the pre-stress level.

Wahab et al. (2012) tested ten concrete beams strengthened with NSM pre-stressed FRP bars under different fatigue load levels. The test variables included the type of CFRP rod (spirally wound or sand-coated) and the fatigue load level. Test results showed that the sand-coated rods exhibited a better bond fatigue performance than the spirally wound rods, whereas at a given load level, the beams strengthened with sand-coated rods had longer fatigue lives than the beams strengthened with spirally wound rods. Also, for a given number of cycles, the beams strengthened with prestressed CFRP rods failed in

bond at a lower applied load range than the beams strengthened with a non pre-stressed CFRP rod.

2.3 Numerical Modelling

A number of research works on numerical analysis have been done to predict the failure mechanism and interfacial stress of strengthened RC beams. Adhikary and Mutsuyoshi (2002), in their modelling of RC beams, took into account the slip effect between the concrete and the strengthening steel plates, and the non-linear behaviour of concrete, reinforcing bars and steel plates. Wolanski (2004) studied flexural behaviour of reinforced and pre-stressed concrete beams using finite element analysis.

Kachlakev and Miller (2001) studied “Finite Element Modeling of RC Structures Strengthened with FRP Laminates” with ANSYS and the objectives of this modeling was to investigate the structural performance of Horsetail Creek Bridge (which is a historic bridge, built in 1914).

Zhang and Teng (2010) predicted the interfacial stresses using the finite element method. Five different finite element modeling approaches based on different assumptions for the deformations of the three components of such a plated beam (i.e. beam, adhesive layer and plate) are described. These results provide a useful insight into the risk of debonding in such plated panels.

Al-Rousan and Issa (2011) validated a non-linear finite element analysis (NLFEA) with experimental test results, the analysis was extended to provide a better understanding of the effect of: fatigue stress ranges, the number of CFRP layers, and the CFRP to concrete contact area on the performance of RC beams. Stress ranges were found to have

a significant effect on the permanent deflection at mid-span especially for higher stress ranges.

Radfar et al. (2012) carried out a non-linear finite element analysis using the commercial program ABAQUS to predict ultimate loading capacity and the failure mode of RC beams in a four-point bending setup. A series of 4 RC beams strengthened with FRP sheets at the bottom were tested to failure under a four-point bending load. By comparing numerical results with experimental ones, the proposed finite element model has been validated and can be used for further prediction of this type of failure.

Hawileh (2012) develop a detailed 3D nonlinear FEM that can accurately predicts the load-carrying capacity and response of RC beams strengthened with NSM FRP rods subjected to four-point bending loading using the finite element code, ANSYS. The developed FE models have been validated by comparing the predicted failure mode and mid-span deflection with that of the measured experimental data obtained by Al-Mahmoud et al. (2009). In addition, the validated FEM are used to study the effect of NSM bar material types and CFRP rod diameter on the global response of the strengthened RC beams. The results of this study showed the practicality and validity of the finite element method in modeling RC beams strengthened in flexure using NSM CFRP bar reinforcement.

Omran and El-Hacha (2012) developed a comprehensive 3D nonlinear Finite Element (FE) analysis of Reinforced Concrete (RC) beams strengthened with prestressed NSM-CFRP strips. Debonding effect at the epoxy-concrete interface was considered in the model by identification of fracture energies of the interfaces and appropriate bilinear shear stress-slip and tension stress-gap models. Prestressing was applied to the CFRP strips by adopting the equivalent temperature method.

Zhang and Teng (2014) presented a novel finite element (FE) approach for predicting end cover separation failures in RC beams strengthened in flexure with either externally bonded or near-surface mounted FRP reinforcement. In the proposed FE approach, careful consideration is given to the constitutive modelling of concrete and interfaces. Furthermore, the critical debonding plane at the level of steel tension bars is given special attention; the radial stresses exerted by the steel tension bars onto the surrounding concrete are identified to be an important factor for the first time ever and are properly included in the FE approach. Their proposed FE approach is shown to provide accurate predictions of test results, including load–deflection curves, failure loads and crack patterns.

Bencardino and Spadea (2014) carried out numerical analysis with reference to external strengthened RC beams with a steel reinforced grout system. Through an appropriate numerical investigation, based on a suitable three-dimensional model, compared with the results of an experimental investigation, a parametric analysis was also developed.

Chen et al. (2015) examined the effectiveness of using a dynamic analysis approach in such FE simulations, in which debonding failure is treated as a dynamic problem and solved using an appropriate time integration method. Numerical results are presented to show that an appropriate dynamic approach effectively overcomes the convergence problem and provides accurate predictions of test results.

Zidani et al. (2015) presented an advanced finite element model using the general purpose FE software ANSYS to simulate the flexural behaviour of initially damaged concrete beams repaired with FRP plates. The model is capable to simulate the full history stages; where the beam is initially loaded to introduce damage, then, after bonding the FRP plates, the beam is reloaded up to failure. The finite element model has been

validated using experimental data in the literature and used to study the effect of concrete-FRP models, interfacial shear stress distribution, crack pattern, and failure mechanism. In addition, the effect of plate thickness and the gained load capacity in terms of damage degree have been also investigated. The predicted results indicated that the load capacity of all repaired beams is higher than that of the control beam for any damage degree. Moreover, when repairing highly damaged beams, the most likely expected mode of failure is plate debonding for any FRP plate thickness

2.4 Optimization in Structural Design

A wide variety of optimization algorithms have been created and studied throughout the last centuries. The first optimization techniques, like the Gauss steepest descent developed in the 18th century, were based on pure mathematics. More complex techniques have been later developed, and the first modern technique, Dantzig's linear programming, appeared in the 1940's (Dantzig, 1949), to be used by the US military. Since then, a rising interest in optimization has led to the development of dozens of different algorithms which can be used in a wide range of applications. Schmit (1960) recognized the potential for applying optimization techniques in structural design in 1960 (Schmit, 1960). He first used non-linear programming techniques to design elastic structures.

2.4.1 Gradient-Based Approach

Gradient-based approaches directly use mathematical tools to find optimal solutions. The gradient-based algorithms are the Sequential Quadratic Programming (SQP) (Fletcher, 1987) and the Hookes-Jeeves algorithms (Hooke & Jeeves, 1961). The working principle is that from an initial value, the local gradient information is used to establish a direction of search at each iteration, until an optimum is reached. These kinds of algorithms only work with objective functions which are twice differentiable or that can

be approximated by terminated first-order or second-order Taylor series expansion around the initial guessed value. This approach can be used for the optimization heating systems and has more recently been used for optimization of cooling plants' control scheme. While this type of approach has been used in past studies, it suffers from two major limitations discussed below.

The first limitation of gradient-based methods is that they are prone to local extrema. Depending on the starting value, they are likely to get trapped in the nearest local optimal value, missing the actual optimum. Taking several different initial values could eventually be seen as a solution to overcome this problem, but it would provide little guarantee, and may become a purely random search. The second major limitation of gradient-based approaches is that, as stated above, they only work with differentiable or at least relatively smooth functions. As far as building phenomena are concerned, functions are very often non-linear problems. Moreover, both discrete and continuous variables are involved, which may lead to discontinuous outputs.

2.4.2 Gradient-Free Approach

The second and more modern school of optimization techniques, referred to as 'gradient-free', relies on stochastic techniques rather than derivatives to determine the search direction. This allows the exploration of the whole search space, focusing only on regions of interest. Unlike the techniques previously described, gradient-free approaches can easily avoid local extrema and have proven their efficiency on optimization problems where classical methods fail (Goldberg, 1989). Several different algorithms from this school of optimization have been developed. A review of the predominant ones used for building applications is detailed by Wetter and Wright (2004). Of all gradient-based techniques, population-based techniques and more precisely genetic algorithms are

predominant, and have proven their efficiencies in hundreds of cases; genetic algorithms will therefore be discussed in more details.

2.4.3 Genetic Algorithms

A genetic algorithm (GA) is an optimization technique developed by Holland (1975) in the 1970s and is based on Darwin's theory of evolution. GA's principle is simple, although unusual. In a nutshell, each solution is referred to as an individual, which may further produce children, and on which an evolutionary mechanism is applied. GA has been used in a wide range of studies, from medicine (Lahanas et al., 2003) to transportation engineering (Syarif & Gen, 2003).

Regarding the efficiency, GA is recognized to enable very detailed optimization and is capable of finding optimal or near optimal solutions using less computation time than other algorithms (Kobayashi et al., 1998; Wetter & Wright, 2003). Another advantage of GA is that it can be used for true multi-objective optimization. GA has been able to successfully handle multiple objectives, where other evolutionary algorithms such as particle swarm optimization have failed (Srinivasan & Seow, 2005). One last quality of GA is that it can perform very well when associated with response surface approximation methods (Chow et al., 2002).

A main drawback of GA is the high number of calls to evaluation function. In building applications, these evaluations are generally estimated by an external simulation program or other simulation software. If accurate results are required, each evaluation can be time consuming, and thus the complete computational process becomes extremely unattractive. For instance, for the two-objective optimization of building floor shape, Wang et al. (2006) used an evaluation tool where each evaluation took 24 seconds (CPU-time). In that case, the total optimization time, which was mainly due to evaluations, was 68 hours. Based on a simple rule of three, one can expect that, using simulation software

where each evaluation would take thirty minutes, a similar optimization would result in a total optimization time of more than 6 months.

Although the GA method has received much attention in recent years with respect to discrete optimization, they have a few areas with unanswered questions. For instance, will they always produce global optima and can they be implemented and tuned to solve discrete structural optimization problems? Using a practical structural system and a GA based method efficiently solved a discrete variable problem with constraints. Rajeev and Krishnamoorthy (1992) efficiently solved a discrete variable problem with constraints. They showed that even though the GA is not well suited for constrained problems a penalty-based transformation can be implemented. They also showed that the GA method is suitable for a parallel computing environment. Near optimal solutions in reasonable computing times were obtained on large design space layout and sizing problems of steel roof trusses using a GA by Koumoussis and Georgiou (1994). They reported that no clear rules exist for tuning of the GA parameters and the estimate of the parameters is delicate. Using the uniform building code as constraints. Camp et al. (1998) developed a GA based method for optimizing two-dimensional steel frame structures. The method was tested on 30 designs. The method always produced structures satisfying the code standards while minimizing the weight but the solution was not guaranteed to be global. Lu and Kota (2005) successfully applied a GA method to a mixed discrete topology and continuous sizing problem.

Despite the foregoing success, evolutionary algorithm (EAs) by themselves are unconstrained optimization methods and suffer from lack of generality when applied to specific engineering problems. That is, the particular encoding from one problem to another is necessarily different, the formulation of problem constraints and objective function is specific and the EA control parameters often must be tuned to the specific

problem group and sometimes even to the specific problem instance. Alternative methods of representing problem requirements more generally deserves further investigation.

A common conclusion in the literature with respect to GA is that it requires considerable user insight and adjustment to the parameters to get reasonable results (Thanedar & Vanderplaats, 1995).

2.4.4 Optimization of RC Structures

The optimal design for beams was first proposed by Galilei (1950), even though his calculation was wrong. Haug and Kirmser (1967) were the first to try to use a digital computer as a tool for the optimal design of this structure element. They reduced the non-linear optimal design problem to Langrange problem in the calculus of variations. Their model includes restrictions and tries to minimize the weight of the beam in several different situations. Venkayya (1971) developed a method to design a structure subjected to static loading based on an energy criteria and a search procedure. He argued that his method can efficiently handle a design with multiple load conditions and stress constraints on size elements. His method has been successfully applied to the design of trusses, frames and beams. Balaguru (1980) designed an algorithm to calculate the optimum dimensions and the amount of reinforcements for a doubly reinforced rectangular beam. Osyczka (1984) applied multi-objective optimization techniques to a beam design problem. Prakash et al. (1988) proposed a model for the optimal design of RC sections in which the cost of steel, concrete and shuttering were included. Chakrabarty's model has some similarities to Prokash's model (Chakrabarty, 1992a; Chakrabarty, 1992b).

Many researchers have applied different optimization techniques to the design of RC structures. The crushing strength of concrete was considered as a design variable in addition to cross-sectional dimensions and steel ratios, for the cost optimization of simply

supported and multi-span beams with rectangular and T-shaped cross-sections (Kanagasundaram & Karihaloo, 1991a, 1991b). They used sequential linear programming and sequential convex programming techniques, formulating the cost optimization as a non-linear programming problem. Adamu et al. (1994) developed a method based on continuum-type optimality criteria, while Han et al. (1996) used discretized continuum-type optimality criteria. Lepš and Šejnoha (2003) used GAs to optimize RC beams while Camp et al. (2003) used GAs to optimize structures.

Leroy (1974) derived an equation to find the optimum ratio of steel to concrete area for a singly reinforced beam based on moment constraints alone. Chou (1977) uses the Lagrange multiplier method to find the minimum cost design of a singly reinforced T-beam using the ACI code. Kirsch (1983) presented a simplified three level iterative procedure for cost optimization of multi-span continuous RC beams with rectangular cross-sections. He optimized the amount of reinforcement at the first level, the concrete dimensions at the second level, and the design moments at the third level. Lakshmanan and Parameswaran (1985) derived a formula for the direct determination of span to effective depth ratios which can avoid the trial and error approach necessary for the flexural design of RC sections as per the Indian standard. Coello et al. (1997) presented a cost optimum design of singly reinforced rectangular beams using GA. They considered the sectional dimensions and the area of tensile reinforcement as variables in their optimum design model. Koumousis and Arsenis (1998) presented the application of GAs for the optimum detailed design of RC members on the basis of multi-criterion objectives that represent a compromise between a minimum weight design, maximum uniformity and the minimum number of bars for a group of members.

Some studies on structural optimization deal with minimization of the weight of structures (Haug & Kirmser, 1967; Karihaloo, 1979; Lakshmanan & Parameswaran,

1985; Venkayya, 1971). However, most researchers have worked on the cost optimization of structures (Al-Salloum & Husainsiddiqi, 1994; Ceranic & Fryer, 2000; Chakrabarty, 1992a; Chakrabarty, 1992b; Friel, 1974a; Kanagasundaram & Karihaloo, 1991a, 1991b; Perumalsamy & Balaguru, 1980; Prakash et al., 1988). Although the weight of a structure may be proportional to its cost, minimization of cost should be the actual objective in economically designing RC structural elements.

Most researchers have used the ultimate load method for the design of beams (Chakrabarty, 1992a; Chakrabarty, 1992b; Friel, 1974a; Karihaloo, 1979; Perumalsamy & Balaguru, 1980), whereas a few have used the limit state method (Adamu et al., 1994; Ceranic & Fryer, 2000; Prakash et al., 1988). While the ultimate load method provides a realistic assessment of safety, it does not guarantee satisfactory serviceability at service loads. On the other hand, the limit state method aims for a comprehensive and rational solution to the design problem by considering safety at ultimate loads and serviceability at working loads and hence is a better design method.

Kwak and Kim (2008) recently developed a simplified and effective algorithm for the practical application of optimum design techniques on RC members. Instead of utilizing a more sophisticated optimization model that requires many design variables and complicated descriptive functions, the proposed algorithm used a more effective direct search method to find the optimum member sections from a predetermined section database. After constructing a database of predetermined RC sections, which were arranged in the order of increasing resisting capacities, the relationship between the section identification numbers and the resisting capacities of the sections was established by regression and was used to obtain an initial solution (section) that satisfies the imposed design constraints. Assuming that an optimum section exists near the section initially selected by the regression formula, a direct search is conducted to determine the discrete

optimum solution. The optimization of the entire structure is accomplished through the optimization of individual members.

During the past two decades, considerable progress has been made in the area of optimizing the design of RC structures using various methods. Most researchers have worked on the cost optimization of structures, although a few studies deal with the optimization of weight. Moreover, most of the studies only consider steel as an internal reinforcement embedded in the concrete. Limited or no studies have been found on the optimization of FRP strengthened RC beams. Compared to steel, FRP reinforcement generally possesses a lower modulus of elasticity, which leads to higher reinforcement strains, wider cracks and larger deflections. Thus, the behaviour of FRP strengthened RC structures will require the use of the serviceability limit state design method.

2.4.5 Optimization of FRP Strengthened RC Beams

No significant works could be found on the optimization of structural strengthening systems except a study by Perera and Varona (2009). They used GAs for the discrete optimization of the design of FRP strengthened RC structures subjected to the limitations and recommendations specified by the European design guidelines (FIB, 2001). A description is given of the GA approach that they take to optimize the FRP external reinforcement used for the flexural and shear strengthening of RC beams. The starting point for natural selection is a database of FRP laminates and sheets of different sizes and dimensions, which contains the usual specifications supplied by manufacturers. FRP laminates were used for flexural strengthening while FRP sheets were applied for shear strengthening. Each candidate in the database was assigned an identification number, so that discrete optimization could be performed using GAs. Flexural plate length as well as the number of sheets were dealt with through discrete values. The objective function was the cost of strengthening. This can be estimated as the cost of the CFRP plates or sheets

plus the cost of the adhesive. The former depends on the volume of composite used for flexural and shear reinforcement while the latter depends on the surface or the interface to which the adhesive must be applied. Penalty functions were also included for the restrictions found in the design guidelines.

2.5 Identification of Research Gaps and Significance of this Study

Externally bonded reinforcement is a commonly practiced method to strengthen structures. Many studies have been conducted to investigate the effect of different parameters on the external bonding technique, using steel plates or FRP composites. As extensive research on this strengthening method has already taken place, a number of design guidelines on this method have been published in different countries. On the other hand, NSM reinforcement is a relatively new, though promising technique in the field of structural strengthening. A number of experimental research works have been conducted on the NSM strengthening method. Moreover, the ACI has already updated their design guidelines to include the NSM technique. However, no significant works could be found on the HSM, which combines the above two techniques. The HSM may eliminate some of the limitations of the existing two strengthening methods, thus experimental investigation needs to be conducted on this method.

A number of codes or design guidelines have been published in different countries for the design of RC structures strengthened with steel and FRP. However, the conventional practice in the design of RC beams and their structural strengthening systems involves performing preliminary elastic analyses based on an assumed cross-section and then checking the member for its adequacy against strength, serviceability and other requirements as imposed by the design codes. If the requirements are not satisfied, then the sectional dimensions are modified repeatedly until it satisfies the requirements of the code. This repetitive process is carried out without considering the relative costs of the

component materials of the structure. Therefore, to optimize the total cost of the structure, a guideline is required to determine the minimum amount of materials needed to design a functional structure.

The main contributions of the current study are thus: the investigation of the effectiveness and feasibility of using the HSM for the flexural strengthening of RC members under monotonic and cyclic loading, the characterization of the experimental behaviour of RC beams strengthened with NSM steel bars, the presentation of a more economical strengthening solution, and the development of an optimum design method for structural strengthening systems. The research is comprised of experimental, numerical and analytical programs to achieve the stated objectives of the current study.

2.6 Research Questions

Based on the research gaps mentioned in the previous section of this chapter, it is urgently important to achieve the answers or the solutions to the following research questions:

- i. Is the newly proposed HSM feasible or not?
- ii. In comparison to the existing two methods, does the HSM perform better or not?
- iii. Which parameters mostly affect the efficiency of the HSM?
- iv. Does strengthening with NSM steel bars and cement mortar give a more economical strengthening solution without compromising technical performance?
- v. Does the use of optimization approaches make the strengthening design process more efficient?

CHAPTER 3: METHODOLOGY

3.1 Introduction

The research methodology applied in this study has been divided into three parts, namely experimental investigation, numerical modelling and mathematical optimization. However, they all share the same goal. The goal of the current research work is to make a more efficient structural strengthening system. Exploring the use of steel bars and cement mortar in NSM reduces the cost and hybridizing the existing two methods improves the technical performance. Therefore, the efficiency of the structural strengthening system will be increased. The optimization method will also reduce the cost and increase the efficiency of the system. Section 3.2 describes the experimental setup including the materials used, the design, preparation and strengthening of the specimens, instrumentation of the specimens and the test procedure to examine effectiveness of the use of steel bars and cement mortar in NSM and the proposed HSM. The development of a semi-numerical model is discussed in Section 3.3. Section 3.4 demonstrates finite element modelling and Section 3.5 describes the application of mathematical optimization techniques.

3.2 Experimental Programme

An experimental programme was developed to verify the proposed HSM and the effectiveness of the use of steel bars with cement mortar in NSM. Experimental data on loading, deflection, strain and failure mode were obtained. The experimental program consisted of thirty-three RC beams. The beams were tested under various strengthening configurations. However, the properties of the basic concrete beam before strengthening were the same for most specimens, except for a few beams which were given a higher internal reinforcement ratio. In this section, a general description is provided of the RC beams and their different fabrication stages, the procedures used to strengthen the beam specimens, the instrumentation of the beams and the test-setup.

3.2.1 Materials Used and Their Properties

3.2.1.1 Concrete and Cement Mortar

Ordinary portland cement (OPC) was used in casting the beams. Crushed stone (granite) was used as a coarse aggregate and the maximum aggregate size was 20 mm. It was sieved through a 4.5 mm sieve and air-dried in the concrete laboratory. Natural river sand was used as a fine aggregate. A sieve analysis was done in accordance with BS 882 to determine the grading of the fine aggregate. The grading of the sand used as fine aggregate was two. Before casting, the coarse aggregate were washed with water and air dried in the concrete laboratory to get the saturated surface dry (SSD) condition. Fresh tap water was used to hydrate the concrete mix during the casting and curing of the beams, cubes, prisms and cylinders. The concrete mix was designed for 30 MPa strength using the DOE method. The mix proportions adopted are shown in Table 3.1. The compressive strengths of the concrete were obtained from three cubes after twenty-eight days curing according to the British Standard (BS 1881). The average compressive strength was 30 MPa. Cement mortar was also used for NSM strengthened beam. 50% cement and 50% sand by weight basis were mixed with water to make mortar. The water to cement ratio of this mortar was 0.50.

Table 3.1: Concrete mix design

Slump (mm)	Water Cement ratio	Concrete (Kg/m ³)			
		Water	Cement	Coarse Aggregate	Fine Aggregate
60-180	0.65	208	320	740	1120

3.2.1.2 Internal Steel Reinforcement

Four types of steel bars were used in the preparation of the beam specimens. These were 12 mm, 10 mm, 8 mm and 6 mm diameter bars. The 12 mm bars were used as flexural reinforcement. The 12 mm bars were bent ninety degrees at both ends to fulfill the anchorage criteria. The 10 mm bars were used as hanger bars in the shear span zone. The 6 mm bars were used for stirrups. 6 mm, 8 mm and 10 mm bars were used for NSM strengthening purpose. The test data obtained for 6 mm plain bars are shown in Appendix A.

3.2.1.3 Steel Plate

Mild steel plates were used for strengthening RC beam. The yield and ultimate tensile strength of the steel plates were 420 MPa and 475 MPa. The modulus of elasticity was 200 GPa. Three different thicknesses of steel plates were used such as 1.5 mm, 2 mm and 2.76 mm. In addition to strengthening, 2 mm thick and 100 mm wide L-shape steel plates were used for end anchorage.

3.2.1.4 CFRP Plate and Fabrics

The tensile strength and modulus of elasticity of CFRP plates were 2800 MPa and 165 GPa, respectively. The design and ultimate strain of CFRP plates were 0.0085 and 0.017, according to the manufacturer's (SIKA) specifications. Fiber in matrix is shown in Figure 3.1. The Sikadur 30 resin has 1% elongation at failure, which is less than the ultimate elongation of the CFRP plate material (1.9%).

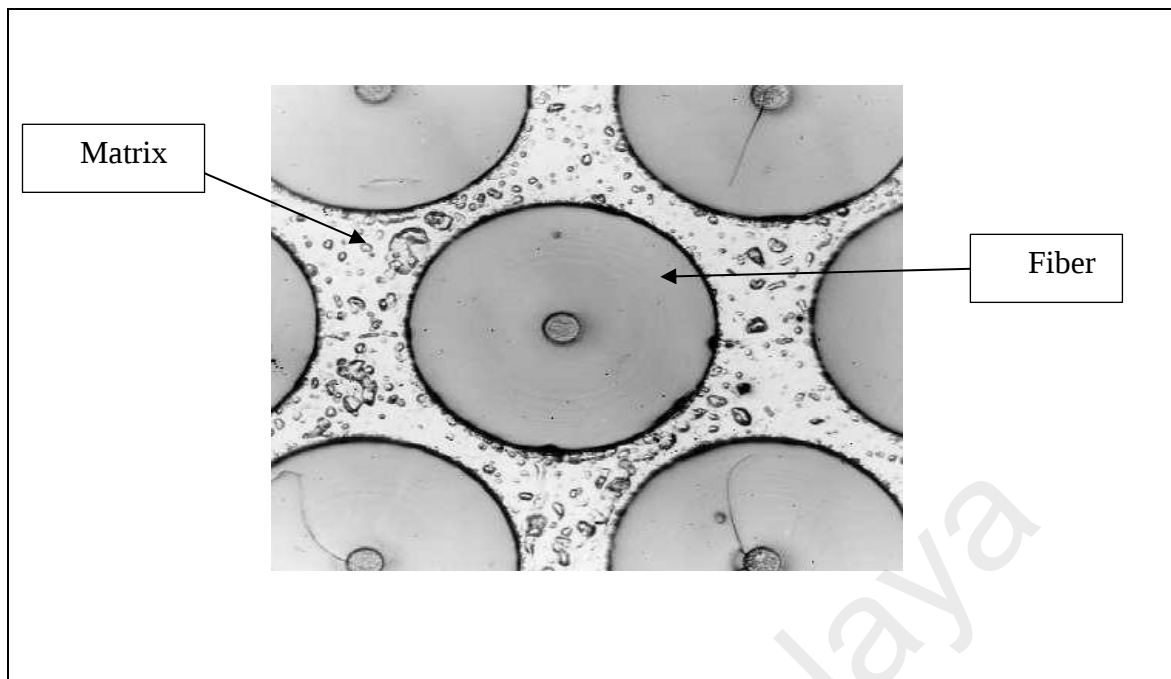


Figure 3.1: Fiber in matrix (Badawi, 2007)

Beside the CFRP plate, CFRP fabric was used for both flexural strengthening and end anchorage. The thickness of this fabric was 0.17 mm. The tensile strength and elastic modulus of dry fiber was 4900 MPa and 230 GPa, respectively. The elongation at breaking point was 2.1%.

3.2.1.5 Adhesive

Sikadur 30 epoxy adhesive was used as a bonding agent between the strengthening materials and the tension surface of the concrete beams. Sikadur 30 is a high strength and high modulus structural epoxy adhesive. It also has a high creep resistance under long term loads. According to the manufacturer, its tensile strength at seven days is 24.8 MPa; it has 1% elongation at failure, and a modulus of elasticity of 11.2 GPa. The bond strength of Sikadur 30 can vary based on the curing conditions and the bonded materials. Sikadur 30 epoxy adhesive has two components, namely component A and component B. Component A is white in color and consists of the epoxy resin. Component B is black in

color and consists of the hardener. Component A and component B are mixed together in 3:1 ratio by weight until a uniform grey- colored paste is achieved. No solvent is added. The paste is then applied to the required surfaces. The surfaces must be prepared before application. Sikadur 30 reaches its design strength seven days after application.

3.2.2 Design and Preparation of Beam Specimen

All the beam specimens were 2300 mm long, 125 mm wide, and 250 mm deep as shown in Figure 3.2. The beams were reinforced with two 12 mm diameter steel bars in the tension zone as the main reinforcement. Two 10 mm steel bars were used as hanger bars in the shear span and were placed at the top of each beam. For shear reinforcement, 6 mm bars were used and were placed symmetrically apart. The spacing of the shear reinforcement was 75 mm. Enough shear reinforcements were provided in an amount calculated to ensure that the beams would fail in flexural. However, certain beams were given additional reinforcement by using 6 mm stirrups spaced at 40 mm instead. The various steel bars were arranged to make a reinforcing steel cage before the concrete casting of each beam. The details of the internal reinforcement used in a typical beam are shown in Figure 3.2. A typical concrete cover of 30 mm was used.

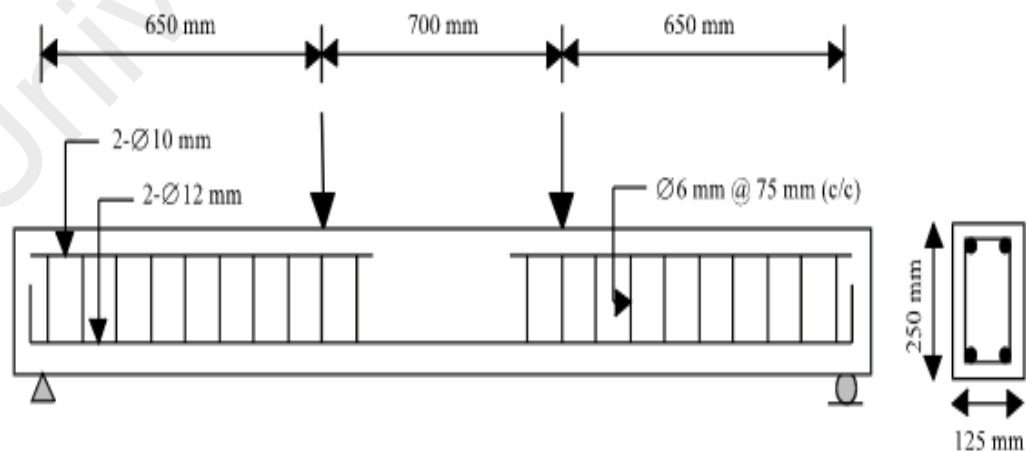


Figure 3.2: Details of the beam specimens

To measure the strain in the tension reinforcement during loading, two 5 mm strain gauges were mounted at the mid-span of the beams on each rebar. In order to place the strain gauges, the surfaces of the reinforcing bars were ground to remove the ribs and to flatten the surfaces. After grinding, the surfaces were cleaned by acetone to remove small dust particles. To allow the adhesive to set, the strain gauges were left for several hours. The strain gauges were then connected to wires by soldering. The connection between the wires and the strain gauges was checked using a multi-meter. After wiring the strain gauges, they were coated with silicon to protect them from damage during and after the concrete casting. The reinforcing steel cages were then placed into steel molds. Proper care was taken to avoid disturbing the strain gauges while the beam was being cast.

The concrete was prepared by mixing cement, sand, coarse aggregate and water in the concrete mix proportions mentioned above (Table 3.1) using a laboratory drum mixer of 500 kg capacity. Steel molds were used for casting. Before pouring the concrete, the steel molds were cleaned and greased. After the concrete was placed in the mold, it was compacted using a poker vibrator. The beams were cast in three layers and each layer was compacted using a poker vibrator to ensure adequate compaction. During the vibration process, each penetration was made at a reasonable distance from each other to avoid bleeding and segregation of the concrete. The subsequent curing was done by covering the beams with wet hessian cloths for at least two weeks.

Besides the beam, nine 100 mm $\times$ 100 mm $\times$ 100 mm cubes, three cylinders of 150 mm diameter $\times$ 300 mm height and three 100 mm $\times$ 100 mm $\times$ 500 mm prisms were cast from the same batch of fresh concrete. These were cured and tested in accordance with BS standards to determine the compressive strength and flexural strength (modulus of rupture) of the concrete as shown in Figure A1 in Appendix A.

3.2.3 Strengthening of RC Beams

All specimens except control beams were strengthened using various methods. Specifically, two strengthening methods were used. These were the NSM and HSM. Strengthening requires careful observation and preparation of the beam. After the beam specimens were cured for 28 days, they were ready for structural strengthening. Basically, two types of material were used for structural strengthening, steel and CFRP, but in various configurations. The basic procedures carried out in strengthening the RC beam specimens are described in the following subsections.

3.2.3.1 Surface Preparation

The surfaces of both concrete and steel plates require special preparation for proper bonding between the concrete and strengthening material is used. All dust, laitance, grease, curing compounds, foreign particles, disintegrated materials and other bond inhibiting materials must be removed from the bonding surfaces. The concrete surface has to be clean and sound. In addition, the texture of the coarse aggregate in the concrete must be exposed. To achieve this, the bonding faces of all concrete beams were ground with the help of a diamond cutter to obtain a rough surface and to expose the texture of the coarse aggregate, as shown in Figure 3.3



Figure 3.3: Prepared surface of a concrete beam

The grounded concrete surfaces were then cleaned to remove dust, loose particles and any other foreign material. A wire brush and a high pressure air jet were used to clean the surface as shown in Figure 3.4. After this surface treatment, putty was applied to fill up any cavities or holes in the bonding surface of the concrete beam.



Figure 3.4: Compressed air jetting



Figure 3.5: Sand blasted steel plate

The surfaces of the steel plates and the CFRP laminates were also prepared. The bonding surfaces of the steel plates were sand blasted in accordance with Swedish standards to ensure adequate bonding between the concrete and steel plates. The sand blasted surfaces of two steel plates are shown in Figure 3.5. The sand blasted steel plates were then cleaned with acetone to remove small foreign particles and dust. The CFRP laminates were cleaned using Colma cleaner to remove carbon dust from the bonding surfaces.



Figure 3.6: Groove cutting

For the beams strengthened using NSM or the HSM, either one or two grooves were cut along the length of the tension faces of the concrete beams for the placement of the NSM bars. The grooves were made by making parallel cuts with a diamond concrete saw as deep as the desired depth (two times bar dia) of the NSM groove. Figure 3.6 shows a groove being cut into a beam. The grooves were then cleaned using a wire brush and a high pressure air jet to remove dust and loose particles as shown in Figure 3.4.

3.2.3.2 Placement of Strengthening Materials

Strengthening was done using two methods, namely the NSM method and the HSM. Steel plates, steel bars and CFRP plates were used in various configurations for strengthening. For the NSM method steel bars were used. For the HSM a combination of either steel plate and steel bars or CFRP plate and steel bars was used. To bond the strengthening materials to the surfaces of the concrete beams an epoxy adhesive was used, namely Sikadur 30. This epoxy was chosen for its excellent engineering properties, which include its high strength, high modulus, and high creep resistance under long term loads.

To prepare the epoxy adhesive, Sikadur 30, its two components (resin and hardener) were mixed in a ratio of 3:1 until a uniform grey-colored paste was achieved. In the case of NSM strengthening, the prepared groove was half-filled with the prepared epoxy adhesive and then the NSM steel bar was pressed into the centre of groove until the adhesive flowed around the sides of the bar. Then, the remaining space in the groove was filled with the epoxy adhesive and levelled using a spatula. The specimens were allowed to cure for at least seven days before testing. In HSM, the beam specimens were first strengthened using the NSM method and then using the externally bonded method. In all cases, the prepared beam specimens were not disturbed for at least seven days to allow proper curing to take place.

3.2.4 Instrumentation

3.2.4.1 Demec Points

Demec points were installed on the side surfaces of each concrete beam to measure strain and to determine the position of the neutral axis of the beam sections. The distance between two horizontally placed Demec points was 200 mm. The concrete surface where each Demec point was to be installed was grounded to ensure proper bonding. The surface was then cleaned with acetone to remove dust. After preparing the concrete surface, the

Demec points were installed using an adhesive as shown in Figure 3.7 and allowed to set for at least 24 hours.

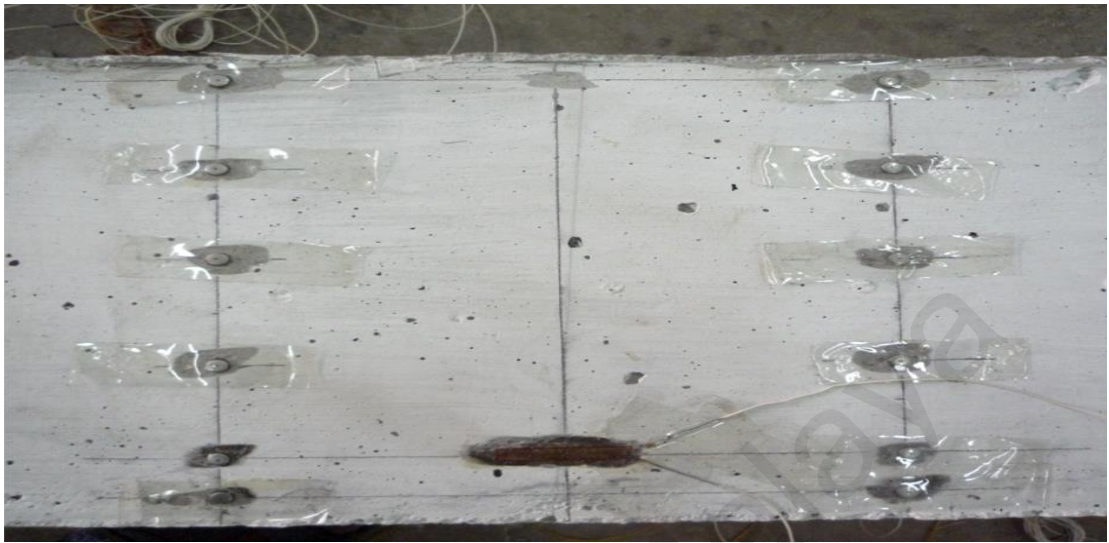


Figure 3.7: Demec points on a concrete beam with a strain gauge

3.2.4.2 Electrical Resistance Strain Gauges

Electrical resistance strain gauges were used to measure strain in the steel bars, steel plates, CFRP plates and concrete. Before casting, the main rebars of each beam were ground using a mechanical grinder at mid span as shown in Figure 3.8. After grinding, the surface was cleaned with acetone to remove steel fragments and dust particles.



Figure 3.8: Surface preparation of steel bars to place strain gauges

Two 5 mm gauges were attached to the middle of the rebars of each beam by fast setting adhesive on the top or bottom face of the two main steel rebars as shown in Figure 3.9. These two gauges were used to record the tension strain in the steel rebars. To allow the adhesive to set properly, the attached strain gauges were left for several hours. The strain gauges were then connected with wire by soldering as shown in Figure 3.9. The electrical connection was checked using a multi-meter. The reading was found to be 120 Ω , which is acceptable.



Figure 3.9: Attachment of strain gauges

Silicon was applied on the strain gauges as well as on the necked wire (shown in Figure 3.10) to seal them from water exposure during and after casting. Proper care was taken to not disturb the electrical resistance strain gauges more than necessary while each beam was being cast. The connection of the strain gauges was checked again after casting.



Figure 3.10: Strain gauges covered with silicone gel

Two 30 mm strain gauges were placed at the middle of the top face of each concrete beam and at the bottom of the strengthening steel or CFRP plate to measure the concrete compressive and plate tensile strains. A 30 mm strain gauge was also installed in the middle of the two lowest Demec points in order to verify demec readings as shown in Figure 3.7.

3.2.4.3 Linear Variable Displacement Transducers (LVDTs)

One LVDT was used for each beam at the middle of the span for all cases except one where three LVDTs were used. The LVDT had workable transverse ranges of 50 mm and were used to measure the deflection of each beam at mid span. All the transducers were connected to a portable data logger to record the deflections of the beams during testing.

3.2.4.4 Data Logger

The data logger used in this study is a TDS-530. It was used in the testing of each beam specimen to record the data of several strain gauges placed at different positions, three LVDTs and loads from an Instron testing machine. The strain gauges were connected as 1G3W120 Ω to the data logger and the unit of strain measurement was micro-strain. The

LVDTs were connected as 4GAGE to the data logger and the unit of deflection measurement was millimeters.

3.2.4.5 Digital Extensometer

The bending deformation of each beam under loading was measured from its Demec points using a digital extensometer. This was used to estimate the strain profile of each beam and to determine the position of the neutral axis. The attachment of Demec points on the side surface of beam specimens is described in section 3.2.4.1.

3.2.4.6 Dino-lite Digital Microscope

This instrument (Figure 3.11) was used to measure the crack widths in concrete beam specimens during tests. Using this device, crack widths could be measured with an accuracy of up to 0.001 mm. The adjustable lens allowed very sharp pictures of the cracks to be taken, from which the crack widths could be estimated accurately. However, the spacing between different cracks along the length of the beams had to be measured manually.

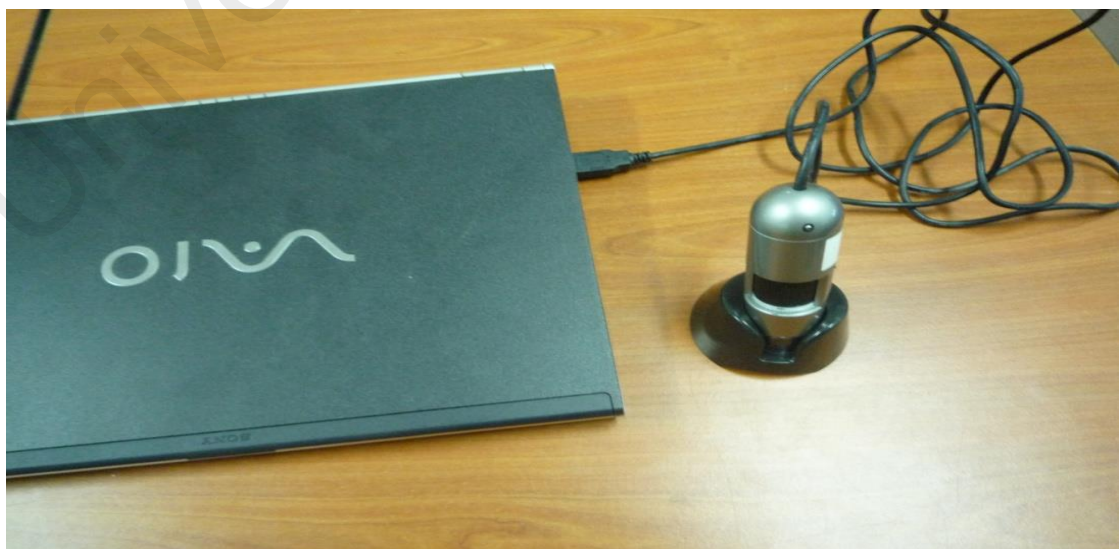


Figure 3.11: Dino-lite digital microscope for crack width measurement

3.2.5 Test Setup and Procedure

All beam specimens were tested in four-point bending as shown in Figure 3.12. All specimens were simply supported using steel roller support (Static) and elastomeric bearing pads (fatigue) and were subjected to two point loading. The distance between the two supports was 2000 mm and the distance between the two loading points of the spreader beam was 700 mm. The resulting shear span to depth ratio was about 3. For the static load tests, the actuator was loaded and moved down at a rate of 1 mm/min so that readings from the data logger could be taken and visible cracks measured easily.

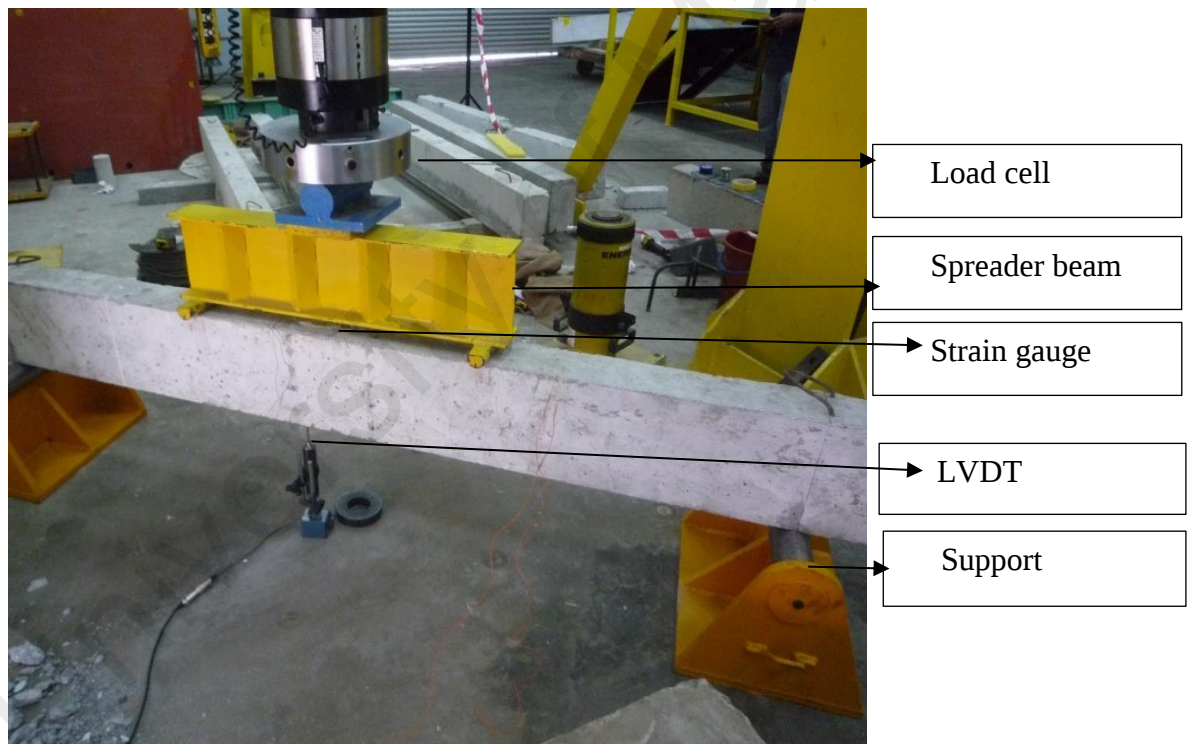


Figure 3.12: Experimental set up

Each beam was lifted and positioned on to the supports leaving 150 mm lengths of beam at both ends so that the beams were simply supported by a span of 2000 mm. The LVDTs were then placed appropriately, ensuring that the transducers touched the plate or

concrete. All the strain gauges and LVDTs were connected to the data logger and the data logger was calibrated. After recording all data from the data logger, the spreader beam was placed on top of the beam specimens to ensure two-point loading. The data from the data logger were again recorded.

The tests were conducted using a closed-loop hydraulic Instron Universal Testing Machine. For repeated loading, a closed-loop system programmed to deliver a sinusoidal load at a frequency of 3 Hz was used. The load span, load set point, frequency and preset number of cycles were controlled by an electronic controller. The sinusoidal waveform was checked through computer.

3.2.6 Test Matrix

The beam specimens were divided into five groups. The first group, series C (Figure 3.13), is the control group, where the beams were left unstrengthened. The second group, series P (Figure 3.14), was strengthened with externally bonded steel or CFRP plates. The data for this series was entirely taken from a previous study by another researcher, Alam (2010) in same laboratory. This was done to compare the effectiveness of the HSM with the external EBR. In the third group, series N (Figure 3.15), the beams were strengthened with NSM steel bars. In the fourth group, series H (Figure 3.16), the beams were strengthened using the HSM where either a combination of steel plates and steel bars or CFRP plates and steel bars were used. The fifth group, series SH (Figure 3.17), was where HSM was applied on the sides of the beams. Within each group, various configurations and dimensions of the different strengthening materials were used.

Table 3.2 gives a detailed overview of the beam specimens tested and analysed in this research. Table 3.2 gives the beams that were directly tested by this researcher. Notation used in Table 3.2 has been described in Table 3.4 and Table 3.5. Table 3.3 shows the

details of beams that were tested in a previous study (Alam, 2010), and is included in this research.

Table 3.2: Test matrix1

No.	Series	Notation	Description	Descriptions of Strengthening
1	C Series (Figure 3.13)	CB	Control beam	-
2	N Series (Figure 3.15)	N2S6C	Beam strengthened with NSM steel bar and cement mortar	2 ϕ 6 mm bar
3		N2S6E	Beam strengthened with NSM steel bar and epoxy	2 ϕ 6 mm bar
4		N2S6EC	Beam strengthened with NSM steel bar, cement mortar and epoxy	2 ϕ 6 mm bar
5		N1S8E	Beam strengthened with NSM steel bar and epoxy	1 ϕ 8mm bar
6		N1S8C	Cracked beam strengthened with NSM steel bar and cement mortar	1 ϕ 8mm bar
7		N3S8C	Beam strengthened with NSM steel bar and cement mortar	3 ϕ 8mm bar
8		N1SH8C	Beam (higher internal ratio) strengthened with NSM steel bar and cement mortar	1 ϕ 8 mm bar
9		N2SS8C	Beam strengthened with NSM steel bar (side of beam)	2 ϕ 8 mm bar
10	H Series (Figure 3.16)	H1B8S19L73W2T	Beam strengthened with hybrid bonded steel plate and steel bar	1 ϕ 8 mm Steel plate (2 $\times$ 73 $\times$ 1900 mm ³)
11		H1B8S16L73W2T	Beam strengthened with hybrid bonded steel plate and steel bar	1 ϕ 8 mm Steel plate (2 $\times$ 73 $\times$ 1650 mm ³)
12		H1B6S16L73W2T	Beam strengthened with hybrid bonded steel plate and steel bar	1 ϕ 6 mm Steel plate (2 $\times$ 73 $\times$ 1650 mm ³)
13		H2B8S19L73W2T	Beam strengthened with hybrid bonded steel plate and steel bar	2 ϕ 8mm Steel plate (2 $\times$ 73 $\times$ 1900 mm ³)
14		H2B6S19L73W2T	Beam strengthened with hybrid bonded steel plate and steel bar	2 ϕ 6 mm Steel plate (2 $\times$ 73 $\times$ 1900 mm ³)
15		H2B6S19L73W2.76T	Beam strengthened with hybrid bonded steel plate and steel bar	2 ϕ 6 mm Steel plate (2.76 $\times$ 73 $\times$ 1900 mm ³)
16		H2B6S19L125W2T	Beam strengthened with hybrid bonded steel plate and steel bar	2 ϕ 6mm Steel plate (2 $\times$ 125 $\times$ 1900 mm ³)
17		H1B8SD19L73W2T	Beam (different spacing of shear reinforcement) strengthened with hybrid steel plate and steel bar	1 ϕ 8 mm 1-Steel plate (2 $\times$ 73 $\times$ 1900 mm ³)
18		H2B6S19L125W1.5T	Beam strengthened with hybrid bonded steel plate and steel bar	1 ϕ 10 mm Steel plate (1.5 $\times$ 73 $\times$ 1900 mm ³)

Table 3.2 continue

19	H Series (Figure 3.16)	H1B8S19L73W2TAS	Beam strengthened with hybrid bonded steel plate and steel bar and shear strengthening with CFRP fabric	1 ϕ 8 mm Steel plate (2×73×1900 mm ³) CFRP fabric – Width = 200 mm
20		H1B8S19L73W2TAF	Beam strengthened with hybrid bonded steel plate and steel bar and shear strengthening with CFRP fabric	1 ϕ 8 mm Steel plate – (2×73×1900 mm ³) CFRP fabric (Width = 100 mm)
21		H1B8F19L80W1.2T	Beam strengthened with hybrid bonded CFRP plate and steel bar	1 ϕ 8 mm CFRP plate (1.2×80×1900 mm ³)
22		H1B8F16L80W1.2T	Beam strengthened with hybrid bonded CFRP plate and steel bar	1 ϕ 8 mm CFRP plate (1.2×100×1650 mm ³)
23		H1BP8F16L80W1.2T	Beam strengthened with hybrid bonded CFRP plate and steel bar	1 ϕ 8 mm 1 CFRP plate (1.2×100×1650 mm ³)
24		H1BP6F16L80W1.2T	Beam strengthened with hybrid bonded CFRP plate and steel bar	1 ϕ 8 mm 1-CFRP plate 1.2×100×1650 mm ³
25		H2BP6F16L80W1.2T	Beam strengthened with hybrid bonded CFRP plate and steel bar	1 ϕ 8 mm CFRP plate (1.2×100×1900 mm ³)
26		H1B8F19L80W1.2TAF	Beam strengthened with hybrid bonded CFRP plate and steel bar	2 ϕ 6mm CFRP plate (1.2×100×1900 mm ³)
27		H1B6FR19L100W.17T	Beam strengthened with hybrid bonded CFRP fabrics and steel bar	1 ϕ 6mm 1-CFRP fabric (.165×100×1900 mm ³)
28	SH Series (Figure 3.17)	SH2S61900L100W2T	Beam strengthened with side-applied hybrid bonded CFRP plate and steel bar	2 ϕ 6mm 2-Steel plate (2×50×1900 mm ³)
29	Fatigue Series	CBF50	Control for fatigue	-
30		CBF80	Control for fatigue	-
31		PSF	Beam strengthened with externally bonded steel plate	1-Steel plate (2.76×100×1900 mm ³)
32		NSF	Beam strengthened with NSM steel bar	1 ϕ 8 mm
33		HSF	Beam strengthened with hybrid bonded steel plate and steel bar	1 ϕ 8mm Steel plate (2×100×1900 mm ³)

Table 3.3: Test matrix2 (Taken from Alam (2010))

Sl. no	Series	Notation	Description	Description of strengthening
1	C Series (Figure 3.13)	CB1	Control beam	
2	P series (Figure 3.14)	PS19L73W2.76T	Beam strengthened with Steel plate (Alam, 2010)	1-Steel plate (2×73×1900 mm ³)
3		PS16L73W2.76T	Beam strengthened with Steel plate (Alam, 2010)	1-Steel plate (2×73×1650 mm ³)
4		PF19L80W1.2T	Beam strengthened with CFRP plate (Alam, 2010)	1- CFRP plate (1.2×80×1900 mm ³)
5		PF16L80W1.2T	Beam strengthened with CFRP plate (Alam, 2010)	1-CFRP plate (1.2×80×1650 mm ³)

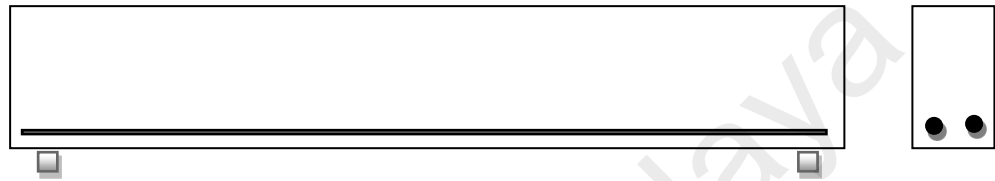


Figure 3.13: Series CB beam (Control beam)

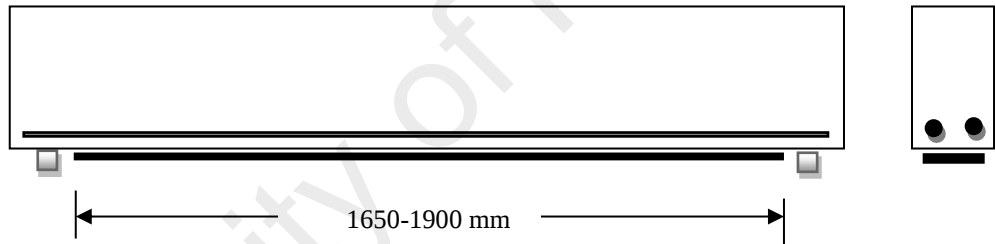


Figure 3.14: Series P (EBR)

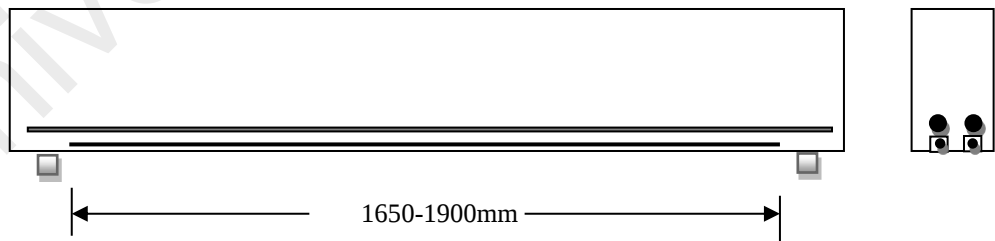


Figure 3.15: Series N (NSM strengthening)

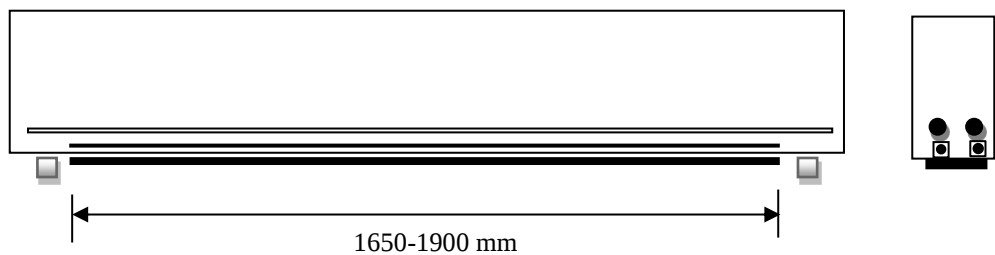


Figure 3.16: Series H (HSM strengthening)

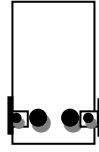


Figure 3.17: Cross-section of series SH beam (HSM at sides)

Table 3.4: Description of beam notation for HSM.

H 1 B 8 S 19L 73W 2T AF

H	1	B	8	S	19L	73W	2T	AF
Name of series	No. of bar	Position of Bar	Diameter of bar in NSM	Materials for strengthening	Length of plate	Width of plate	Thickness of plate	Anchorage
H = HSM		B = bottom	6 = 6 mm	S=Steel	19L= 1900 mm	73W = 73 mm	2T = 2 mm	AF = Anchorage with Full wrap
N = NSM		S = side	8 = 8 mm	F=FRP	16L= 1650 mm	80W = 80 mm	2.76T = 2.76 mm	AS = Anchorage with side wrap
P = EBR				SD=Steel but different spacing of internal shear reinforcement		125W= 125 mm	1.5T = 1.5 mm	
SH = Side HSM								

Table 3.5: Description of beam notation for NSM strengthening.

N 3 S 8 C

N	3	S	8	C
Name of series	No. of bar	Material for strengthening with position	Diameter of bar	Adhesive type
N-NSM	3=3 Nos	S = Steel bar with bottom		E = Epoxy
		SS = Steel bar with side face	8=8 mm	C = Cement mortar
		SH = Steel bar with higher internal reinforcement		EC = 50% Epoxy and 50% Cement along the length

3.3 Development of Semi-numerical Model

Numerical analysis is the study of algorithms that use numerical approximation (as opposed to general symbolic manipulations) for the problems of mathematical analysis. The main objective of numerical analysis is to obtain approximate solutions while maintaining reasonable bounds on errors. Iterative methods are more common in numerical analysis. Starting from an initial guess, iterative methods form successive approximations that converge to the exact solution. The number of steps needed to obtain the exact solution is large that an approximation is accepted. The method of mathematical optimization (alternatively, optimization or mathematical programming) can be used to reduce these large number of steps. Therefore, application of mathematical optimization in numerical analysis is referred to semi-numerical method.

Mathematical optimization is the selection of a best constituent from some set of available alternatives based on certain criteria. An optimization problem comprises of maximizing or minimizing a function by scientifically choosing the values of input variables from an allowable set and estimating the value of the function. The generalization of optimization theory and techniques to other formulations comprises a large area of applied mathematics. More generally, optimization includes finding "best available" values of some objective function given a defined set of constraints.

Application of mathematical optimization technique make the semi-numerical method more efficient because of less number of steps or iterations. However, it has still some drawbacks. In current study, tension stiffening effect of internal reinforcement was not considered. Incorporation of proper tension stiffening effect will make the semi-numerical model more perfect and robust.

3.3.1 Material Properties

3.3.1.1 Concrete

Developing a model for the behaviour of concrete is a complex task. Concrete is a semi-brittle material and behaves differently in tension and in compression. The ultimate uniaxial tensile strength and compressive strength are required to define a failure surface for the concrete. In tension, the concrete stress–strain curve is linearly elastic up to the ultimate tensile strength. After this value, the concrete cracks and the strength reduces to zero. The tensile strength of concrete is usually 8 - 15% of its compressive strength (Shah et al., 1995). Figure 3.18 shows a typical stress-strain curve for normal weight concrete (Bangash, 1989).

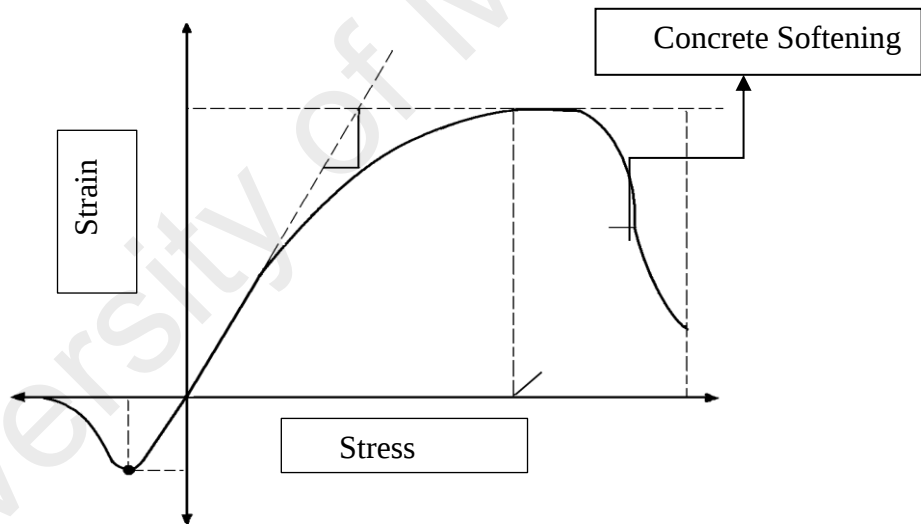


Figure 3.18 : Stress-strain relationship of concrete (Bangash, 1989)

The stress in the concrete and corresponding strain can be expressed by the Equations (3.1) (3.2) and (3.3) according to Hognestad's parabola:

$$\sigma_c = f'_c \left[2 \frac{\epsilon_c}{\epsilon'_c} - \left(\frac{\epsilon_c}{\epsilon'_c} \right)^2 \right] \quad (3.1)$$

$$\epsilon'_c = 2 \frac{f'_c}{E_c} \quad (3.2)$$

$$E_c = 5700(f'_c)^{1/2} \quad (3.3)$$

where:

σ_c = the concrete stress corresponding to a given concrete strain (ϵ_c),

f'_c = the concrete compressive strength,

ϵ_c = the concrete strain corresponding to a given concrete stress (σ_c),

ϵ'_c = the concrete strain corresponding to the concrete compressive strength, and

E_c = Young's modulus of concrete.

3.3.1.2 Steel Bars and Plates

The compression and tension reinforcement are assumed to be elastic-plastic with a 1% strain hardening slope (bi-linear behaviour). The idealized stress-strain relationship is shown in Figure 3.19.

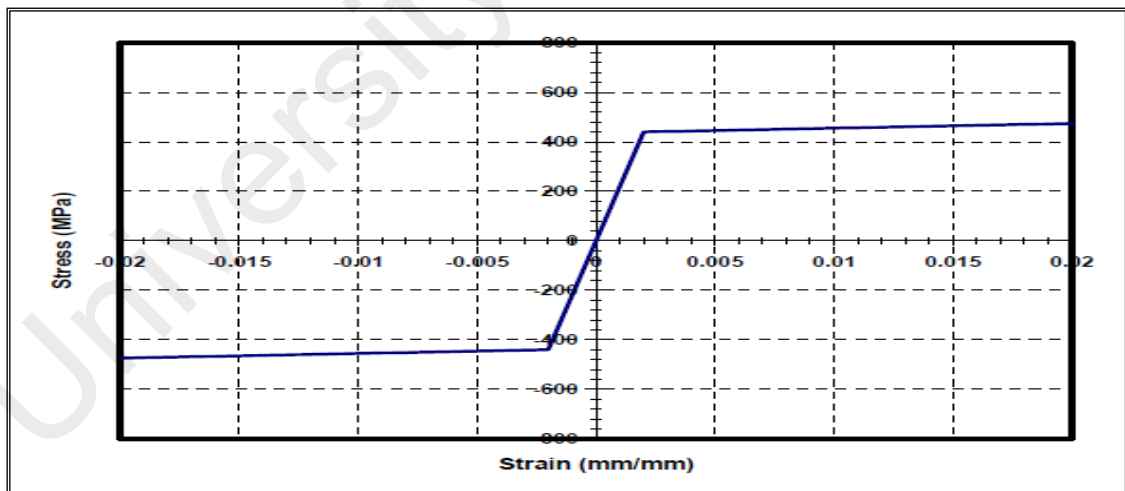


Figure 3.19: Stress-strain relationship of steel bar and plate

Equations (3.4) and (3.5) express the relationship between steel stress and the corresponding strain.

$$\sigma_s = \varepsilon_s E_s \quad \text{if } \varepsilon_s < \varepsilon_y \quad (3.4)$$

$$\sigma_s = \varepsilon_s E_s + E_{sp}(\varepsilon_s - \varepsilon_y) \quad \text{If } \varepsilon_s > \varepsilon_y \quad (3.5)$$

where:

σ_s = the steel stress corresponding to a given steel strain (ε_s),

f_y = the steel yield stress corresponding to the steel yield strain (ε_y),

ε_s = the steel strain corresponding to a given steel stress,

ε_y = the steel yield strain corresponding to the steel yield stress,

E_s = the modulus of steel before yielding,

E_{sp} = the modulus of steel after yielding.

3.3.1.3 CFRP Composite

The stress-strain curve for a CFRP plate is linearly elastic up to failure. The relationship is given in Equation (3.6).

$$\sigma_{cfrp} = E_{cfrp} \varepsilon_{cfrp} \quad (3.6)$$

where:

σ_{cfrp} = the CFRP stress corresponding to a given CFRP strain, and

E_{cfrp} = Young's modulus of CFRP.

3.3.2 Modeling Methodology

In this study, section analysis is used to estimate the strains and the curvatures along the length of the beam for modeling by using the Equations (3.7) to (3.18). It is a familiar topic to engineers as the idea is strongly embedded in codes of practice. Sectional analysis lies between graphical hand method of analysis and finite element computer program (Bentz, 2000). In using the sectional analysis approach, the problem of determining the response of a reinforced concrete structure to applied loads is broken up into two interrelated tasks. First, the sectional forces at various locations in the structure caused by

the applied loads are determined. This step is usually performed assuming that the structure remains linearly elastic. Then the response of a local section to the sectional forces is determined. The second step, which is the sectional analysis, the non-linear characteristics of cracked reinforced concrete are taken into account. Figure 3.20 shows the strain, stress, and force distribution on a section of a beam.

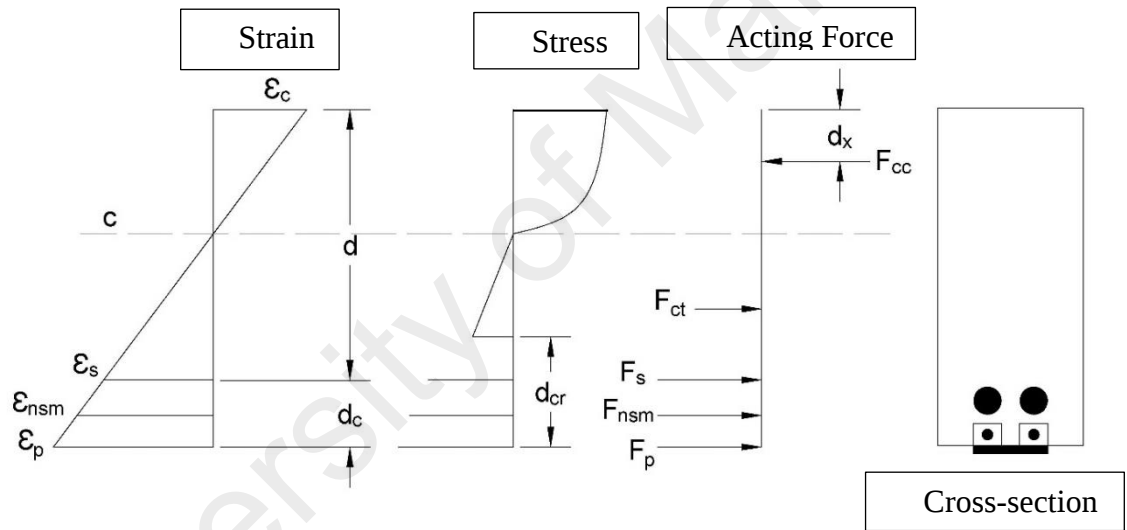


Figure 3.20: Strain, stress and force distribution on a section

$$\epsilon_s = \epsilon_c \frac{d - c}{c} \quad (3.7)$$

$$\epsilon_p = \epsilon_c \frac{h - c}{c} \quad (3.8)$$

$$F_{cc} = b c f'_c \left(\frac{\epsilon_c}{\epsilon'_c} \right) \left(1 - \frac{\epsilon_c}{3 \epsilon'_c} \right) \quad (3.9)$$

$$F_s = A_s E_s \epsilon_s \quad \text{if } \epsilon_s < \epsilon_y \quad (3.10)$$

$$F_s = A_s(f_y + 0.01E_{sp}(\epsilon_s - \epsilon_y)) \quad \text{if } \epsilon_s > \epsilon_y \quad (3.11)$$

$$F_{nsm} = A_{nsm}E_{nsm}\epsilon_{nsm} \quad \text{if } \epsilon_{nsm} < \epsilon_y \quad (3.12)$$

$$F_{nsm} = A_{nsm}(f_y + 0.01E_{sp}(\epsilon_{nsm} - \epsilon_y)) \quad \text{if } \epsilon_{nsm} > \epsilon_y \quad (3.13)$$

$$F_p = A_pE_p\epsilon_p \quad \text{if } \epsilon_s < \epsilon_y \quad (3.14)$$

$$d_x = \left[1 - \frac{\frac{2}{3} - \frac{\epsilon_c}{\epsilon'_c}}{1 - \frac{\epsilon_c}{3\epsilon'_c}} \right] \quad (3.15)$$

$$F_c - F_s - F_{nsm} - F_p - F_{ct} = 0 \quad (3.16)$$

$$F_c(h - d_x) - F_s(h - d) - F_{nsm}\frac{d_c}{2} - F_{ct}\left\{(h - c) - \frac{2d_{cr}}{3}\right\} = M_{int} \quad (3.17)$$

$$M_{ext} = M_{int} \quad (3.18)$$

3.3.3 Deflection Prediction Model

3.3.3.1 Steps to Predict the Deflection:

The calculation procedure to predict the load-deflection of beam specimen (control and strengthened) beam is as follows:

- i. Assume a given external applied load on the beam.
- ii. Calculate the external moment.
- iii. Assume a strain at the compression fiber of the concrete.
- iv. Assume the neutral axis depth.
- v. Calculate the strains in the tension steel, NSM steel and steel/CFRP reinforcement by using triangular rule.
- vi. Calculate stresses and forces in the compression concrete, tension steel, NSM steel bar and steel/CFRP plate.

- vii. Evaluate force equilibrium equations. If not in equilibrium, change the neutral axis depth in step 4 and repeat steps 4 to 7 until in equilibrium.
- viii. If the forces are equilibrium, calculate internal moment by taking moment against strengthening plate level.
- ix. Compare the calculated internal moment to the external moment obtained in step 2. If not equal, change the assume strain in step iii and repeat steps iii to ix.
- x. If external moment is equal to internal moment, calculate the deflection using semi-numerical approach described in sub-section 3.3.3.2 and record the load and deflection data.
- xi. Calculate the deflection and record the load and deflection data.
- xii. Apply the load increment and repeat steps 2 to 10 until failure.

3.3.3.2 Semi-numerical Approach

The sectional analysis is usually done by making assumption of two unknown, strain of any material and depth of neutral axis, and applying trial and error approach. These two unknown could be solved by using two equilibrium equations (force and moment equilibrium). However, the direction of assumption can be complicated due to divergence problem. This problem can be eliminated by the application of mathematical programming (non-linear programming and genetic algorithm) technique. Therefore, complicated several steps will be reduced to one easy step. The calculation procedure to predict the load-deflection curve of beam specimen (control and strengthened) beam is as follows:

- i. Assume a given external applied load on the beam.
- ii. Calculate the external moment.

- iii. Apply an optimization algorithm to estimate concrete strain and the neutral axis depth by using an objective function as equation (3.19) and constraint as $\varepsilon_c < 0.0035$ at the compression fiber of concrete and $\varepsilon_p < 0.0065$ rupture of FRP fiber.
- iv. Calculate curvature an element of the beam and curvature distribution along the beam according to the procedure described in Badawi (2007).
- v. Calculate the deflection by integrating the curvature.

Objective function can be expressed using Equation (3.19) given below:

$$\text{Minimize error} = (M_{\text{ext}} - M_{\text{int}})^2 + (F_{\text{com}} - F_{\text{ten}})^2 \quad (3.19)$$

3.3.4 Flexural Strength Model

The main objective of flexural strength model is to estimate ultimate flexural capacity of the beam. Since M_{ext} is equal to M_{int} according to moment equilibrium theory, maximum value of M_{ext} calculated from Equation (3.17) will be ultimate flexural capacity of the beam when either ε_c , compression fiber of concrete will reach 0.0035 or ε_p , strain of FRP fiber will reach 0.0065.

3.3.5 Debonding Strength Model

3.3.5.1 Modelling Methodology

The objective of this subsection is to develop a simple and rational methodology for predicting the debonding failure load for strengthened concrete beams that can be used in practical design applications. The methodology is employed in two steps: 1) predicting the principle stresses at the plate end; and 2) comparing these stresses with the limiting stresses in an appropriate concrete failure criteria.

Jones et al. (1988) recommended the use of elastic shear stress (τ) calculated from classical beam theory to predict the interfacial shear stress at plate ends. It is a simple procedure with a strong theoretical background. To calculate the interfacial shear stress the following expression is used. The elastic shear stress expression is:

$$\tau = \frac{V_o A_f b_f n_f y_f}{I_c b_f} \quad (3.20)$$

where:

V_o = shear force at the plate curtailment location,

A_f = area of the plate,

n_f = modular ratio of the late (E_f / E_c),

y_f = distance of the plate from the neutral axis,

I_c = transformed moment of inertia of beam cross section in terms of the concrete and

b_f = width of the plate.

El-Mihilmy and Tedesco (2001) modified and simplified Robert's expressions to account for the non-linearities that exist at the concrete-adhesive interface and developed expressions for calculating the interfacial normal stress directly from interfacial shear stress (Equation (3.21) and Equation (3.22)).

$$\sigma_x = 1.3(\alpha_f)^{\frac{1}{2}} \tau \quad (3.21)$$

$$\alpha_f = \sqrt{\frac{G_a t_f}{t_a E_f}} \quad (3.22)$$

Elastic normal stress that is perpendicular to the beam's cross section is mainly responsible for flexural cracking. This normal stress is also responsible for increasing the principle stress that causes diagonal cracks at plate ends. Concrete cover separation is believed to be accelerated by such diagonal cracks. This elastic normal stress can be

estimated using classical bending theory from the external moment at the plate ends (Equation (3.23)).

$$\sigma_x = \frac{M}{I} (d_f - c) \quad (3.23)$$

Finally the principle stresses, σ_1 and σ_2 , can be calculated using the Mohr theory of stress transformation from elastic normal stress, σ_x , interfacial normal stress σ_y and interfacial shear stress τ_{xy} . The major principle stress, σ_1 is responsible for causing diagonal crack while the minor principle stress, σ_2 reduces the uniaxial tensile strength of concrete through biaxial action of these stress. The position of elastic normal stress, σ_x , interfacial normal stress σ_y and interfacial shear stress τ_{xy} are shown in Figure 3.21. The expression for the principle stresses are given in Equation (3.24)

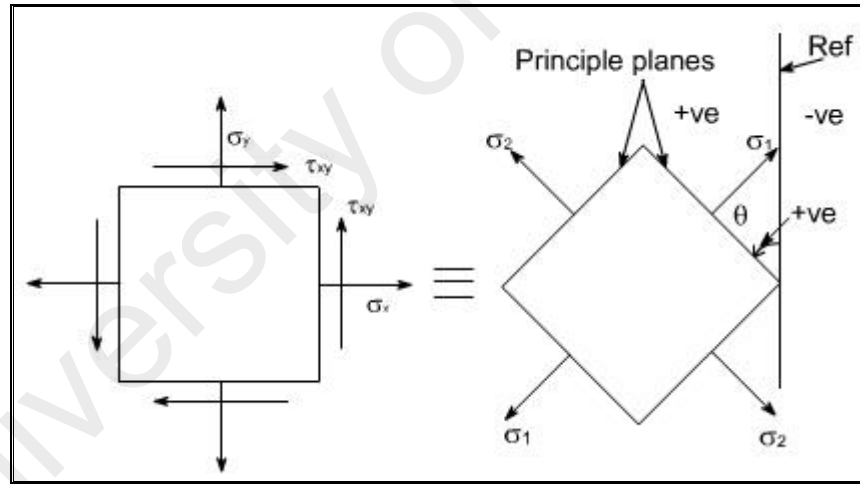


Figure 3.21: The principle and interfacial stress

$$\sigma_1, \sigma_2 = \left(\frac{\sigma_x + \sigma_y}{2} \right) \mp \sqrt{\left(\frac{\sigma_x - \sigma_y}{2} \right)^2 + \tau_{xy}^2} \quad (3.24)$$

3.3.5.2 Failure Criteria for Debonding Failure

The concrete at the strengthening plate in a strengthened beam is under a state of combined shear, τ and biaxial tensile stresses σ_x , and σ_y which result from the combination

of beam flexural and plate-peeling stresses (Figure 3.22). Thus, the two-dimensional principle stress state in the concrete is usually either tension-tension or tension-compression, depending on the magnitude of the shear stress. Figure 3.22 shows the typical biaxial failure criteria for concrete.

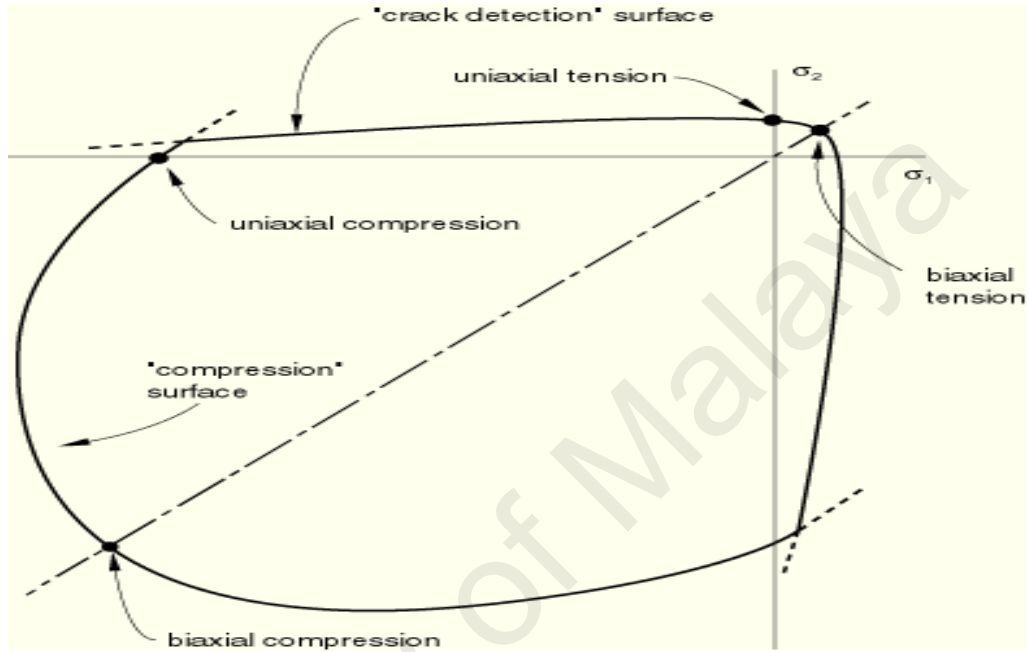


Figure 3.22: Typical biaxial failure criteria for concrete (Tysmans et al., 2015)

$$f_{tu} = f_t + \frac{-\sigma_2}{f'_c} \quad (3.25)$$

When the principle stress, σ_1 becomes greater than the bi-axially applied reduced tensile strength of concrete (f_{tu}), the diagonal cracks occur at the plate end. This diagonal cracking subsequently initiates and further accelerates the concrete separation process. The above debonding prediction model directly considers elastic shear and normal stress. It is therefore completely based on theoretical concept. It can consider and distinguish both end delamination and concrete cover separation.

3.4 Finite Element Modelling

3.4.1 Introduction

The finite element method is a useful technique in solving highly non-linear problems in continuum mechanics as reinforced concrete structures exhibit highly non-linear behaviour, especially approaching failure load. Numerical models have been developed using the ABAQUS program to predict the load deflection behaviour of reinforced concrete beams strengthened by FRP applied at the bottom of the beams. For many structural material, such as steel and aluminium which have well-defined constitutive properties, this finite element method works very well. However, when the constitutive behaviour is not so straight forward, like concrete in which discrete cracking occurs, the task is more difficult. The objective of this part of the study is to establish a reliable, convenient and accurate methodology for analysing FRP strengthened RC beams which can correctly represent global beam behaviour.

One of the advantages of finite-element models is to capture quantities that are virtually impossible to measure experimentally. In addition, they provide insight effects of micro- and macro-cracking on the interfacial behaviour and they allow us to obtain better results which may vary significantly from researcher to researcher, such as FRP strain.

As a part of the present study, experiments were performed on strengthened RC beams to investigate the flexural behaviour and to determine the ultimate failure load. The beam is subjected to two-point quasi-static load up to failure. The material properties and their constitutive modelling, analysis approach, verification of the finite element model, and special modelling considerations and modifications are outlined in this section. A series of RC beams strengthened with steel and FRP plates at the bottom were tested to failure under a four-point bending load. By comparing numerical results with experimental ones,

the proposed finite element model has been validated and can be used for further prediction of this type of failure.

3.4.2 Material Properties and their Constitutive Model

The materials used in the model engage steel reinforcing bars, concrete, steel and FRP plates. Reliable constitutive models related to steel reinforcing bars, steel plates, and concrete are obtainable in the ABAQUS material library. Thus, their input properties and related constitutive models are briefly discussed.

3.4.2.1 Concrete

Development of a constitutive model to simulate the behaviour of concrete is a challenging task. Concrete is a semi-brittle material and exhibit different performance in compression and tension. Concrete was modelled using a solid element with eight nodes and with three translation degrees of freedom at each node. The concrete solid element in the ABAQUS model is called 'C3D8R'. The concrete has an uni-axial compressive strength (f_c') selected as 30 MPa according to the experimental result. Under uni-axial compression, the concrete strain (ϵ_0) corresponding to the ultimate compressive stress (f_c') is usually around the range of 0.002 to 0.003. A representative value recommended by ACI Committee 318 and used in the analysis is $\epsilon_0 = 0.003$. The value for Poisson's ratio, $\nu = 0.2$ was used for the isotropic inelastic stages. The concrete damaged plasticity model (CDP) was used for defining concrete material behaviour in the inelastic range. The main failure mechanisms of concrete in CDP include: (1) tensile cracking, and (2) compressive crushing of the concrete.

The compressive stress-strain behaviour of concrete is simulated using a uniaxial non-linear constitutive model. The program computes the concrete compressive stress-strain curve based on the input of stress versus inelastic strain. The concrete behaviour under axial tension is assumed to be linear until the formation of the initial cracking at the peak

stress known as failure stress. Post failure stress is defined in the program in terms of stress versus cracking strain. This behaviour allows for the effect of the interaction between the concrete and the reinforcement rebar through introducing tension stiffening to the softening side of the curve.

3.4.2.2 Reinforcement

A classical metal plasticity model is applied for the non-linear material effects of steel reinforcement cast in concrete. Incremental theory is used in the plasticity model to relate load, deformation and stress non-linearity, once yielding has occurred. For an arbitrary load history, the final state of stress and deformation can be determined by accounting for the history of stress and strain. The history is taken into account by formulations that relate increments of stress to increments of strain.

An elastic-perfectly plastic material was used for steel with an equal behaviour in tension and compression. The steel reinforcement used in the beam is assumed to have the yielding stress of 500 MPa while its modulus of elasticity is assumed to be E_s 200 GPa. The stress-strain curve of the reinforcing bar is assumed to be elastic-perfectly plastic as shown in Figure 3.19. The steel reinforcement has a Poisson's ratio of 0.3. Perfect bonding between the steel and the concrete is presumed. The embedded element option was used for connecting the reinforcement element to the concrete element, and steel reinforcement was used as the embedded element while concrete was used as the host element.

3.4.2.3 Carbon Fiber Reinforced Polymer

The CFRP is designated as a linear elastic orthotropic material, because the composite is unidirectional and the behaviour is essentially orthotropic. The uniaxial behaviour of the FRP composites used in this study is assumed to be linear-elastic until failure with no

post-peak or ductile behaviour. Failure of these materials is occurred when the strain, ε_{pu} reaches to its rupture stress, f_{pu} as shown in Figure 3.23. Since the FRP is used primarily to carry tensile forces, it has stiffness in only one direction (along the fibres), thus no lateral and shear resistance is observed. Because the fiber reinforced plastics are relatively thin compared to the concrete beam, they are modeled by the 4-node shell elements (six degrees of freedom per node). The FRP shell elements are attached to the bottom surface of the concrete beam directly.

The modulus in the fibre direction is a significant factor, because the composite is mainly stressed in the fibre direction. The experimental value of 165 GPa is assigned for the elastic modulus in the fibre direction where the unidirectional CFRP material is used in the experimental study. This modulus of elasticity was specified by the model. For CFRP-concrete interface, full bond assumption was made for the interaction between FRP and concrete surfaces.

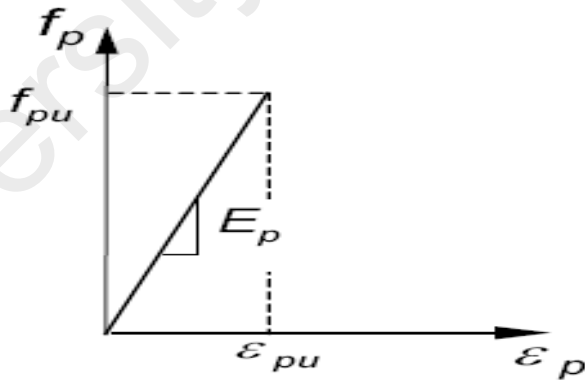


Figure 3.23: Stress-strain diagram of CFRP

3.4.3 Boundary Conditions

The boundary conditions were set in the model to mimic the experimental test conditions. One end of the beam was restrained in three degrees of freedom in the U_x , U_y , and U_z , directions, representing hinge support. In this scenario, the support was

allowed to rotate in every direction. The other end of the beam in the model was assumed as a roller support that is restrained in U_y .

3.4.4 Loads on RC Beams

In order to incorporate gravity and lateral loads in the finite element (FE) model, two steps were defined in the FE simulation. The gravity load was simulated in the first step as uniform pressure applied at the top of the beam. Load step sizes were automated by ABAQUS.

3.4.5 Discretization

The structural member is broken down into finite elements to model the composite beam. Since more than one type of material and interface is considered in the analysis, different types of elements are required to discretize the structure. The structural member is modelled as a mesh of finite elements. A wide range of elements are available in ABAQUS. Among these, continuum elements are the most comprehensive as they can be used in almost any linear/non-linear stress-displacement and crack propagation analysis. Both two- and three-dimensional (2D and 3D) continuum elements are available however, 2D continuum elements can adequately investigate the behaviour of the beams in this research. The 2D elements can be either triangular (3 or 6 nodes) or quadrilateral (4 or 8 nodes).

The concrete is modelled using continuum elements. Continuum elements are provided with first-order (linear) and second-order (quadratic) interpolation and careful consideration must be used as to which is more appropriate for the application. First-order elements use linear interpolation to obtain displacements at nodes, whereas second-order elements use quadratic interpolation to obtain displacements at nodes. ABAQUS offers two integration options.

Linear reduced-integration continuum elements are employed throughout the analysis with a fine mesh for their ability to withstand severe distortion in plasticity and crack propagation applications. All the elements in the model were purposely assigned the same mesh size to ensure that two different materials each share the same node. The type of mesh selected in the model was structured. The mesh element for the concrete, rebar and FRP laminate element were 3D solid, 2D truss and shell, respectively.

3.4.6 Finite Element Procedure

Displacement-controlled finite element methods are commonly applied in structural analysis and result in a system of equations corresponding unknown nodal deformation to specified loads by the stiffness matrix. Based on the calculated displacements, stresses and strains are computed. The equations engaged are derived from suitable structural theory and satisfy the following equilibrium (relate stresses to applied forces), compatibility (strains to displacement), and constitutive (stresses to strains). Together, these relationships are used to form the displacement based FEA equations in the matrix form. Cracking has been modelled using predefined crack line in ABAQUS.

The matrix equation is then solved for the displacement vector. Solving the equations allows us to go directly from forces to displacements. Strains and stresses are then computed from the displacement results. Shape functions are used to describe displacements. They are created through the use of Lagrangian interpolation to perform the necessary function of relating local coordinate position to global coordinate position. Once the displacements are calculated, they can be related to the strains within the element. The determination of strain requires partial differentiation of the displacement function with respect to the global coordinates.

3.5 Mathematical Optimization

“Optimal” means the most economical solution (Kasperkiewicz, 1995). Optimization is the act of estimating the best results under certain circumstances. In the design, construction and maintenance of any system, several decisions take place. The ultimate objective of these decisions is to either minimize cost or to maximize the required benefit. To optimally solve engineering problems, it is essential to convert design problems into optimization formulations, including objective functions and constraint functions. Optimization procedures try to seek the ‘best’ solutions for a desired objective function, $f(x)$, while satisfying the prevailing constraints. Maximization can be easily converted into a minimization problem since the maximization of $f(x)$ is equivalent to the minimization of $-f(x)$ (Perera & Varona, 2009).

3.5.1 Algorithm for Optimum Design Solution

Optimization of RC beams and their strengthening systems involves choosing design parameters in such a way that cost is minimized, while behavioural and geometrical constraints as recommended by design codes are also satisfied (Saini et al., 2007). In operational research, methods to find optimum solutions, such as mathematical programming techniques, are often studied. Mathematical programming methods are helpful in finding the minimum function of a number of variables under a certain set of constraints. The optimization tasks often uses mathematical maximization or minimization of an objective function $f(x_i)$ of n design variables, x_i , subjected to m equality constraints, g_j , and n inequality constants, h_k . In more realistic terms, optimization refers to finding the best possible arrangement for a given problem. Presented below is a formulation of the needed objective function.

In this study, the objective function is the total cost of the strengthening system subjected to applied force. The behavioural constraints are the requirements for flexural

strength and serviceability, while the geometrical constraints can be the upper limits on beam arising from practical considerations. It is now essential to search for a configuration characterized by a minimum price, which yet complies with all selected allowable strength and serviceability limits.

3.5.2 Objective Function

The problem of optimization largely depends on the type and nature of the objective function. The selection of the objective function thus has a significant influence on the optimization problem. This function is utilized to demonstrate a measure of how the different variables have performed in the problem domain. In the case of a minimization problem, the best solution will have the smallest possible numerical value of the related objective function (Perera & Varona, 2009).

Objective functions also form hyper-surfaces. When the objective function surfaces are illustrated along with the constraint surfaces on the design space, the optimum location can be easily predicted graphically as shown below in Figure 3.24.

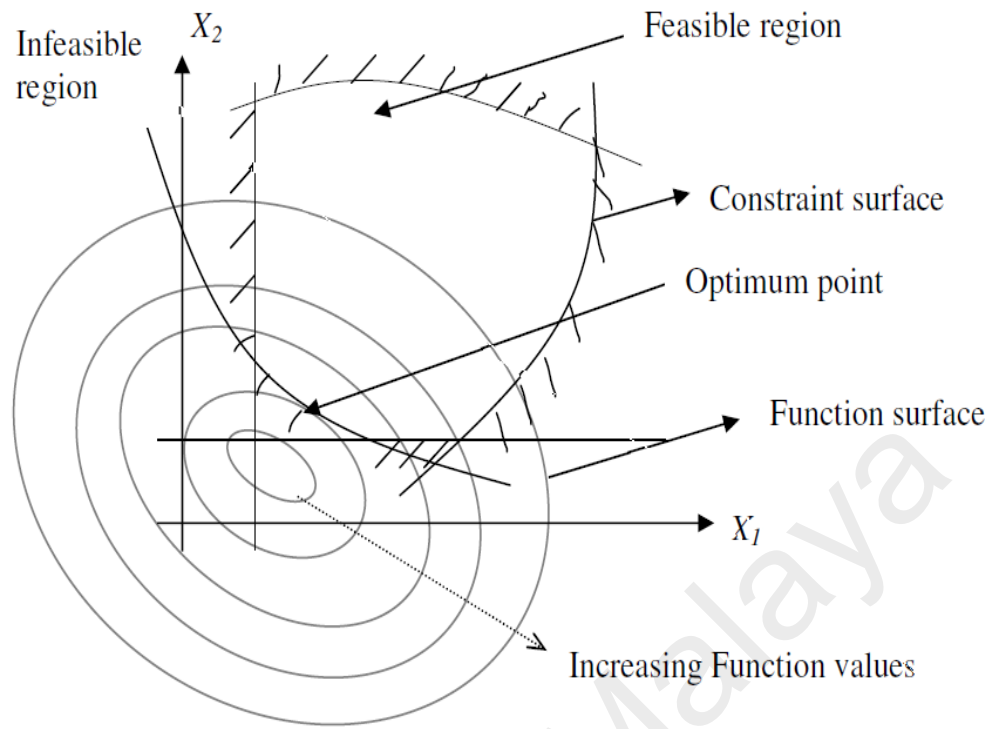


Figure 3.24: Function plot depicting optimum for a two design variable set
(Menon, 2005)

The optimization of FRP strengthened RC beams can be formulated by using total material cost as an objective function. The cost of FRP strengthening systems depends not only on the volume or weight of FRP material used, but also on the amount of adhesive used. Hence, the design variables are the dimensions of FRP materials and the quantity of adhesive. These variables can be changed to minimize the cost of the strengthening system.

The mathematical form of the cost function i.e. the objective function C (Equation 3.25) for the design of FRP strengthened RC beam is as follows:

$$C = C_f b_f t_f L_f + C_a b_f L_f \quad (3.26)$$

where:

C = the cost of the FRP strengthening system,

C_f = the unit price of the FRP plate,

C_a = the unit price of the epoxy adhesive,

b_f = the width of the FRP plate,

t_f = the thickness of the FRP plate, and

L_f = the length of the FRP plate.

For this study, it can reasonably be assumed that the unit cost of the CFRP plate is RM 1.70 per cubic centimetre and the unit price of epoxy adhesive is RM 1.00 per cubic centimetre.

3.5.3 Design Constraints

In structural optimization problems, technical performance and practical limitations are satisfied through the application of constraint functions. Constraints reduce the extent of the design space to be searched in accordance with the objectives that have to be achieved. Constraints include flexural constraints and serviceability constraints.

3.5.3.1 Flexural Constraints

The design guidelines proposed by the concrete society, TR55, on the flexural strengthening of RC beams with FRP are formulated in the optimization problem through constraint functions, g_i . Flexural resistance of the FRP strengthened RC beam must be greater than the external moments caused by the applied loads. These constraints provide acceptable levels of safety against ultimate limit states. The design loading and the design strength of the materials are required to evaluate these limit states. This constraint is presented in the following form (Equation 3.26).

$$M \leq M_r \quad (3.27)$$

where:

M = the design ultimate moment of the strengthened section, and

M_r = the resisting bending moment of the strengthened section.

The resisting bending moment of FRP strengthened RC beams can be estimated by using load factors and safety factors as required by the TR 55 guidelines, based on strain compatibility, internal force equilibrium and by taking into account the failure mode of the strengthened beam. The resisting bending moment of the strengthened section of a singly reinforced beam can be calculated by the Equation (3.28) which is given below. The balanced resisting moment of the strengthened beam, $M_{r,b}$, can be calculated by using Equation (3.29) and Figure 3.25, if the design ultimate moment exceeds the balanced resisting moment.

$$M_r = F_s z + F_f(z + (h - d)) \quad (3.28)$$

$$M_{r,b} = \left(\frac{0.67 f_{cu}}{\gamma_{mc}} \right) b \times 0.9(z + (h - d)) \quad (3.29)$$

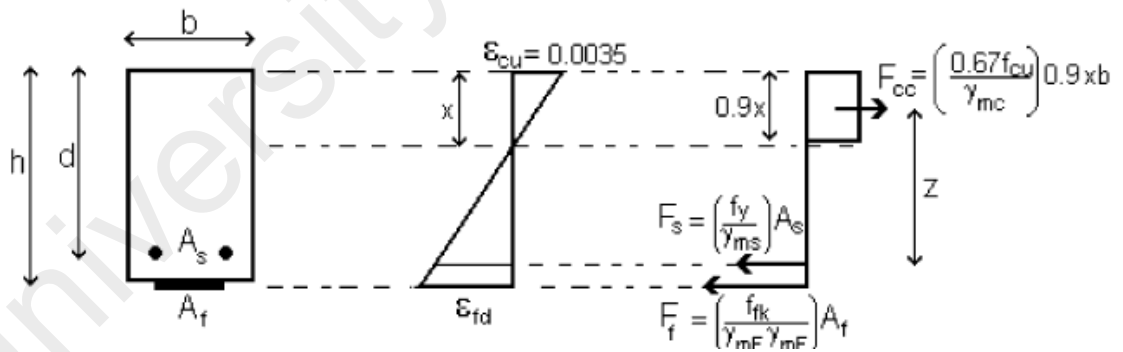


Figure 3.25: Stress and strain distribution of balanced failure

where:

$x = h (\epsilon_{fu} / \epsilon_{cu} + 1) =$ depth of neutral axis,

$h =$ over all depth,

$\epsilon_{fu} =$ design ultimate failure strain of FRP $= \epsilon_{fk} / \gamma_{mF}$,

$\epsilon_{fk} =$ ultimate failure strain of FRP,

$\gamma_{mF} =$ factor of safety against ultimate failure strain of FRP,

ϵ_{cu} = ultimate strain of concrete=0.0035,

d = effective depth,

z = lever arm,

F_s = force on steel reinforcement = $(f_y/\gamma_{ms}) A_s$,

f_y = yield strength of steel,

γ_{ms} = factor of safety against steel yield strength,

A_s = area of steel, Force acting on FRP, $F_f = f_f A_f$,

$A_f = t_f b_f$, stress in FRP, $f_f = E_{fd}(\epsilon_{cft} - \epsilon_{cit})$, design modulus of FRP,

$E_{fd} = E_{fk}/\gamma_{mE}$,

E_{fk} = modulus of elasticity of FRP,

γ_{mE} = factor of safety against modulus of FRP,

ϵ_{cft} = final strain of FRP = $\epsilon_{cu}(h-x)/x$, and

ϵ_{cit} = initial strain before strengthening.

The design ultimate moment of a simply supported beam with uniformly distributed load and a clear span l is calculated as follows:

$$M = \frac{wl^2}{8} \quad (3.30)$$

3.5.3.2 The Constraints against Separation Failure

The RC beam strengthened externally with FRP can fail prematurely due to separation of the FRP plate. There are two different types of failure mechanisms: peeling and debonding. This is still a controversial subject among the researchers and a lot of research is going on to develop a precise method to avoid premature plate separation.

Separation due to peeling usually occurs at the ends of the FRP plate due to the abrupt termination of the plate. The shear stresses and normal stresses are concentrated in the adhesive layer due to the deformation of FRP plate under applied loads. A number of factors affect the magnitude of these shear and normal stresses. Generally, end peeling

can be prevented by limiting the magnitude of the longitudinal shear stress and extending the FRP plate beyond the theoretical cut-off point. According to field experience in FRP installation, the longitudinal shear stress should be limited to 0.8 N/mm^2 . The longitudinal shear stress, τ , can be calculated using the Equation (3.31):

$$\tau = \frac{V \alpha_f A_f (h - x)}{I_{cs} b_a} \quad (3.31)$$

where:

V = ultimate shear force,

$\alpha_f = E_{fd}/E_c$, short term modular ratio,

E_{fd} = modulus elasticity of FRP,

E_c = modulus elasticity of concrete,

b_a = width of adhesive layer which is normally equal to width of beam, b_w , and

I_{cs} = second moments of area of strengthened concrete equivalent crack section.

Regarding the extension of FRP plates, Neubauer and Rostasy (1997) proposed a simple model that is accepted in the TR55 guideline. The maximum ultimate bond force, T_k and the corresponding maximum anchorage length, $l_{t,max}$ that are needed to activate this bond force can be calculated using the Equation (3.32), Equation (3.33) , Equation (3.34):

$$\tau_{k, \max} = 0.5 k_b b_f \sqrt{E_{fd} t_f f_{ctm}} \quad (3.32)$$

$$l_{y,max} = 0.7 \sqrt{\frac{E_{fd} t_f}{f_{ctm}}} \quad (3.33)$$

$$k_b = 1.06 \sqrt{\left(\frac{2 - \frac{b_f}{b_w}}{1 + \frac{b_f}{400}} \right)} \geq 1 \quad (3.34)$$

where:

b_w = beam width, and

$$f_{ctm} = 0.18(f_{cu})^{\frac{2}{3}} \quad (3.35)$$

It is also recommended that, where the FRP is terminated in the span, a minimum anchorage length of 500 mm should be provided. Debonding failure which normally occurs away from the plate end can be prevented by limiting the strain in the FRP to 0.8% for uniformly distributed loading and to 0.6% when there is a combination of high shear forces and bending moment.

3.5.3.3 Serviceability Constraints

Serviceability constraints are formulated in terms of limits on the steel reinforcement and concrete stress. TR 55 requires that the stresses in the steel reinforcement and concrete at working loads should not exceed $0.8f_y$ and $0.6f_{cu}$, respectively in order to avoid excessive deformation of the structure. The material stress can be calculated using the elastic principle. The equivalent transformed section for long term loading has to be determined by making an assumption that modular ratios of steel to concrete, α_e and FRP to concrete, α_f can be calculated using the following formula.

$$\alpha_e = \frac{E_s}{0.5 E_c} \quad (3.36)$$

$$\alpha_f = \frac{E_{fd}}{0.5 E_c} \quad (3.37)$$

3.5.4 Application of Optimization Method

3.5.4.1 Non-linear Programming

Non-linear problems can be solved using several methods. A model in which the objective function and all of the constraints (other than integer constraints) are smooth non-linear functions of the decision variables is called a non-linear programming (NLP) or non-linear optimization problem. Such problems are intrinsically more difficult to solve than linear programming (LP) problems. They may be convex or non-convex, and an NLP solver must compute or approximate derivatives of the problem functions many times during the course of the optimization process. Since a non-convex NLP may have multiple feasible regions and multiple locally optimal points within such regions, there is no simple or fast way to determine with certainty that the problem is infeasible, that the objective function is unbounded, or that an optimal solution is the “global optimum” across all feasible regions.

The Non-linear Solving method uses the Generalized Reduced Gradient (GRG) method as implemented in Lasdon and Waren’s GRG2 code. The GRG method can be viewed as a non-linear extension of the Simplex method, which selects a basis, determines a search direction, and performs a line search on each major iteration – solving systems of non-linear equations at each step to maintain feasibility. The reduced gradient method also known as the ‘Frank and Wolfe’ algorithm, is an iterative method for non-linear programming. Other methods for non-linear optimization include Sequential Quadratic Programming (SQP) and Interior Point or Barrier methods.

3.5.4.2 Genetic Algorithm

A simple genetic algorithm was applied to solve the problem of the optimization of FRP strengthened RC beams using a continuous search space. Genetic algorithms cannot handle constraints explicitly. Therefore, it is essential to transform all constraints into

penalty functions. In using genetic algorithms, a number of genetic operations like generation, selection, crossover and mutation are performed.

Generation is an operation that creates a population of candidate solutions as a starting point which is usually random. The population size used in this study was 100. Among the three selection operators, tournament was applied in this application. Crossover and mutation make the genetic algorithm more powerful. Crossover forms a new chromosome from two parental chromosomes by a reproduction operation. In this case, single point crossover was used. Mutation creates diversity among the population by changing a gene. The mutation rate used in this study was 0.05. The optimization problem of this study was solved by applying a simple genetic algorithm using the previously mentioned parameters.

CHAPTER 4: RESULTS AND DISCUSSION

4.1 Introduction

This chapter presents the results of this study and discusses the results. Section 4.2 presents the results and discussion of experimental investigation. The results include strength, deformation, damage and failure characteristic of the specimens. Another purpose of experimental investigation on the NSM technique is to compare it with HSM. Some parametric studies were done but they were not carried out primarily for investigating the effect of different parameters, rather they tried to identify the most suitable configuration to achieve the best performance. A brief fatigue behaviour is also discussed in this section. Verification of semi-numerical and finite element modelling is discussed in Sections 4.3 and 4.4, respectively. Section 4.5 provides example solutions of the mathematical optimization technique in a structural strengthening system.

4.2 Result of Experimental Investigation

The extensive data obtained from the experimental investigation are presented in this section. Sub-section 4.2.1 presents the test data of the properties of the materials used for the preparation of beam specimens. The achievement of experimental research objectives is discussed in sub-section 4.2.2 to sub-section 4.2.6.

4.2.1 Material Properties

The average concrete cube strength of all tested beams was 29.35 MPa and the modulus of rupture shown was found to be 3.85 MPa. Test results showed some slight variations in the concrete strengths and modulus of rupture although they were cast from the same mix design. The average concrete cube strengths and modulus of rupture of each tested beam was given in Appendix A. The measured yield and ultimate tensile strengths of the 6, 8, 10, 12 mm steel bars were shown in Table 4.1. The modulus of elasticity for all the steel bars was 200 GPa. The test data obtained for 6 mm plain bars are shown in

Appendix A. The yield and ultimate tensile strength of the steel plates are 420 MPa and 475 MPa. The modulus of elasticity is 200 GPa.

Table 4.1: The properties of steel bar

Bar diameter (mm)	Yield strength (MPa)	Ultimate strength (MPa)	Modulus of Elasticity (GPa)
6	580	650	200
8	551	641	
10	520	572	
12	551	641	

4.2.2 Experimental Behaviour of Steel HSM Strengthened Beams

4.2.2.1 Load Carrying Capacity and Failure Mode

A summary of the flexural behaviour of all tested beams strengthened using HSM with steel bars and steel plates is shown in Table 4.2. The summary is given in terms of first crack load, yield load, flexural loading capacity and failure mode. As shown in Table 4.2, the addition of steel bars and steel plates increased the ultimate load capacity by 32% to 72% as compared to the control beam. On the other hand, in the previous study, the ultimate load of an EBR strengthened beam increased by 37% as compared to the control beam (Sena-Cruz et al., 2012). In this study, yield load of the beam also increased after strengthening. The yield load of the beam was sometimes not distinguished because of early debonding. The first crack load of strengthened beams increased most significantly as compared to the control beam. These results have proven the effectiveness of HSM to increase the flexural capacity in accordance with Objective (i).

Table 4.2: First crack, yield and failure (and modes) load of HSM-steel

Beam no	First crack load (kN)	Increase in first crack load (%)	Bar yield load (kN)	Failure load (kN)	Increase in failure load (%)	Mode of failure
CB	12.5	-	72	80.0	-	Flexural failure
H1B8S19L73W2T	40.0	220	120	132.0	65	Cover separation
H1B8S16L73W2T	58.0	364	100	105.6	32	Cover separation
H1B6S16L73W2T	60.0	380	90	102.0	27	Cover separation
H2B8S19L73W2T	48.0	284	100	108.3	35	Cover separation
H2B6S19L73W2T	30.0	140	90	109.0	37	Cover separation
H2B6S19L73W2.76T	40.0	220	-	130.0	63	Cover separation
H2B6S19L125W2T	62.0	396	-	115.0	44	Cover separation
H1B8SD19L73W2T	40.0	220	-	125.0	56	Cover separation
H1B8S19L73W2TAS	53.0	324	-	135.0	68	Flexural failure (concrete crushing)
H1B8S19L73W2TAF	40.0	220	-	137.0	71	Flexural failure (Concrete crushing)
H1B8S19L73W1.5T	40.0	220	-	137.3	72	Cover separation + concrete crushing
SH2B6S19L100W2T	60.0	380	-	123.0	53	Flexural failure (Concrete crushing)

The failure modes of all beams strengthened with HSM steel plates and bars are shown in Figure 4.1 to Figure 4.12. The failure modes of most of these beams were found to be very close to each other i.e. concrete cover separation initiated by a diagonal tension crack. Concrete cover separation is the most commonly reported mode of failure (Kang et al., 2012). This type of failure is generally demonstrated by a crack forming in the

concrete at or near the plate end, which spreads to the level of the tension reinforcement and then progresses horizontally, along the level of the reinforcement, resulting in separation of the concrete cover. The un-strengthened control beam, CB, failed, as expected, in flexure with extensive yielding of the tension steel, followed by crushing of the concrete in the compression zone.



Figure 4.1: Debonding failure mode of H1B8S19L73W2T



Figure 4.2: Debonding failure mode of H1B8S16L73W2T



Figure 4.3: Debonding failure mode of H1B6S16L73W2T



Figure 4.4: Debonding failure mode of H2B8S19L73W2T



Figure 4.5: Debonding failure mode of H2B6S19L73W2T



Figure 4.6: Debonding failure mode of H2B6S19L73W2.76T



Figure 4.7: Debonding failure mode of H2B6S19L125W2T

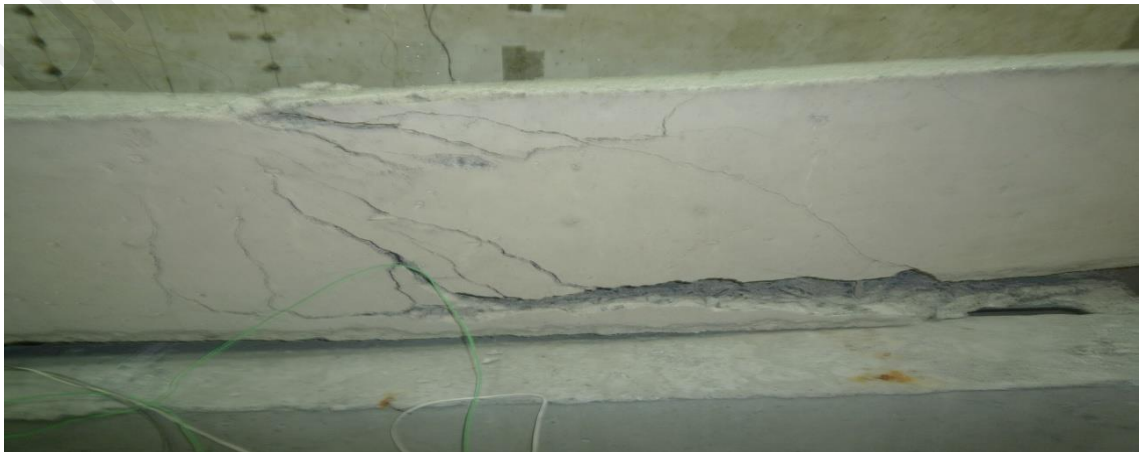


Figure 4.8: Debonding failure mode of H1B8SD19L73W2T



Figure 4.9: Flexure failure mode of H2B6S19L125W1.5T



Figure 4.10: Flexure failure mode of H1B8S19L73W2TAS



Figure 4.11: Flexure failure mode of H1B8S19L73W2TAF



Figure 4.12: Flexure failure mode of SH2B6S19L100W2T

4.2.2.2 Effect of Strengthening on Deflection and Cracking Behaviour

The deflection and reduction in deflection due to HSM strengthening at 20 kN, 40 kN, and 60 kN service loads are shown in Table 4.3. The deflection of the strengthened beams was reduced compared to the control beam due to increased stiffness.

Table 4.3: Reduction in deflection due to HSM strengthening

Beam No.	Load at 20 kN		Load at 40 kN		Load at 60 kN	
	Deflection in mm (LVDT)	Reduction (%) over CB	Deflection in mm (LVDT)	Reduction (%) over CB	Deflection in mm (LVDT)	Reduction (%) over CB
CB	1.34	-	4.34	-	6.92	-
H1B8S19L73W2T	1.00	25	2.47	43	4.07	41
H1B8S16L73W2T	0.72	46	1.84	58	2.97	57
H1B6S16L73W2T	1.30	3	2.46	42	4.74	32
H2B8S19L73W2T	1.26	6	2.26	48	3.14	55
H2B6S19L73W2T	1.26	6	2.48	43	3.43	50
H2B6S19L73W2.76T	1.18	12	2.12	51	3.00	57

The crack width increased with increased loading according to the data obtained. The first crack loads of the beams are shown in Table 4.2. The strengthened beams, H1B8S19L73W2T and H1B8S16L73W2T, had higher cracking load compared to that of the control beam.

4.2.2.3 Comparison of HSM with EBR using Steel Plates and Bars

In this section, the HSM strengthened beams (H1B8S19L73W2T and H1B8S16L73W2T) are compared to the corresponding EBR beams (PS19L73W2.76T and PS16L73W2.76T). The detailed experimental behaviour of these two HSM strengthened beams will be analyzed, interpreted and compared with the corresponding EBR beam. Other HSM strengthened beams will be used for parametric study.

(a) Effect of HSM strengthening on Ultimate Load

Figure 4.13 shows the effect of hybridization on the ultimate load of the strengthened RC beams. In both cases (with plate length 1900 mm and 1650 mm), the failure load of the HSM strengthened beams (H1B8S19L73W2T and H1B8S16L73W2T) was greater than that of the corresponding EBR beams (PS19L73W2.76T and PS16L73W2.76T). The amount of strengthening materials used was almost same (total cross-sectional area = 200 mm²) for HSM and corresponding EBR but improvement in HSM was significantly higher. Specifically, the improvement in the debonding failure loads of H1B8S19L73W2T and H1B8S16L73W2T was 27% and 24%, respectively as compared to the improvement of PS19L73W2.76T and PS16L73W2.76T. This improvement was achieved in two ways: 1) reduction of plate thickness, 2) increased surface area, as mentioned in the research background of chapter 1. The increase in surface area leads to improved performance, and this is demonstrated by comparing the failure load of H2B6S19L73W2.76T (130 kN) and PS19L73W2.76T (104 kN) with same plate thickness (2.73 mm). Though the plate thickness of H2B6S19L73W2.76T and PS19L73W2.76T are the same, the failure load of H2B6S19L73W2.76T (HSM) is 26 kN (25%) higher than that of the PS19L73W2.76T (EBR). Similar improved performance was found in the externally bonded reinforcement on grooves (EBROG) method, where only contact surface area increased (Mostofinejad & Shameli, 2013). It has further proved the effectiveness of HSM and it is more efficient than EBR.

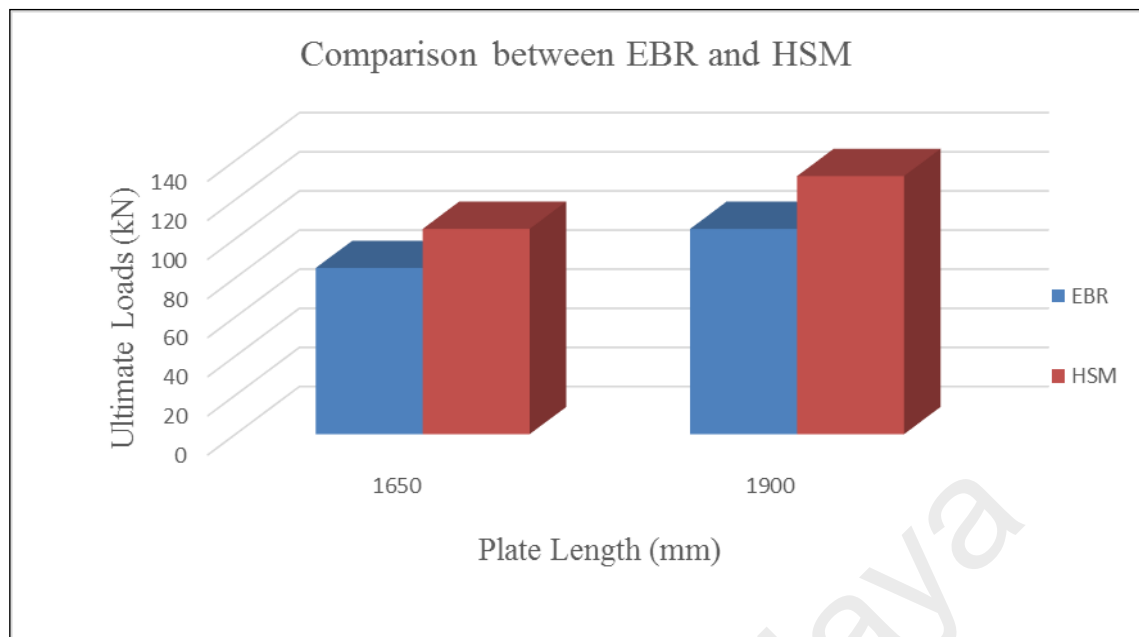


Figure 4.13: Comparison of failure load between HSM and EBR

(b) Deflection Characteristics

Deflection data was collected from both the LVDTs and the actuator position of the Instron Universal Testing machine. The LVDTs were removed immediately after failure initiation to avoid probable damage. Deflections calculated from the position of the actuator are also presented in the load deflection diagram in order to observe the actual deformability of the beams as far as possible. The load versus mid-span deflection curves of H1B8S19L73W2T, PS19L73W2.76T and the control beam, CB are shown in

Figure 4.14. The deflections of the strengthened beams were lower than that of the control beam as the stiffness of the strengthened beam increased due to the presence of the strengthening steel plate and steel bar. However, the deflection of the HSM strengthened beams, H1B8S19L73W2T and H1B8S16L73W2T, was almost similar to the EBR beams, PS19L73W2.76T and PS16L73W2.76T, because a similar amount of steel was used in strengthening. The load versus mid-span deflection curves of H1B8S16L73W2T, PS16L73W2.76T and the control beam, CB are shown in Figure 4.15.

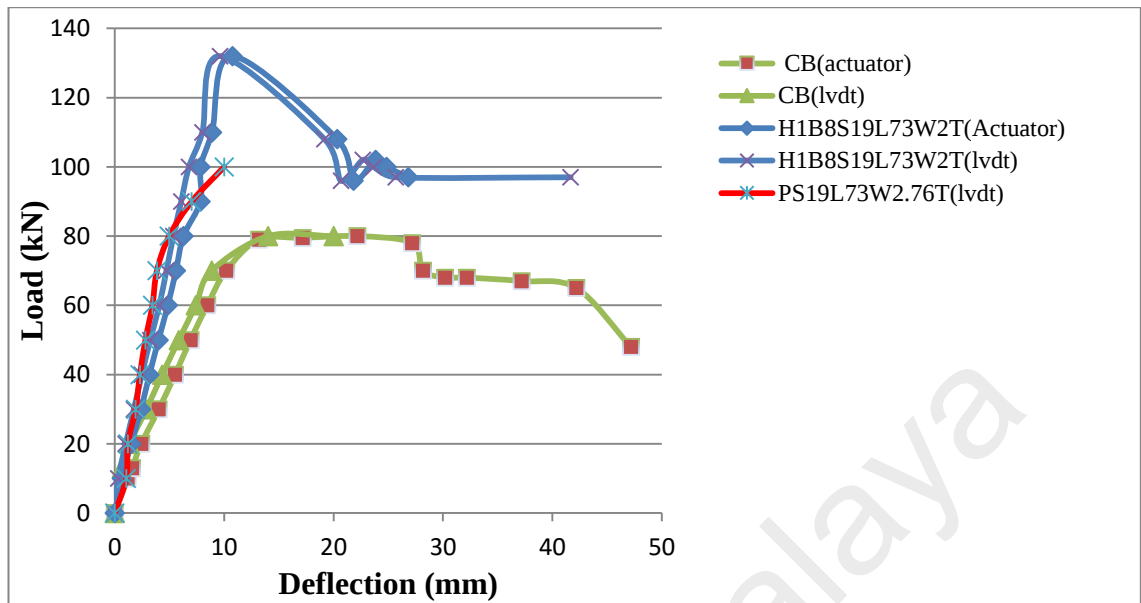


Figure 4.14 : Load-deflection of CB, H1B8S19L73W2T and PS19L73W2.76T.

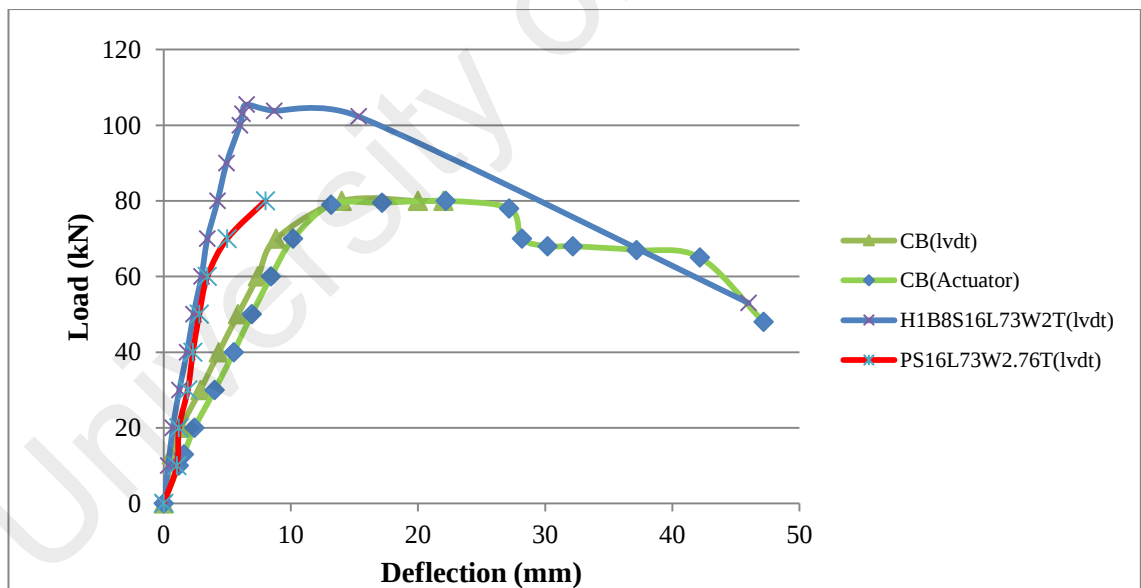


Figure 4.15: Load-deflection of CB, H1B8S16L73W2T and PS16L73W2.76T

According to the deflection data obtained from actuator positions, the deformability of the HSM strengthened beams was nearly similar to that of the control beam although the ultimate load decreased significantly. However, the maximum loads of the HSM

strengthened beams after debonding were even more than the maximum load of the control beam. The ductility index at ultimate load of the HSM strengthened beams ($15/12=1.25$) was almost similar to that ($12/10=1.2$) of control beam. This is another important advantages of HSM technique.

(c) Cracking Behaviour

The cracking behaviour of the HSM strengthened beam improved because of improved composite action through hybridization. Improvement in the first crack load of the HSM strengthening technique is shown in Figure 4.16. The first crack loads of H1B8S19L73W2T and H1B8S16L73W2T increased 14% and 65% over the EBR beams PS19L73W2.76T and PS16L73W2.76T, respectively.

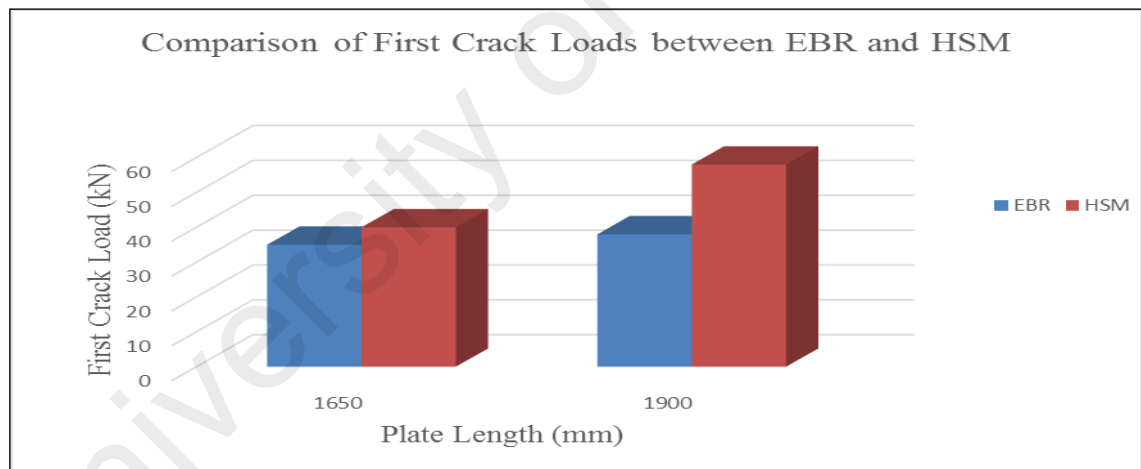


Figure 4.16: Improvement of first crack loading in HSM strengthening

(d) Internal Reinforcing Bar Strain

The strains in the internal reinforcing steel bars and the reduction in these strains due to strengthening at 20 kN, 40 kN, and 60 kN service loads are shown in Table 4.4. The bar strains in strengthened beams were significantly reduced. Consequently, according Hooke's law, the stress in the bars should also be reduced and therefore, the fatigue life

of the strengthened beams should increase according to the S-N curve relation of steel bars (Helagson & Hanson, 1974; Moss, 1982). However, differences in bar strain reduction between the HSM and the EBR were not noticed clearly.

Table 4.4: Bar strain at different service loads

Beam No.	Load at 20 kN		Load at 40 kN		Load at 60 kN	
	Bar Strain	Reduction (%)	Bar Strain	Reduction (%)	Bar Strain	Reduction (%)
CB	793	-	1661	-	2507	-
H1B8S19L73W2T	360	55	785	53	1021	59
H1B8S16L73W2T	299	62	583	65	708	72
PS19L73W2.76T	303	62	606	64	910	64
PS16L73W2.76T	485	39	970	42	1456	42

(e) Efficiency of HSM

Figure 4.17 shows the amount of steel required to strengthen concrete beams and the corresponding increase in load carrying capacity of beams strengthened with EBR and HSM. As shown in Figure 4.17, although HSM uses a smaller amount of steel, it increases the load carrying capacity of the beam by 65% as compare to control beam. On the other hand, the performance of the EBR only increases by 30%.

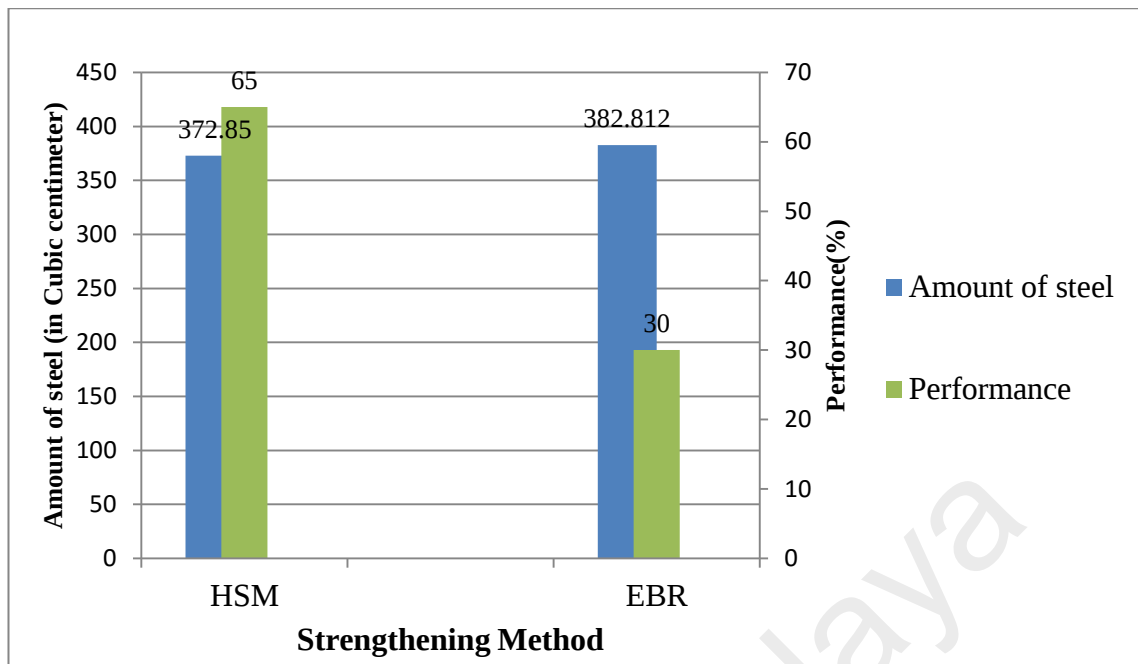


Figure 4.17: Efficiency of the HSM

4.2.2.4 Effect of Plate and Bar Length, Bar Dia. and No. of Grooves

Improvement in the flexural capacity of the strengthened beams depends on various parameters. Therefore, it is very important to take into account different factors that can have a major influence on the overall results. In this study, the effect of strengthening with different configurations was investigated. However, strengthening with different configurations was investigated mainly to identify optimum arrangements to achieve the best improvement in flexural performance of RC beams strengthened with steel plates and bars using the HSM. The effect of individual parameters on the performance was not a primary concern of this study.

(a) Effect of Plate and Bar Length

The plate and bar length influenced the flexural performance of the strengthened beam. The plate and bar length are always equal in all HSM strengthened beams. These lengths have influence flexural performance. The effect of plate and bar length on the performance of strengthened beams is shown in Figure 4.18. Based on the experimental

data of H1B8S19L73W2T, H1B8S16L73W2T, PS19L73W2.76T and PS16L73W2.76T, it can be said that increasing plate length results in increased failure loads. This is a common observation in studies on EBR strengthening. Increasing the plate length reduces the distance between the plate end and the support, which is an influential parameter in end debonding behaviour of externally bonded plates (Täljsten, 1997).



Figure 4.18: The effect of plate and bar length on failure load

(b) Effect of Bar Diameter

Bar diameter has a significant influence on the flexural resistance of normal RC beams. However, in NSM or HSM, increasing the strengthening bar diameter reduces the amount of adhesive and thereby may affect performance of the bond. Hence, it is important to investigate the influence of bar diameter on the strengthening performance. Figure 4.19 shows the effect of bar diameter on failure load. Data collected from beams H1B8S16L73W2T, H1B6S16L73W2T, H2B8S19L73W2T and H2B6S19L73W2T were

used to analyze this parameter. As shown in Figure 4.19, the failure load increased with increasing bar diameter when one bar was used in the NSM groove of the HSM strengthened beams (H1B8S16L73W2T and H1B6S16L73W2T). However, when two bars were used (H2B8S19L73W2T and H2B6S19L73W2T), the failure load decreased with increasing bar diameter. Thus, the effect of bar diameter on the strengthening performance of HSM beams is controversial. Although increasing the number of bars provides additional reinforcement to the concrete beam, it decreases both edge clearance and clear spacing between two adjacent grooves. This increases the possibility of edge breakage. The beam specimens used in this study may not have had enough width to place two bars with sufficient clearance. This may have led to accelerated concrete separation due to early edge breakage. Similar results were found in Bilotta et al. (2015)'s study where an increase in the number of grooves caused break down the concrete cover rapidly.

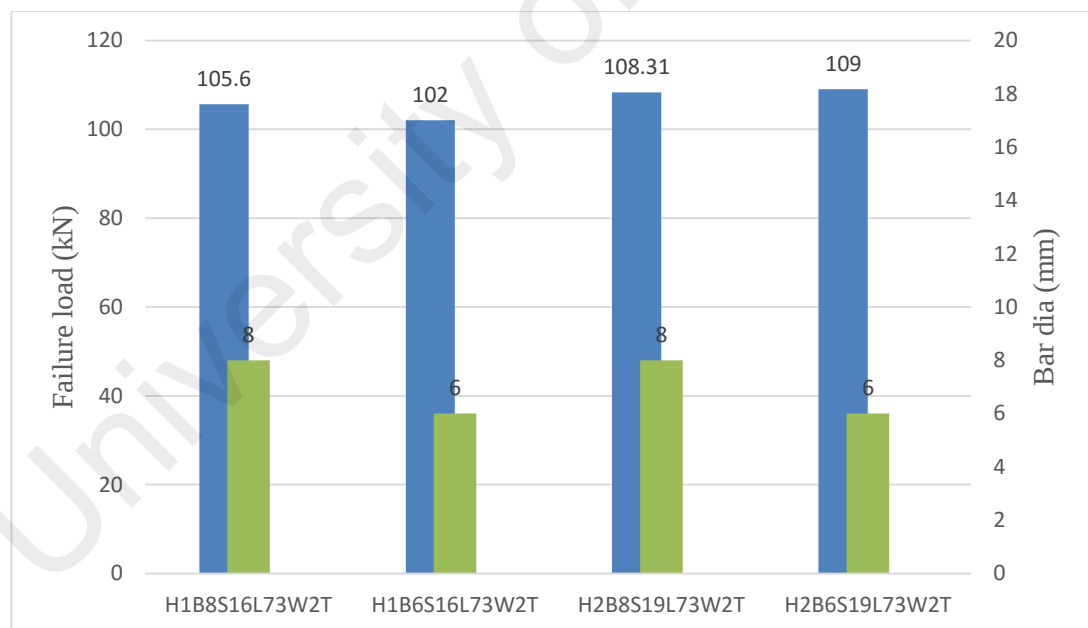


Figure 4.19: The effect of bar diameter

(c) *Effect of Number of Bars or NSM Grooves*

Each strengthening bar used requires a separate groove to place the bar in the RC beam. Thus, the number of bars used in strengthening is equal to the number of grooves. Figure 4.20 shows the effect of the number of grooves or bars on the performance of beams strengthened using the HSM. The experimental data of H1B8S19L73W2T and H2B8S19L73W2T were used to investigate this effect. As shown in Figure 4.20, failure load decreased when the number of bars or grooves increased. This issue has been discussed in the previous sub-section.

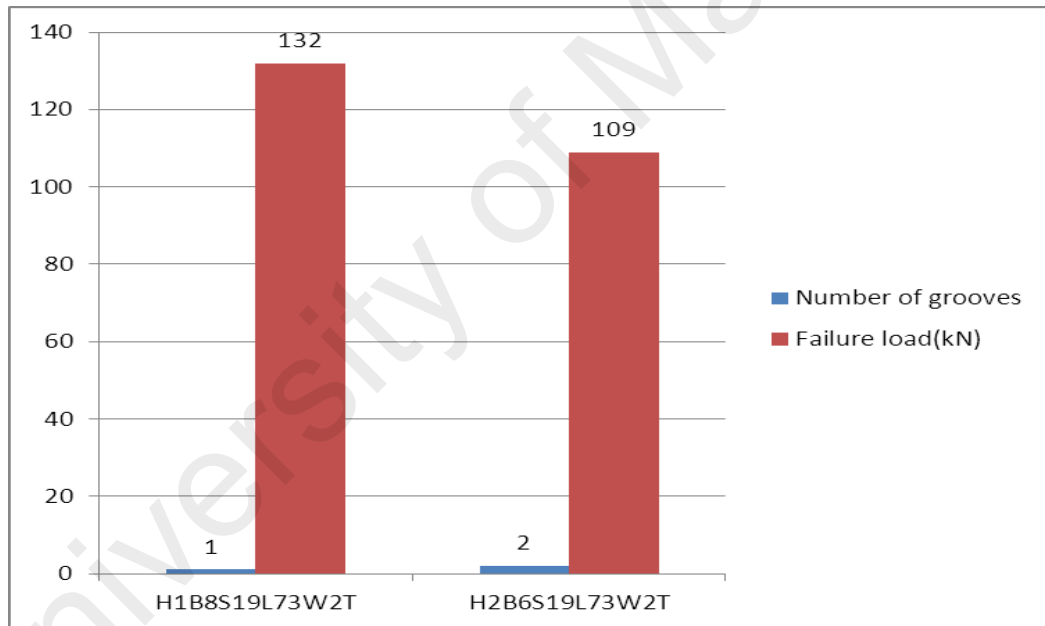


Figure 4.20: The effect of number of bars or grooves

4.2.3 Experimental Behaviour of CFRP-HSM Strengthened Beam

4.2.3.1 Load Carrying Capacity and Failure Mode

Table 4.5 summarizes the flexural behaviour of the tested beams strengthened using the HSM with CFRP in terms of first crack load, yield load, flexural loading capacity and failure mode. As shown in Table 4.5, the addition of steel bars and CFRP plates increased

the ultimate load capacity by 35% to 104% as compared to the control beam. The yield load of the beam also increased after strengthening. The first crack load of the strengthened beams increased significantly compared to the control beam. The addition of a steel bar and CFRP fabrics in the form of HSM also increased the load capacity by 43%.

Table 4.5: First crack, yield and failure (and modes) load of HSM-CFRP

Beam no	First crack load (kN)	Increase in first crack load (%)	Bar yield load	Failure load	Increase in failure load (%)	Mode of failure
CB	12.5	-	72	80	-	Flexural failure
H1B8F19L80W1.2T	30.0	146	120	133.0	66	Cover separation
H1B8F16L80W1.2T	35.0	187	100	129.8	62	Cover separation
H1BP8F16L80W1.2T	28.0	129	107	107.0	34	Cover separation
H1BP6F16L80W1.2T	40.0	233	100	120.0	50	Cover separation
H2BP6F16L80W1.2T	40.0	233	90	127.0	57	Cover separation
H1B8F19L80W1.2TAF	32.0	162	-	164.0	104	Flexural failure
H1B6FR19L100W.17T	27.0	126	-	114.0	43	Flexural failure

The failure modes of the beams in the above table are shown in Figure 4.21 to Figure 4.27. The failure modes of most of the beams strengthened using the HSM with CFRP were found to be very close to each other i.e. concrete cover separation initiated by a diagonal crack. Concrete cover separation is a commonly reported mode of failure. This type of failure is generally demonstrated by a crack forming in the concrete at or near the FRP plate end, propagating to the level of tension reinforcement and then progressing horizontally, along the level of the reinforcement, resulting in a separation of the concrete cover. Similar to the unstrengthened control beam, CB, which failed in flexure with extensive yielding of the tension steel, followed by crushing of the concrete in the

compression zone, the beams strengthened with CFRP fabrics and steel bars failed in the desirable flexural failure mode as shown in Figure 4.27.



Figure 4.21: Debonding failure mode of H1B8F19L80W1.2T



Figure 4.22: Debonding failure mode of H1B8F16L80W1.2T



Figure 4.23: Debonding failure mode of H1BP8F16L80W1.2T



Figure 4.24: Debonding failure mode of H1BP6F16L80W1.2T



Figure 4.25: Debonding failure mode of H2BP6F16L80W1.2T



Figure 4.26: Flexure failure mode of H1B8F19L80W1.2TAF



Figure 4.27: Flexure failure mode of H1B6FR19L100W.17T

4.2.3.2 Effect of Strengthening on Deflection and Cracking Behaviour

The deflection and reduction in deflection due to HSM at 20 kN, 40 kN, and 60 kN service loadings are shown in Table 4.6. The deflection of the strengthened beams was reduced compared to the control beam due to increased stiffness of the strengthened beams.

Table 4.6: Reduction in deflection due to HSM strengthening with FRP

Beam No.	Load at 20 kN		Load at 40 kN		Load at 60 kN	
	Deflection (mm)	Reduction (%)	Deflection (mm)	Reduction (%)	Deflection (mm)	Reduction (%)
CB	1.34	-	4.34	-	6.92	-
H1B8F19L80W1.2T	1.18	12	2.83	35	4.09	41
H1B8F16L80W1.2T	0.68	49	2.00	54	3.32	52
H1BP8F16L80W1.2T	0.86	36	2.14	51	3.51	49
H1BP6F16L80W1.2T	0.96	6	2.33	48	3.72	55
H2BP6F16L80W1.2T	1.26	6	2.48	43	3.43	50
H1B8F19L80W1.2TAF	0.83	38	2.83	35	4.09	41
H1B6FR19L100W.17T	1.17	14	3.32	23	6.71	3

The crack width increased with increased load. The first crack loads of all HSM CFRP beams are shown in table Table 4.6. The strengthened beams, H1B8F19L80W1.2T and H1B8F16L80W1.2T, showed higher cracking loads compared to that of the control beam.

4.2.3.3 Comparison of HSM with EBR

In this study, the HSM strengthened beams H1B8F19L80W1.2T and H1B8F16L80W1.2T, are comparable to the EBR beams, PF19L80W1.2T and PF19L80W1.2T6L80W1.2T. The detailed experimental behaviour of these two HSM strengthened beams will be analyzed, interpreted and compared with corresponding EBR beams. The other HSM strengthened beams will be used to investigate the effect of strengthening with different configurations.

(a) Effect of HSM Strengthening on Ultimate Load

Figure 4.28 shows the effect of hybridization with CFRP on the static failure performance of the strengthened RC beams. In both cases (with plate length 1900 mm and 1650 mm), failure loads of the HSM strengthened beam (H1B8F19L80W1.2T and H1B8F16L80W1.2T) were greater than that of the corresponding EBR beams (PF19L80W1.2T and PF19L80W1.2T6L80W1.2T). Specifically, the improvement of debonding failure loads of H1B8F19L80W1.2T and H1B8F16L80W1.2T are 9% and 36% more than that of PF19L80W1.2T and PF19L80W1.2T6L80W1.2T. This improvement was achieved by increasing the bonding surface area with the same plate thickness (1.2 mm).

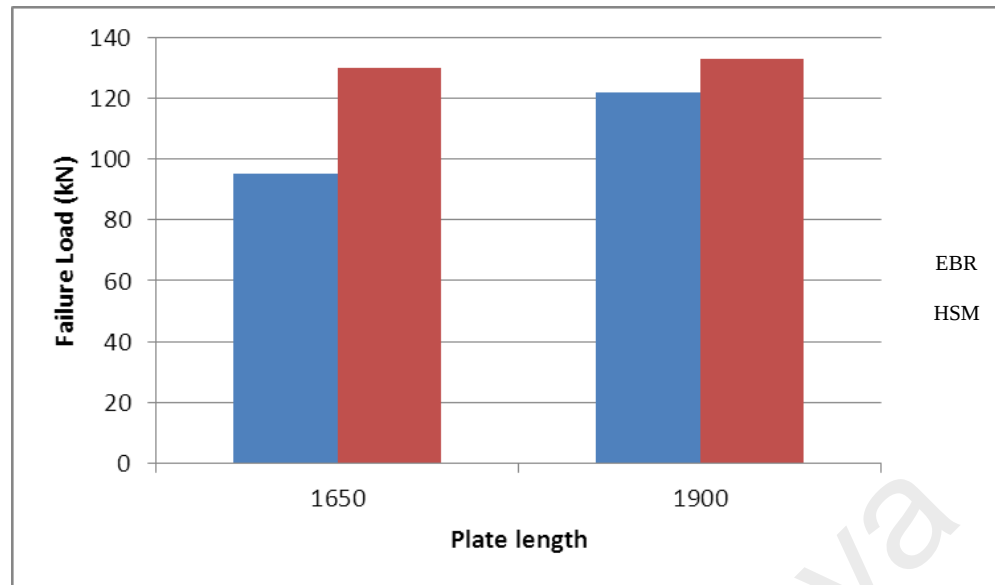


Figure 4.28: Comparison of Ultimate load between HSM and EBR

(b) Deflection Characteristics

The load versus mid-span deflection curves of H1B8F19L80W1.2T, PF19L80W1.2T and CB are shown in Figure 4.29. The deflections of the strengthened beams were lower than that of the control beam because the stiffness of the strengthened beam was greater than that of the control beam due to the presence of the strengthening CFRP plate and steel bar. However, the deflection of HSM strengthened beams, H1B8F19L80W1.2T and H1B8F16L80W1.2T, was similar to the EBR beams, PF19L80W1.2T and PF19L80W1.2T6L80W1.2T, perhaps due to similar amounts of strengthening material being used. The load versus mid-span deflection curves of H1B8F16L80W1.2T, PF19L80W1.2T6L80W1.2T and CB are shown in Figure 4.30.

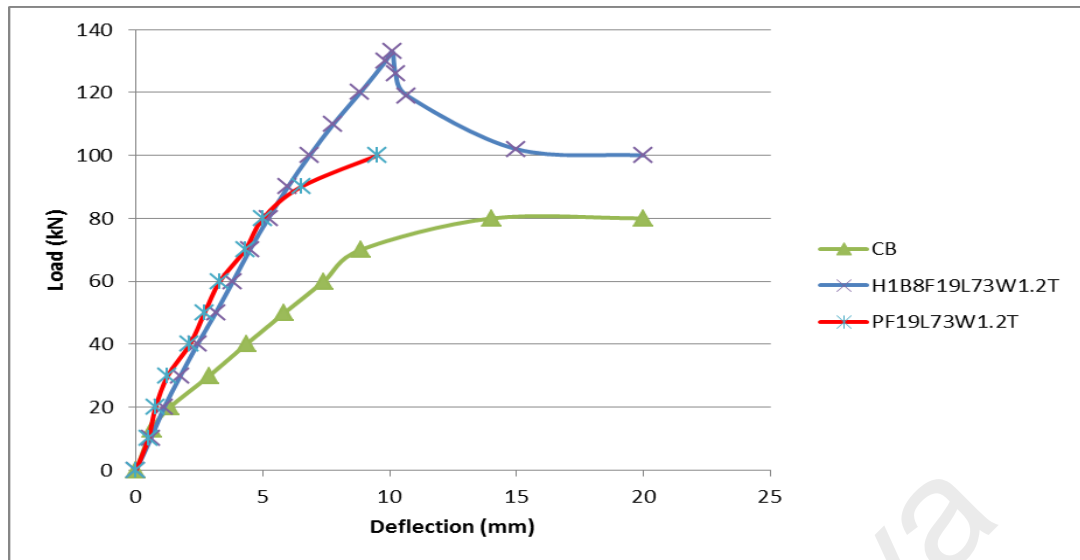


Figure 4.29: Load-deflection of CB, H1B8F19L80W1.2T and PF19L80W1.2T

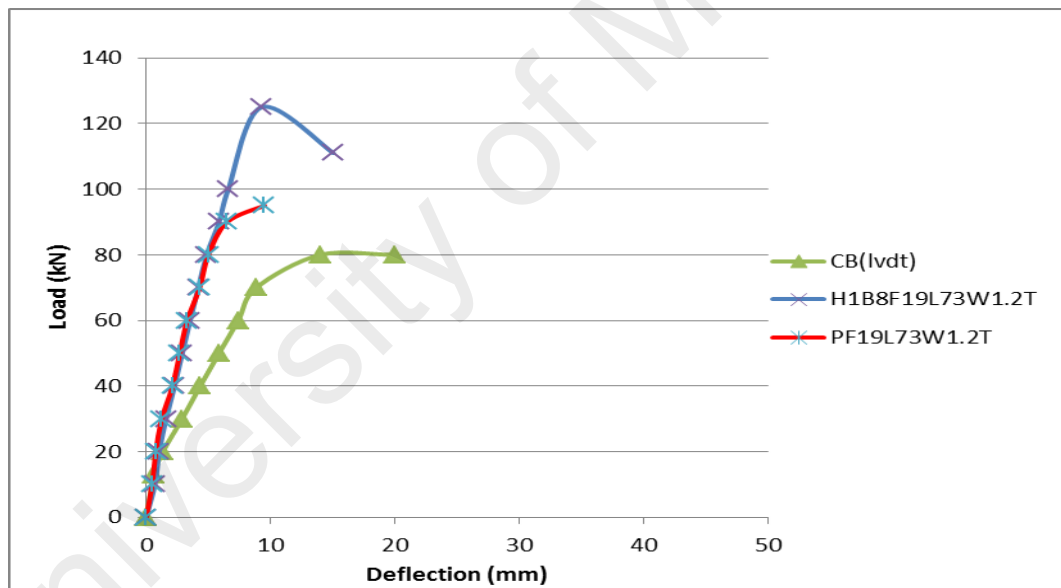


Figure 4.30: Load-deflection of CB, H1B8F16L80W1.2T and PF16L80W1.2T

Controversially, the deformability of the HSM strengthened beams were nearly similar to the deformability of the control beam although ultimate loads increased significantly. However, as loads after debonding were more than the yield load of the control beam, it cannot be directly said that the ductility of the HSM strengthened beams was reduced significantly.

(c) Cracking Behaviour

The cracking behaviour of the beams strengthened with CFRP improved because of improved composite action through hybridization. Improvement in the first crack loads of the CFRP-HSM strengthened beams is shown in Figure 4.31. The first crack load of H1B8F19L80W1.2T and H1B8F16L80W1.2T increased 14% and 65% over the cracking loads of the EBR beams PF19L80W1.2T and PF19L80W1.2T6L80W1.2T, respectively.

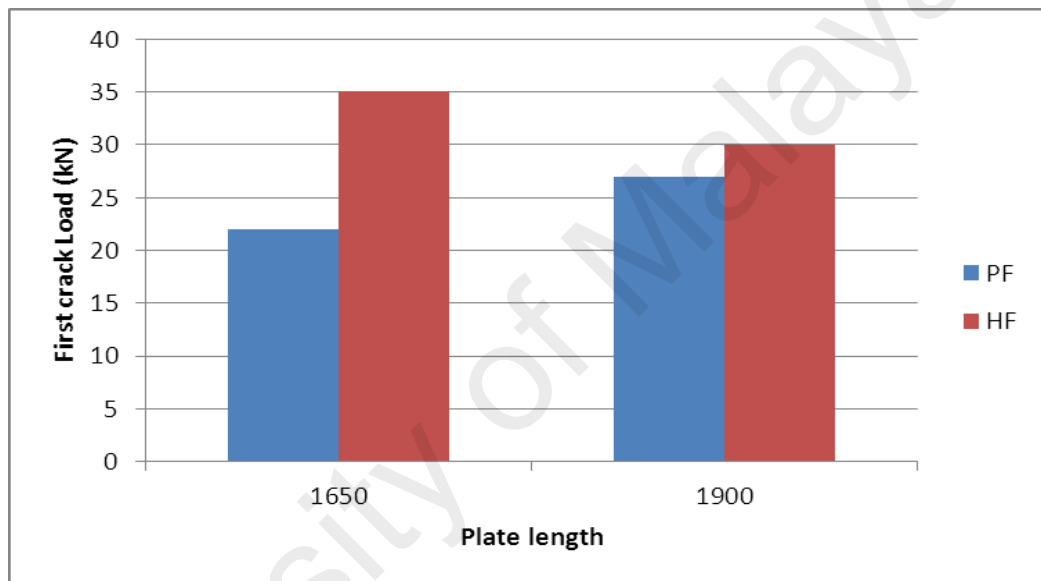


Figure 4.31: Improvement in first crack loads of HSM strengthened CFRP beams

(d) Internal Reinforcing Bar Strain

The strains in the internal reinforcing steel bars and the reduction of these bar strains due to strengthening were measured at 20 kN, 40 kN, and 60 kN service loads, as shown in Table 4.7. The internal bar strains in strengthened beams were significantly reduced. According to Hooke's law, reduced bar strains result in reduced bar stresses. Thus, the fatigue life of the strengthened beams should have increased according to the S-N curve relation of steel bar (Helagson & Hanson, 1974; Moss, 1982). However, any difference

in the reduction of bar strains between the CFRP-HSM strengthened beams and the corresponding EBR beams was not clearly noticeable.

Table 4.7: Bar strain at different service loads

Beam No.	Load at 20 kN		Load at 40 kN		Load at 60 kN	
	Bar Strain	Reduction (%) over CB	Bar Strain	Reduction (%) over CB	Bar Strain	Reduction (%) over CB
CB	793		1661		2507	
H1B8F19L80W1.2T	197	75	792	52	1305	48
H1B8F16L80W1.2T	222	72	629	62	1212	52
PF19L80W1.2T	200	75	800	52	1572	37
PF16L80W1.2T	212	73	970	42	1605	36

4.2.3.4 Effect of Plate and Bar Length, Bar Dia. and No. of Grooves

Strengthening with different configurations was investigated to identify the most suitable arrangement to achieve the best improvement in flexural performance of the RC beam strengthened with FRP and steel bars.

(a) *Effect of Plate and Bar Length*

The effect of plate and bar length is shown in Figure 4.32. Based on the experimental data of H1B8F19L80W1.2T, H1B8F16L80W1.2T, PF19L80W1.2T and HF16L80W1.2T beams, increasing the plate length resulted in increased failure loads, which is expected behaviour commonly observed in experiments. Increasing the plate length reduced the distance between the plate end and the support which is an influential parameter in end debonding behaviour of externally bonded reinforcement (Täljsten, 1997).

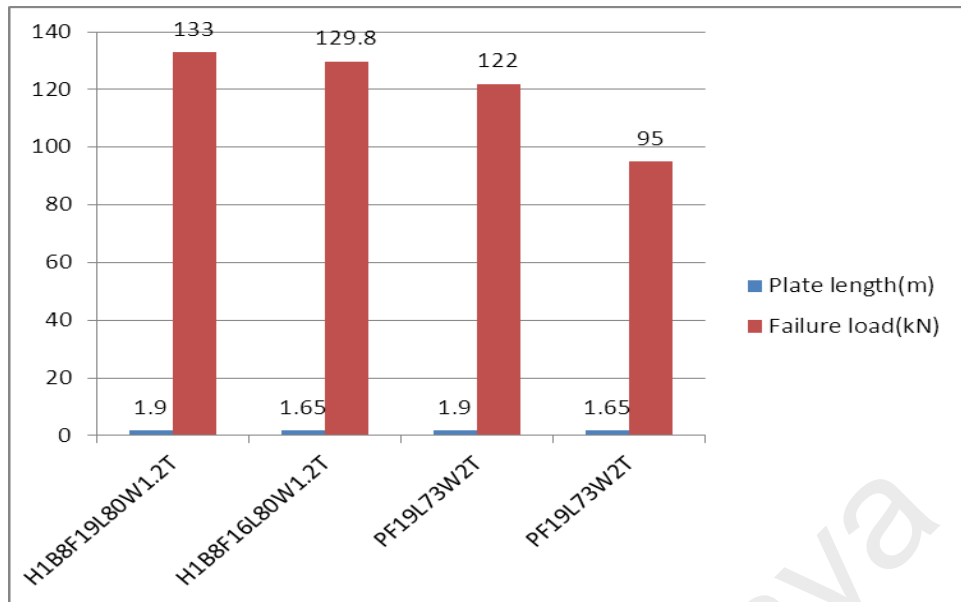


Figure 4.32: The effect of plate and bar length on failure load

(b) Effect of Bar Diameter

Bar diameter has a significant influence on the flexural resistance of normal RC beams. However, in NSM and HSM, increasing the bar diameter reduces the amount of adhesive and thereby may affect the performance of the bond. Hence, it is important to investigate the influence of bar diameter on the performance of strengthened beams. Figure 4.33 shows the effect of bar diameter on failure load in beams strengthened using the HSM with CFRP. Data collected from beams H1BP8F16L80W1.2T and, H1BP6F16L80W1.2T were used to analyze this influence. As shown in Figure 4.33, the failure load increased with increasing bar diameter.

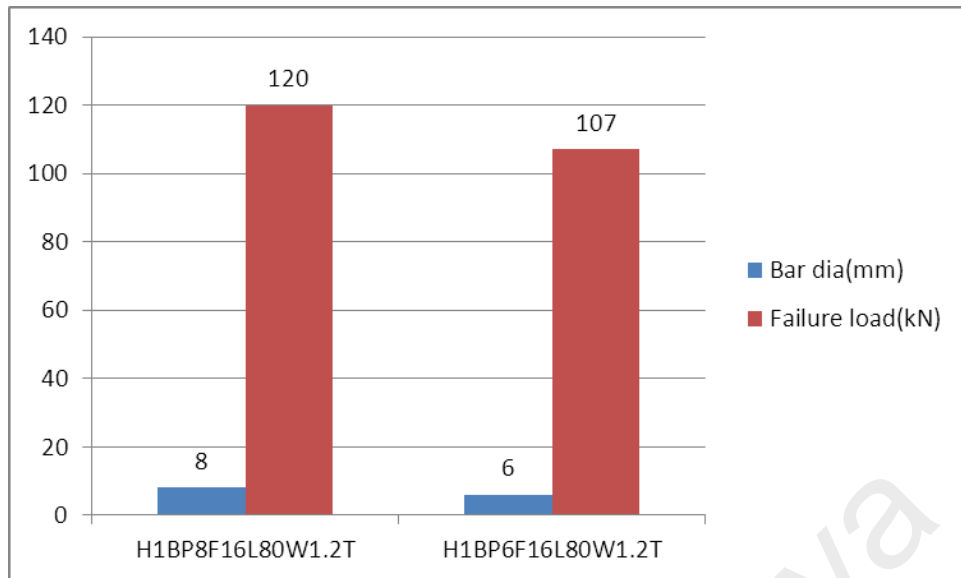


Figure 4.33: The effect of bar diameter

(c) Effect of Number of Bars or NSM Grooves

For every NSM bar used, a separate groove is required to place the bar in the concrete beam. Thus, the number of bars is equal to the number of grooves. Figure 4.34 shows the effect of number of grooves or bars on the performance of the HSM strengthened beams with CFRP. The experimental data of H1B8F19L80W1.2T and H2B6F16L100W1.2T were used to investigate this effect. It is important to note that the plate widths of the plates are not the same and it cannot be compared directly. However, the harmful effect of increased groove number can be easily understood. As shown in Figure 4.34, the failure load decreased with increasing number of bars or grooves. Increasing the number of grooves decreases both edge clearance and clear spacing between two adjacent grooves. This increases the possibility of edge breakage. The beam specimens used in this study may not have had enough width to place two bars with sufficient clearance. Therefore, concrete separation may have been accelerated due to early edge breakage. A similar effect was found in the study of Sharaky et al. (2014), where end slips for the two beams each with two NSM bars were slightly higher than those of the beam with one NSM bar, due to a lower confinement (edge effect) in the case of two NSM bars.

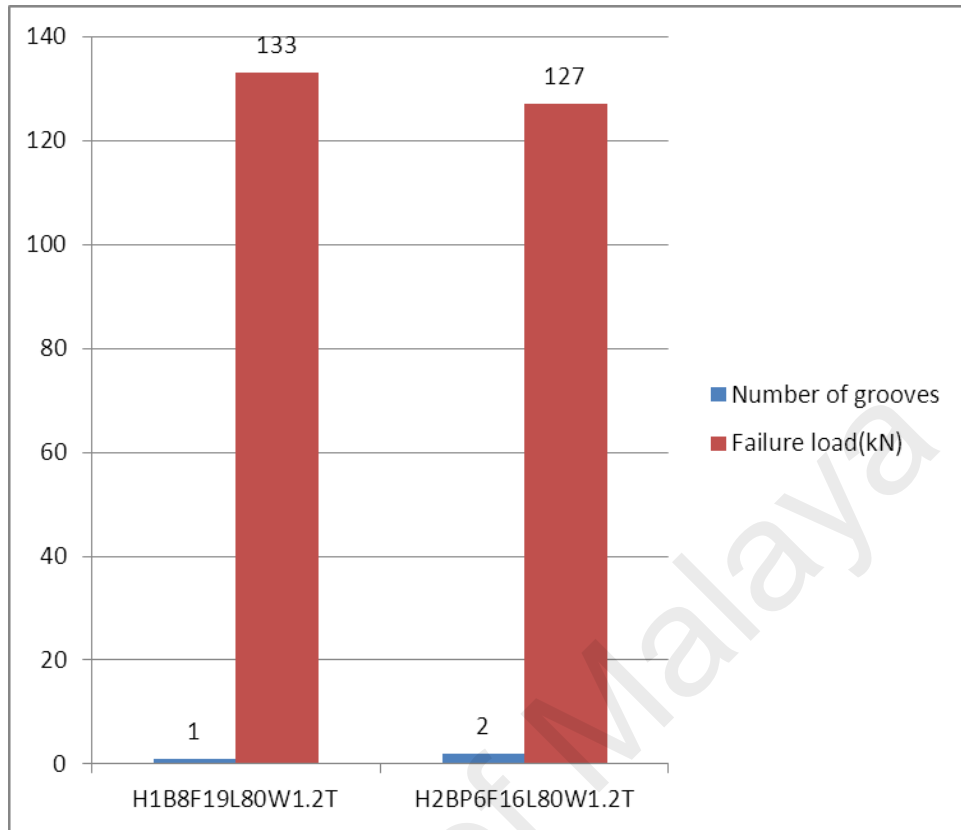


Figure 4.34: The effect of number of grooves on failure load

4.2.4 Eliminating End Debonding

4.2.4.1 Effect of Plate Thickness

Plate thickness is an important parameter for plate end debonding from concrete substrate. The beam H1B8S19L73W1.5T, H1B8S19L73W2T and H2B6S19L73W2.76T are used to observe this effect. According to Figure 4.35, the ultimate failure load decreased with increasing thickness. An increase in thickness from 1.5 to 2.76 mm resulted in, the failure load decreased from 137 kN to 130 kN. This happened due to increased interfacial shear stress. Lousdad et al. (2010) found similar results.

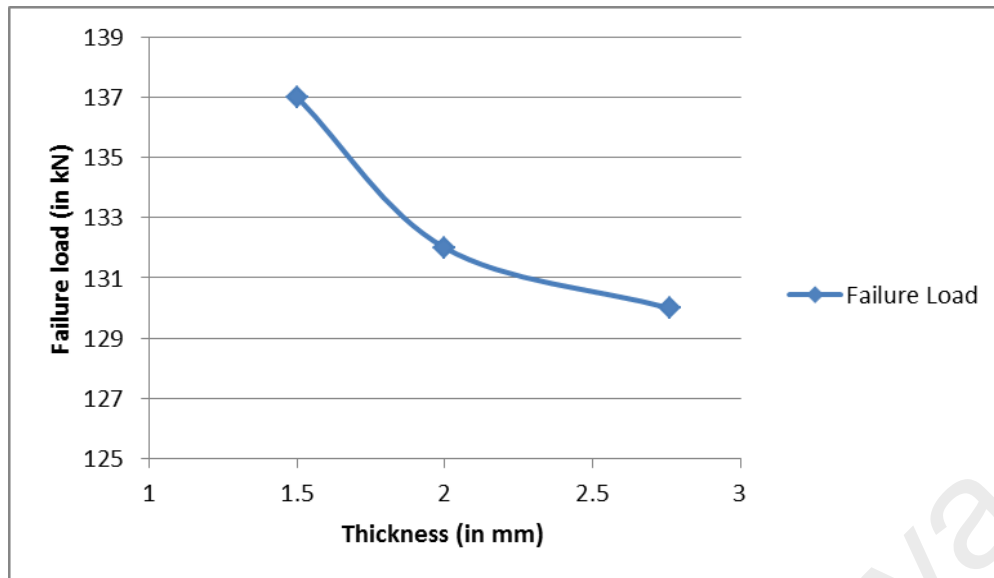


Figure 4.35: The effect of plate thickness.

4.2.4.2 Effect of Shear Strengthening

Although the hybrid strengthening system improved the performance of strengthened beams, it could not prevent premature debonding failure. Most of the HSM strengthened beams failed in concrete cover separation. The cause of this failure mode was mostly shear cracks and partly flexural shear cracks. To counter this, it was decided that the internal shear capacity of a strengthened beam should be increased. In H1B8SD19L73W2T, the spacing of the internal reinforcement was reduced to 40 mm c/c. However, the failure load was decreased to 124 kN from 132 kN (H1B8S19L73W2T) and the debonding failure mode could not be avoided. On the other hand, beam H1B8S19L73W2TAS was strengthened in shear with externally bonded CFRP wrap. The CFRP fabric was applied only to the side of the beam and not to the soffit. The failure mode was interestingly changed to flexural failure. The failure load was increased to 135 kN, which was very close to the failure load of the end anchored beam, H1B8S19L73W2TAF, which was fully wrapped with CFRP fabric.

4.2.4.3 Effect of End Anchorage

The effect of end anchorage was observed in H1B8F19L80W1.2TAF. CFRP wrap was used in end anchorage at both ends. The failure load was increased to 164 kN without premature debonding failure.

4.2.4.4 Effect of Location of the Steel Plate and Bar

The position of steel plate and bar influenced the failure mode of the HSM strengthened RC beam. In beam SH2S61900L100W2T, the position of plate and bar was changed from bottom to side (lower portion). The failure mode of the beam was changed to flexural failure mode. However, the ultimate failure load of SH2S61900L100W2T (HS12) decreased to 124 kN compared to H1B8S19L73W2T (HS1). This was attributed to the reduction of the effective depth of the HSM strengthened composite beam.

4.2.5 Experimental Behaviour of Steel NSM Strengthened Beam

4.2.5.1 Load Carrying Capacity and Failure Mode

A summary of the flexural behaviour of all tested NSM strengthened beams in terms of first crack load, yield load, flexural loading capacity and failure mode is shown in Table 4.8. As shown in Table 4.8, the addition of steel bars as NSM reinforcement increases the ultimate moment capacity by 22.5%, 46.8%, 43.75%, 26.46%, 23.26%, 32.8% and 25% for N2S6C, N2S6E, N2S6EC, N1S8E, N1S8C, N3S8C, N3S8C N1SH8C and N2SS8C, respectively, as compared to the control beam. The yield capacity and the first crack loads of the beams also increased after strengthening. The highest improvement in ultimate load capacity was achieved in NS2, which increased by 46.8%. This is greater than the increase in ultimate load capacity achieved by concrete beams strengthening with NSM FRP in the studies done by Hassan and Rizkalla (2001) and Soliman (2008). Hassan and Rizkalla (2001) found that NSM FRP strengthening increased the performance of the RC beams by 39%, while Soliman (2008) found that such strengthening improved the

ultimate load capacity of the beams by 18%. As the cost of an FRP bar is twenty times that of a steel bar (Hefferman, 1997), using NSM steel bars is a better option in terms of both strengthening capacity and cost.

Table 4.8: First crack, yield and failure (and mode) of NSM beams

Beam no	First crack load (kN)	Increase in first crack load (%)	Bar yield load (kN)	Failure load (kN)	Increase in failure load (%)	Mode of failure
CB	12.5	-	72	80.0	-	Flexural failure
N2S6C	20.0	62.5	90	98.0	22.5	Flexural failure
N2S6E	26.0	112.5	100	117.4	46.8	Flexural failure
N2S6EC	22.0	79.1	92	115.0	43.8	Flexural failure
N1S8E	28.0	129.1	100	101.2	26.5	Flexural failure
N1S8C	Pre-cracked	-	90	98.0	23.3	Flexural failure
N3S8C	25.0	104.1	100	106.2	32.8	Debonding failure
N1SH8C	20.0	-	-	135.0	-	Flexural failure
N2SS8C	20.0	62.5	-	100.6	25.0	Flexural failure

The failure modes of the control beam and all the NSM strengthened beams are shown in Figure 4.36 to Figure 4.44. The failure modes of the strengthened beams were found to be very similar to each other, mostly by flexural failure. In flexural failure, concrete crushing is followed by steel yielding. It is the most commonly reported mode of failure in NSM strengthened structures. NSM strengthening is less prone to debonding. However, the failure mode of the most heavily strengthened beam, N3S8C, was premature debonding. An NSM steel bar separated from the concrete side face as shown in Figure 4.42. Soliman et al. (2010) reported failure by concrete cover separation observed in most of the beams tested in the study, which were strengthened using NSM

FRP bars. From this, it can be concluded that the bond performance of steel bars to concrete is better than that of FRP bars to concrete.



Figure 4.36: Failure mode of control beam



Figure 4.37: Failure mode of N2S6C

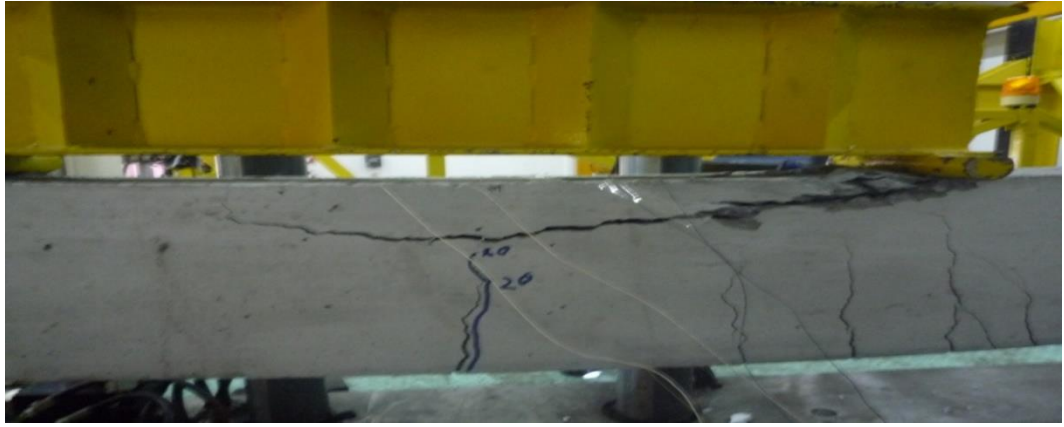


Figure 4.38: Failure mode of N2S6E



Figure 4.39: Failure mode of N2S6EC



Figure 4.40: Failure mode of N1S8E



Figure 4.41: Failure mode of N1S8C



Figure 4.42: Failure mode of N3S8C



Figure 4.43: Failure mode of N1SH8C



Figure 4.44: Failure mode of N2SS8C

4.2.5.2 Effect of Strengthening on Deflection, Crack and Strain

Deflections (actuator) and reduction in deflections due to strengthening using NSM steel bars were measured under 20 kN, 40 kN, and 60 kN service loads, as shown in Table 4.9. The deflections of the strengthened beams were reduced in comparison to the control beam due to the increased stiffness of the strengthened beams.

Table 4.9: Reduction in deflection due to NSM strengthening

Beam No.	Load at 20 kN		Load at 40 kN		Load at 60 kN	
	Deflection in mm (Instron)	Reduction (%) over CB	Deflection in mm (Instron)	Reduction (%) over CB	Deflection in mm (Instron)	Reduction (%) over CB
CB	2.42	-	5.51	-	8.48	-
N2S6C	1.48	39	3.71	33	5.01	41
N2S6E	1.55	36	3.23	41	5.26	38
N2S6EC	1.81	25	3.69	33	5.47	35
N1S8E	2.16	10	5.48	1	7.83	8
N1S8C	3.86	-*	5.17	6	6.81	20
N3S8C	1.39	43	3.12	43	6.94	18
N1SH8C	2.10	13	4.00	27	5.23	38
N2SS8C	2.04	16	4.20	24	6.44	24

* Deflection is more due to prior cracking.

The strain in the internal reinforcing steel bars and the reduction of strain in these bars due to strengthening was measured at 20 kN, 40 kN, and 60 kN service loads as shown in Table 4.10. The strain in the internal steel bars was significantly reduced in the strengthened beams. Thus, according to Hooke's law, the stress in the rebars was likewise reduced. Due to the reduction in stress, the fatigue life of the strengthened beam should also have increased according to the S-N curve relation for steel bars (Helagson & Hanson, 1974; Moss, 1982).

Table 4.10: Reduction of strain in steel rebars due to NSM strengthening

Beam No.	Load at 20 kN		Load at 40 kN		Load at 60 kN	
	Bar Strain	Reduction (%)	Bar Strain	Reduction (%)	Bar Strain	Reduction (%)
CB	793	-	1661	-	2507	-
N2S6C	556	30	1059	36	1509	40
N2S6EC	248	69	1465	12	3006	20
N1S8E	406	49	1528	8	2418	4
N3S8C	-*	-	933	44	1523	39
N1SH8C	319	60	737	56	1451	51

*The value was missing due to delay in interval setting.

The strain in the concrete at the surface of the beams and the reduction of these concrete strains due to strengthening were measured at 20 kN, 40 kN, and 60 kN service loads, as shown in Table 4.11. The concrete strain was reduced significantly in the strengthened beams.

Table 4.11: Reduction in concrete strain due to NSM strengthening

Beam No.	Load at 20 kN		Load at 40 kN		Load at 60 kN	
	Concrete strain	Reduction (%)	Concrete strain	Reduction (%)	Concrete strain	Reduction (%)
CB	252	-	602	-	990	-
N2S6C	175	31	376	38	510	48
N2S6E	126	50	337	44	542	45
N1S8C	200	21	383	36	550	44
N3S8C	-*	-	450	25	697	30
N1SH8C	215	15	408	32	609	38

*The value was missing due to delay in interval setting.

4.2.5.3 Effect of Different Parameters

Strengthening with different configurations was investigated to identify the most suitable arrangement to achieve the best improvement in flexural performance of the RC beam strengthened with steel bars.

(a) Effect of Adhesive Type

The effect of adhesive type on the performance of the NSM strengthened RC beam is shown in Figure 4.45. Based on experimental data of N2S6C and N2S6E, as the adhesive type changed from cement mortar to epoxy, the failure load increased from 98 kN to 117.44 kN. This is normal behaviour because bonding strength of epoxy is significantly higher than that of cement mortar. First crack load also increased from 20 kN to 26 kN due to change of adhesive type. The load deflection behaviour of CB, N2S6C and N2S6E is shown in Figure 4.46.

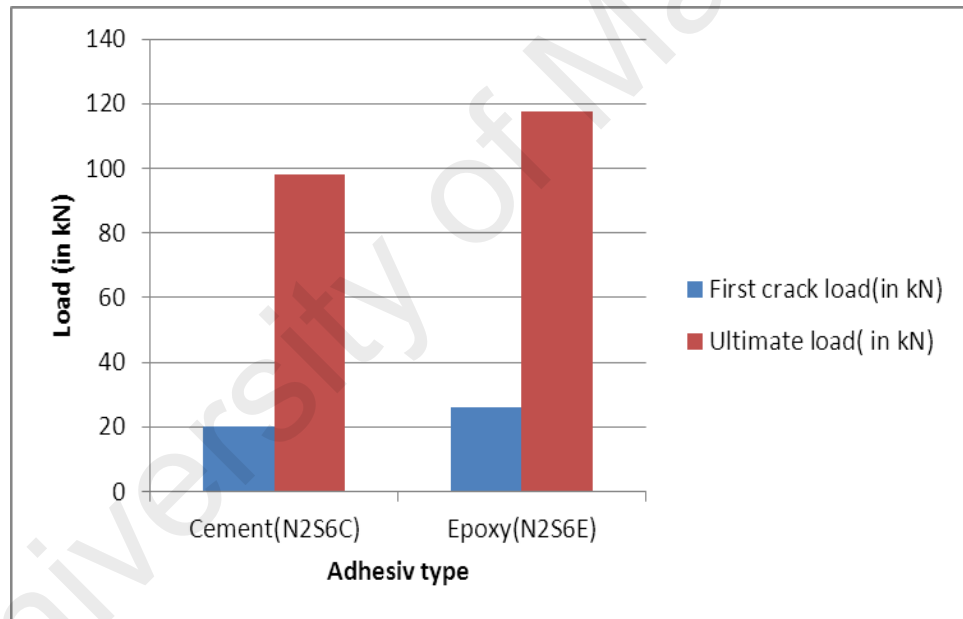


Figure 4.45: The effect of adhesive type on first crack and failure load

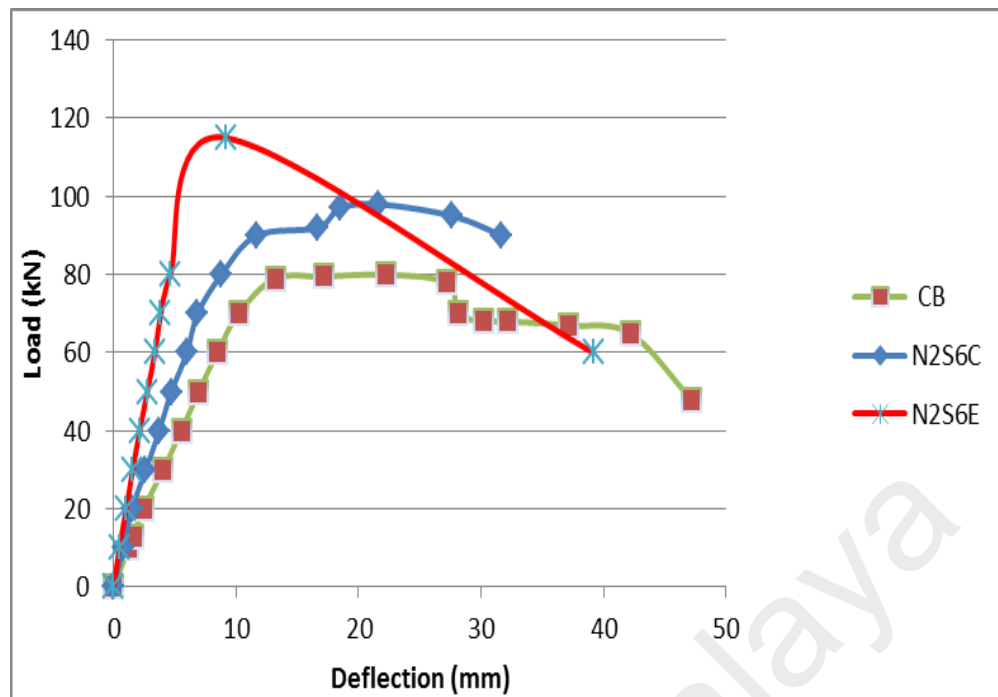


Figure 4.46: Load-deflection diagram of CB, N2S6C and N2S6E

(b) Effect of Partial Epoxy Replacement with Cement Mortar

Cement mortar has inferior mechanical properties and durability, with a tensile strength lower than that of commercially available epoxies (De Lorenzis & Teng, 2007). Results of bond tests and flexural tests (Nordin & Taljsten, 2003; Taljsten et al., 2003) have identified some significant limitations of cement mortar as a groove filler. However, bond stresses are not equally distributed along the length of an NSM bar, as shown in Figure 4.47. Maximum bond stresses are found near the ends of the NSM bar and they gradually decrease towards the mid span of the beam. This characteristic variation of bond stresses may allow the partial replacement of epoxy with cement mortar.

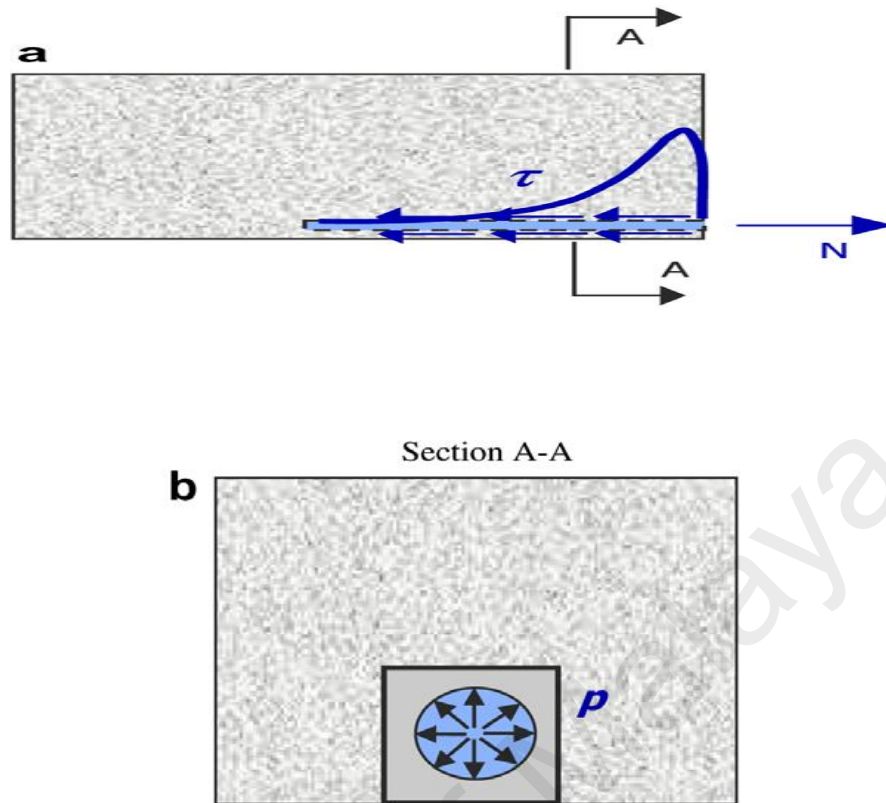


Figure 4.47: Bond stresses in the longitudinal plane (De Lorenzis & Teng, 2007)

Since the bond stresses at the midsection of a beam are relatively low, the NSM grooves could be filled with cement mortar at this location. However, in other places, particularly at the groove end, the groove should be filled with epoxy adhesive due to the presence of higher bond stresses. The effect of the partial replacement of epoxy with cement mortar is shown in Figure 4.48. The failure load of N2S6EC (50% epoxy replaced with cement mortar) is almost similar to the failure load of N2S6E (epoxy used entirely) but significantly higher than that of N2S6C (where cement mortar is used entirely). However, the stiffness of N2S6EC is slightly lower than that of N2S6E because the deflection of beam N2S6EC is lower than the deflection of N2S6E due to presence of cement mortar as shown in the load-deflection diagram of Figure 4.49 (LVDT data are used). This is due to the presence of cement mortar.

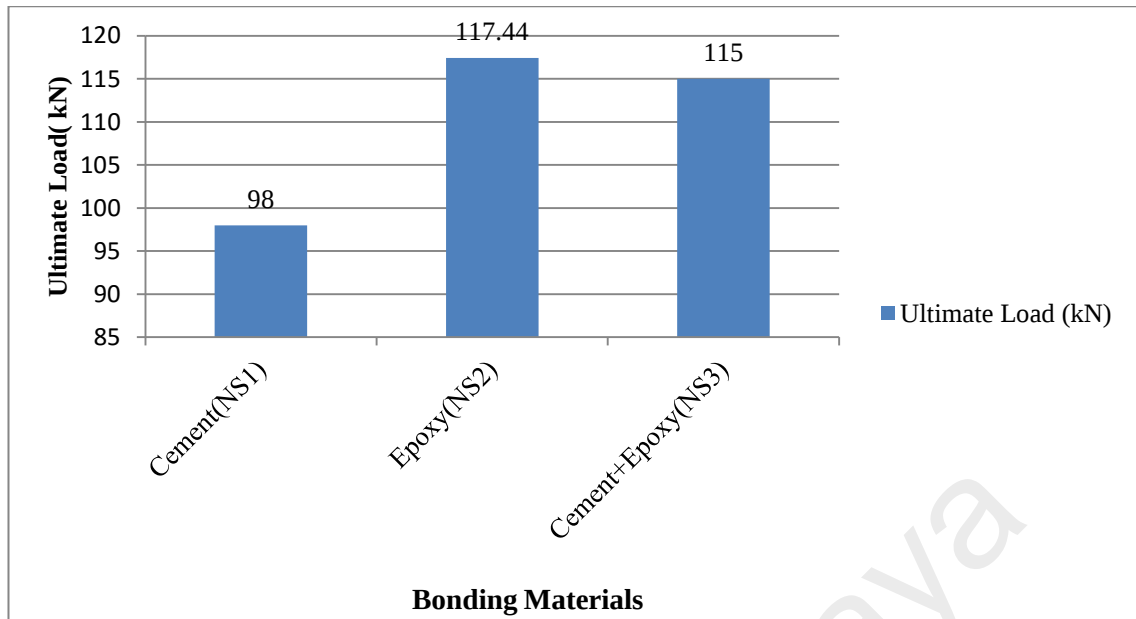


Figure 4.48: The effect of partial replacement of epoxy with cement mortar

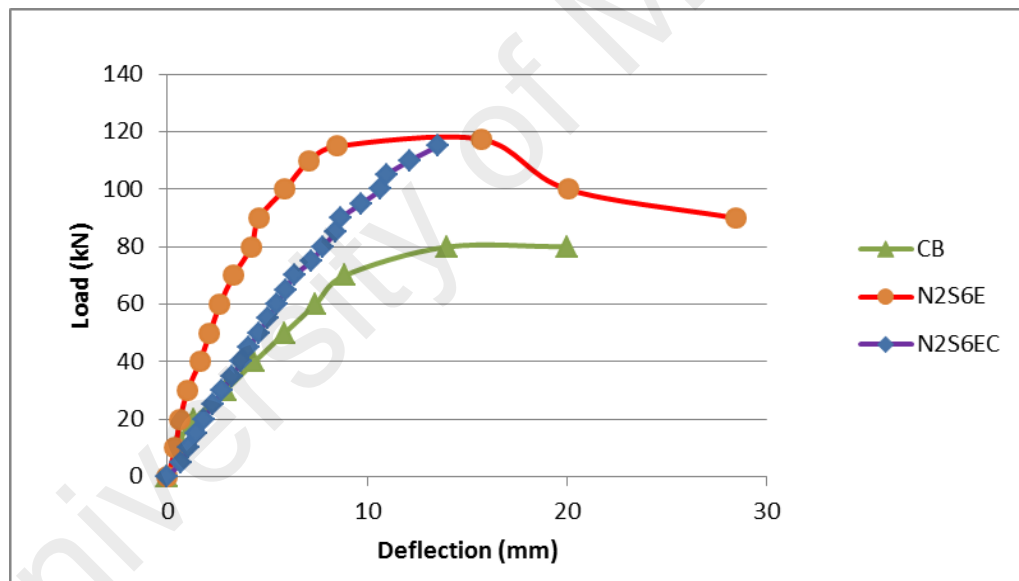


Figure 4.49: Load-deflection diagram of CB, N2S6E and N2S6EC

(c) Effect of Number of NSM Grooves

Each NSM bar used to strengthen a concrete beam requires a groove to be placed in. Thus, the number of grooves is equal to number of bars used. The experimental data of specimens N2S6C and N1S8C were used to investigate the effect of number of grooves on the performance of beams where cement mortar was used in strengthening. Data from

specimens N2S6E and N1S8E were used to investigate this effect on beams that used epoxy adhesive. However, the total amount of strengthening reinforcement in both cases was similar (56 mm). Figure 4.50 shows the effect of number of grooves on the performance of beams strengthened using the NSM technique, with the same amount of reinforcement, but different adhesives.

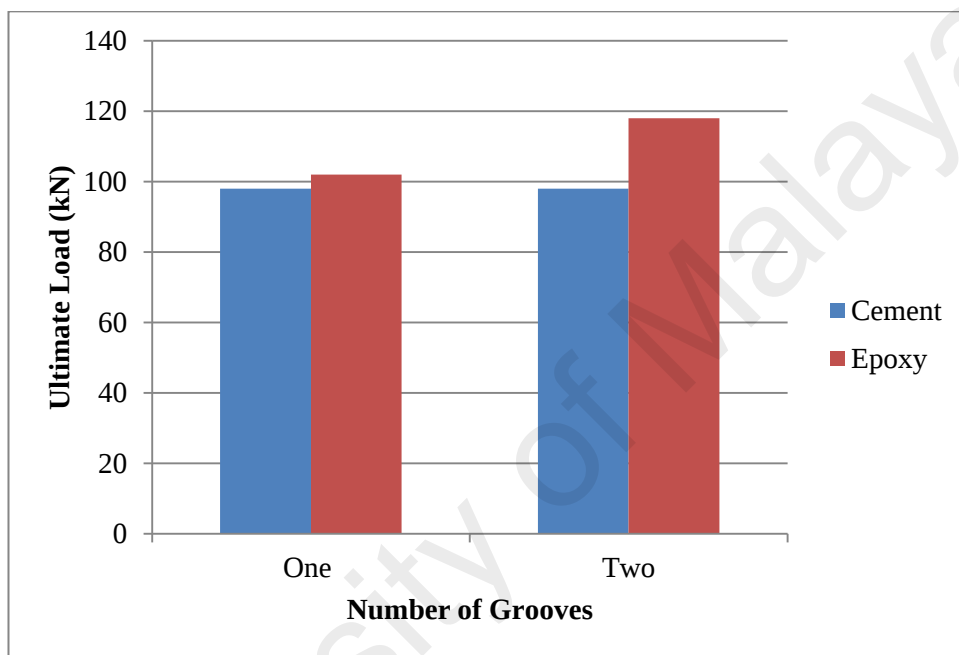


Figure 4.50: The effect of number of grooves

As can be seen from Figure 4.50, the failure load increased when the number of grooves was increased from one to two in the case of beams using epoxy adhesive. However, in the case of the beams using cement mortar, the failure load was almost the same. The increase in groove number provides an additional amount of groove fillers in concrete beam. The load-deflection diagram of CB, N2S6C, N2S6E, N1S8E, N1S8C are shown in Figure 4.51 and Figure 4.52

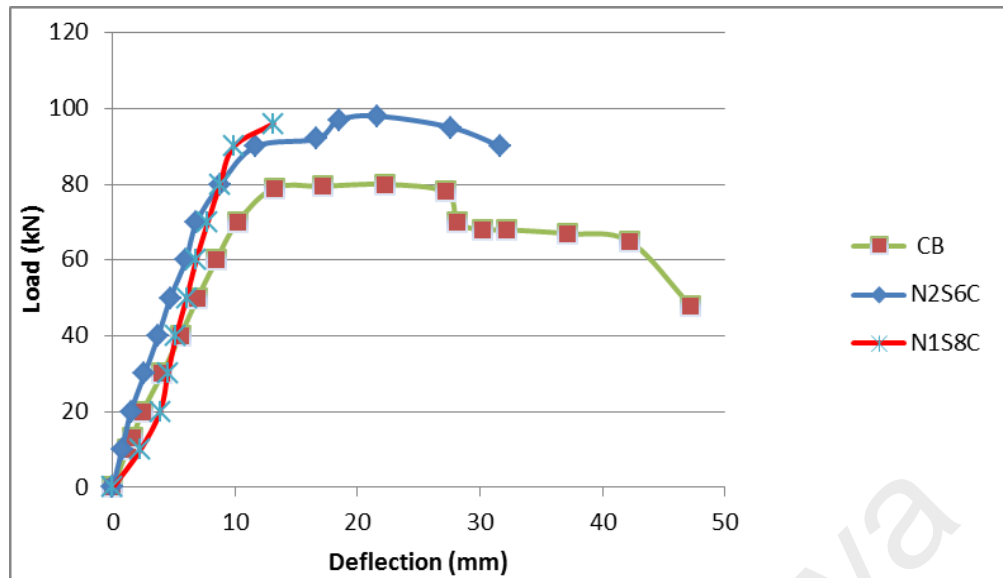


Figure 4.51: Load-deflection of CB, N2S6C and N1S8C with cement mortar

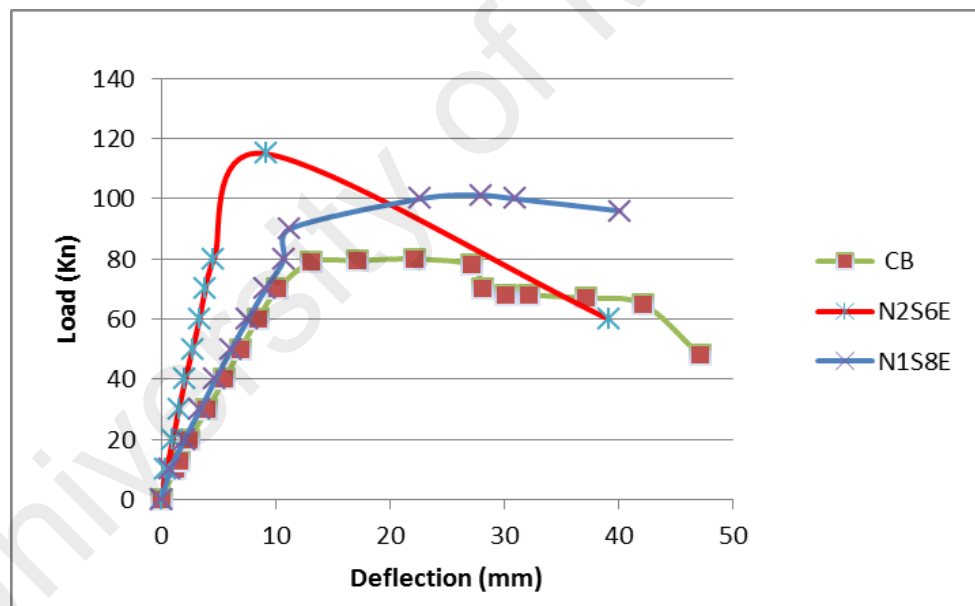


Figure 4.52: Load-deflection diagram of CB, N2S6E and N1S8E

(d) Effect of Bar Numbers with the Same Diameter

The effect of bar number is shown in Figure 4.53. The amount of reinforcement used is the single most important parameter in the flexural strengthening of RC beams. Increasing the number of NSM bars provides additional reinforcement to the concrete

beam, but decreases both edge clearance and the clear spacing between two adjacent grooves. This increases the possibility of the edge of the beam breaking off. The width of the beam specimens used in this study was not sufficient to place two or three 8 mm bars on the bottom face of the beam according to ACI 440. Thus, in specimen N3S8C, which used three 8 mm bars, the bars were placed at different positions (one at the bottom and two at opposite sides and in N2SS8, which used two 8 mm bars, the bars were placed at opposite sides of the beam

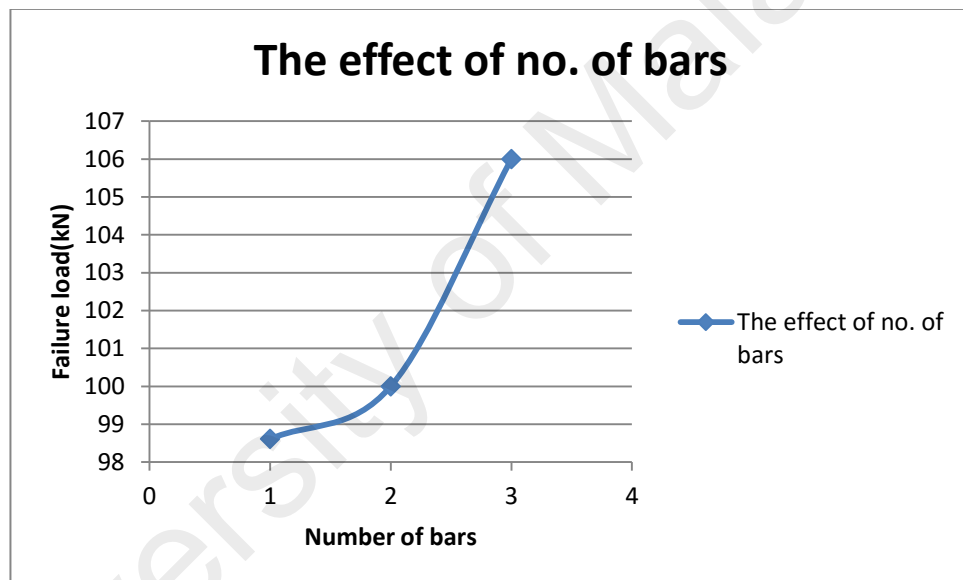


Figure 4.53: The effect of bar number on the performance of NSM beam

Figure 4.53 shows the effect of number of bars on failure behaviour of the RC beam strengthened with NSM steel bar using cement mortar as adhesive. Beams N1S8C, N3S8C and N2SS8C are used to observe this effect. The load deflection diagram of CB, N1S8C and N3S8C is shown in Figure 4.54. Failure modes were changed in N3S8C from flexural failure to debonding failure.

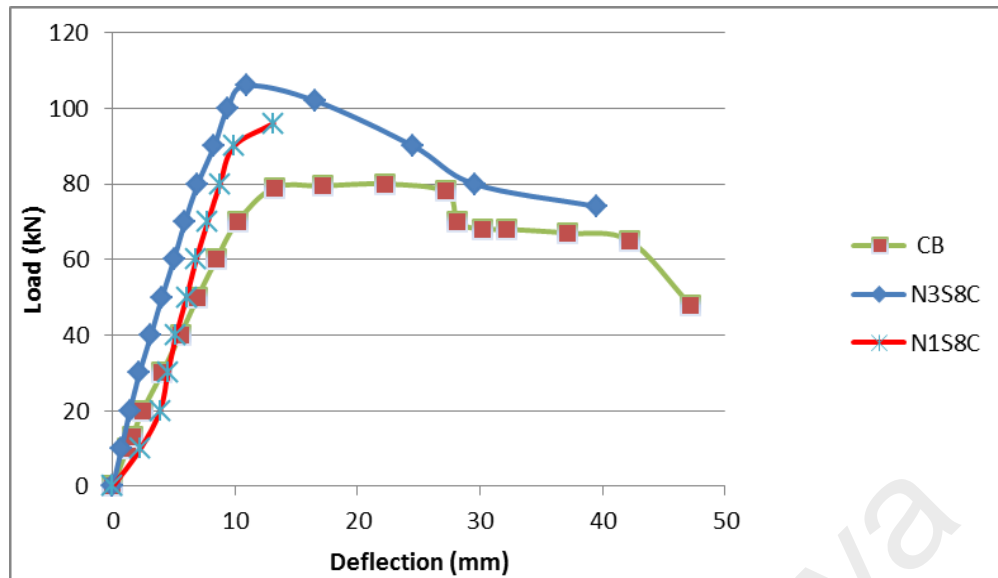


Figure 4.54: Load-deflection diagram of CB, N1S8C and N3S8C

(e) Effect of Internal Reinforcement

In beam N1SH8C, a higher internal reinforcement was used. Instead of bar diameter of 12 mm, two 16 mm bars were used as internal reinforcement. This caused the failure load to increase to 135 kN. However, the failure mode remained the same, i.e. flexural failure (concrete crushing followed by steel yielding).

4.2.5.4 Comparison of NSM with EBR

Compared to externally bonded reinforcement, the NSM system has several advantages. Figure 4.55 compares the performance of NSM strengthening and EBR strengthening.

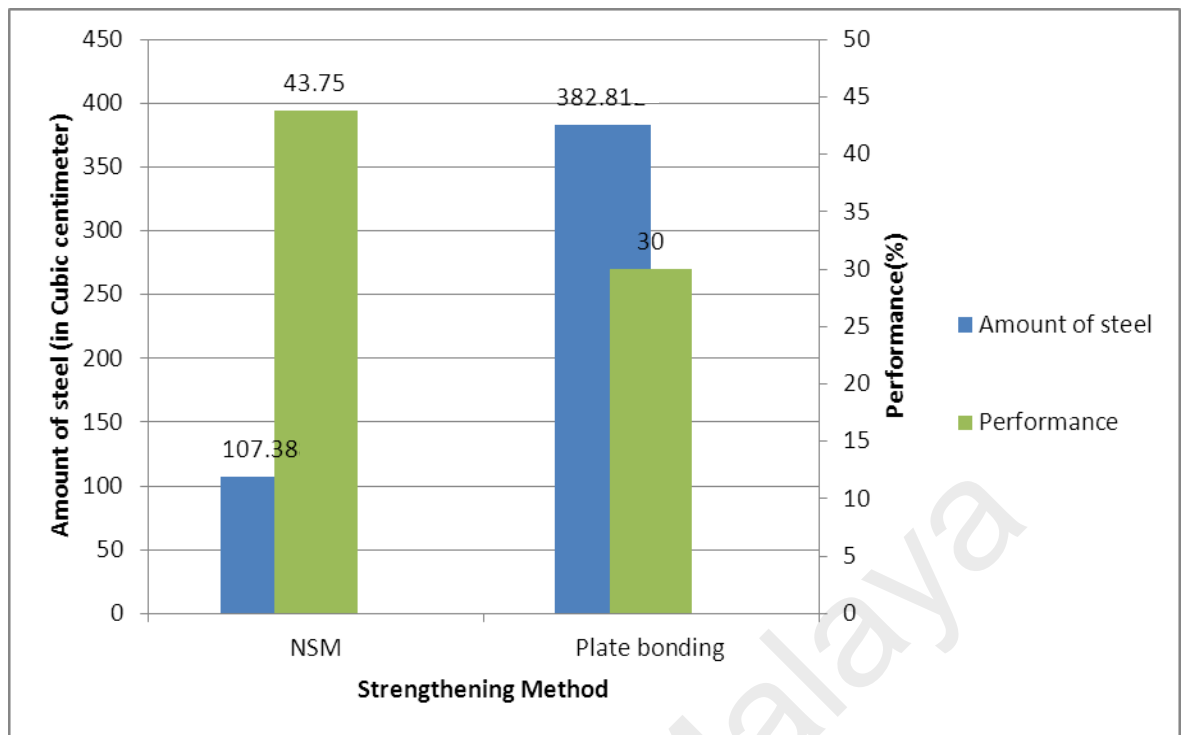


Figure 4.55: Comparison of NSM with EBR

4.2.6 Fatigue Performance of the HSM Strengthened Beam

4.2.6.1 Failure Mode

Two modes of failure were observed for the cyclically loaded RC beams. Fatigue failure in the tension steel reinforcement was the usual mode of failure. This mode of failure was expected as the stress range in the tension steel reinforcement was high enough to cause fatigue failure in the steel. The control beams, CBF1, CBF2 and the NSM strengthened beam, NSF, failed in this mode of failure. Figure 4.56 and Figure 4.57 shows the fatigue failure mode of CBF1. Figure 4.58 shows the failure mode of NSF. The EBR beam and the HSM strengthened beam (with steel bar and steel plate) both failed in debonding failure, as shown in Figure 4.59 and Figure 4.60.



Figure 4.56: Fatigue failure mode of control beam



Figure 4.57: Fatigue fracture of steel



Figure 4.58: Failure mode of NSF



Figure 4.59: Failure mode of PSF

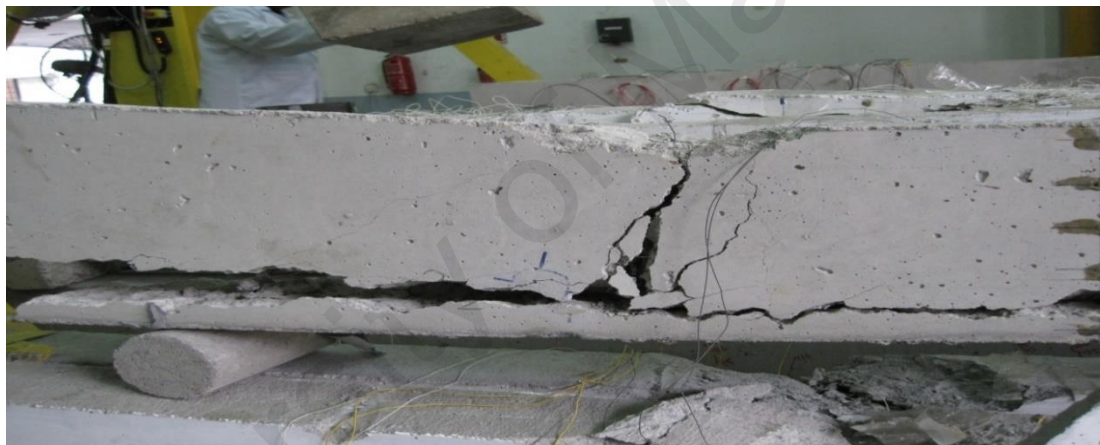


Figure 4.60: Failure mode of HSF

4.2.6.2 Number of Cycles to Failure

Table 4.12 shows the number of cycles to failure of the beams tested under fatigue loading. The fatigue life of the strengthened beams increased. The fatigue testing of PSF ceased after 2×10^6 cycles. PSF and HSF were loaded monotonically to failure. The fatigue life of the strengthened beams increased after strengthening due to the redistribution of stresses between the internal reinforcement and the external reinforcement, resulting in lower stresses in the internal steel reinforcement.

Table 4.12: Result of fatigue test

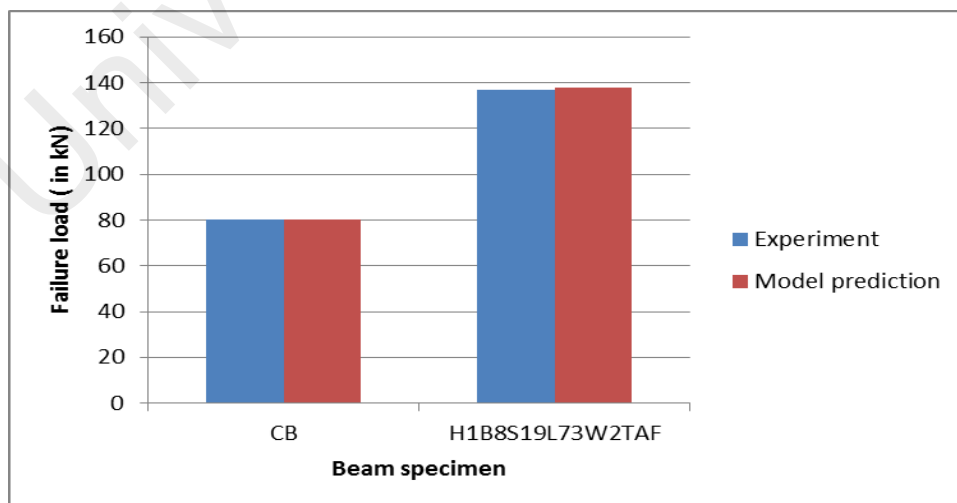
Sl. No.	Beam	Minimum Load (kN)	Maximum Load (kN)	Number of cycles to failure	Post fatigue load (kN)	Failure mode
1	CBF1	10	40 ($.5f_y$)	485000	-	Fracture of steel
2	CBF2	10	64 ($.8f_y$)	188000	-	Fracture of steel
3	NSF	10	64 ($.8f_y$)	198000	-	Fracture of steel
4	PSF	10	64 ($.8f_y$)	>2000000	98.00	Debonding
5	HSF	10	64 ($.8f_y$)	211000*	136.34	Debonding

*After this cycle the load of the Instron machine accidentally increased from 64 to 136.34 kN due to some error (tripped) and fatigue testing could not be continued.

4.3 Verification of Semi-numerical Model

4.3.1 Verification of Flexural Strength Model

To verify the flexural strength model, the ultimate failure loads of the control beam and beam H1B8S19L73W2TAF were evaluated. The correlation between the experimental and the predicted results for these beams is within a reasonable range of agreement. Figure 4.61 shows the predicted and experimental failure loads.

**Figure 4.61: Predicted and experimental failure load**

4.3.2 Verification of Deflection Prediction Model

To verify the deflection prediction model, the measured load versus deflection relationships at mid-span during loading were compared with the analytical results obtained from the model. Figure 4.62 and Figure 4.63 show the predicted and the experimental deflection measurements of the control beam, CB, and H1B8S19L73W2T (HSM with steel plate and steel bar) at different service loads.

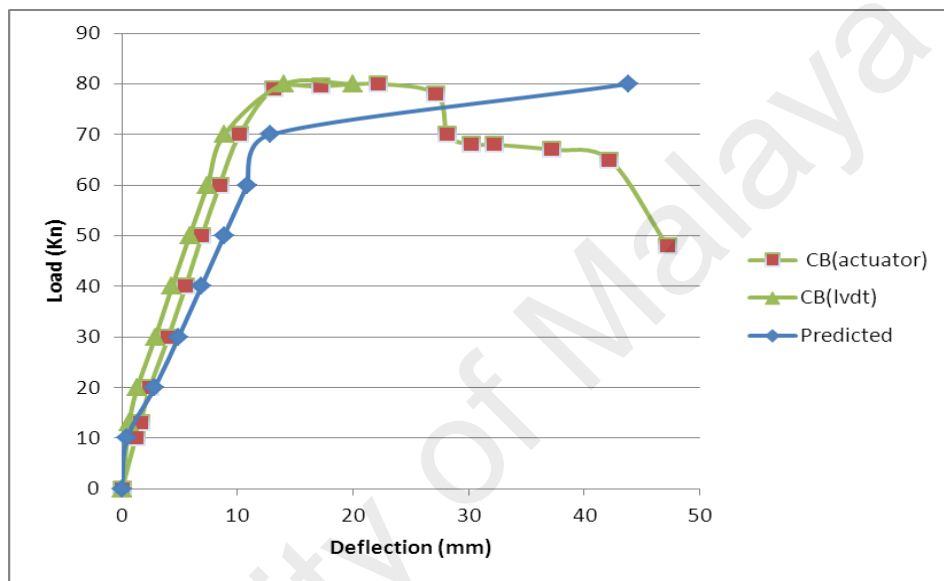


Figure 4.62: Predicted and experimental load-deflection diagram of CB

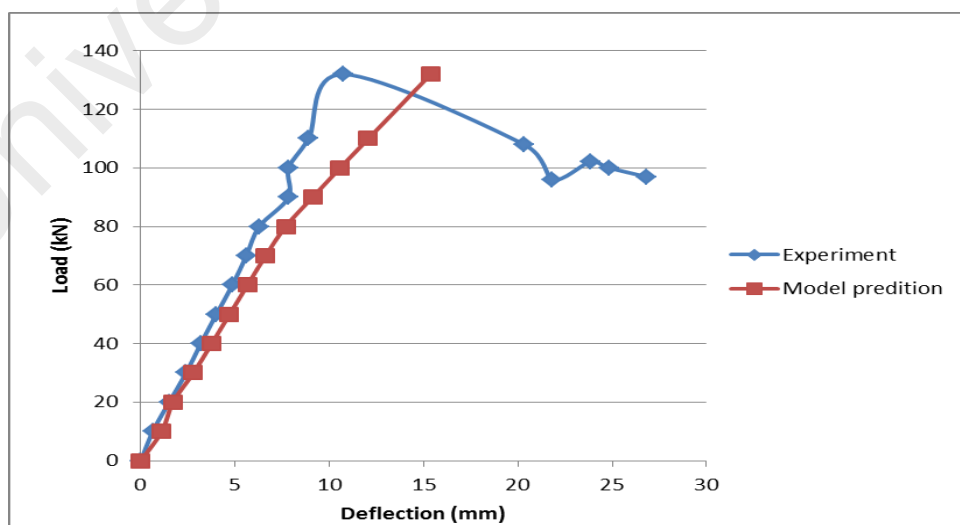


Figure 4.63: Predicted and experimental load-deflection of H1B8S19L73W2T

4.3.3 Verification of Debonding Strength Model

To verify the debonding strength models, debonding failure loads of PS19L73W2.76T (PS1), PS16L73W2.76T (PS2), H1B8S19L73W2T (HS1) and H1B8S16L73W2T(HS2) were evaluated. In addition, beams A3, A5, SM4, SM5, and B2 were taken from the previous studies (Arduini et al., 1997, Arduini and Nanni, 1997, Quantrill et al., 1996). These beams were evaluated using the proposed debonding strength model. The correlation between the experimental and predicted results for the test beams is within reasonable agreement for both EBR and HSM strengthened beam. Figure 4.64 shows the predicted and experimental measurements of load.

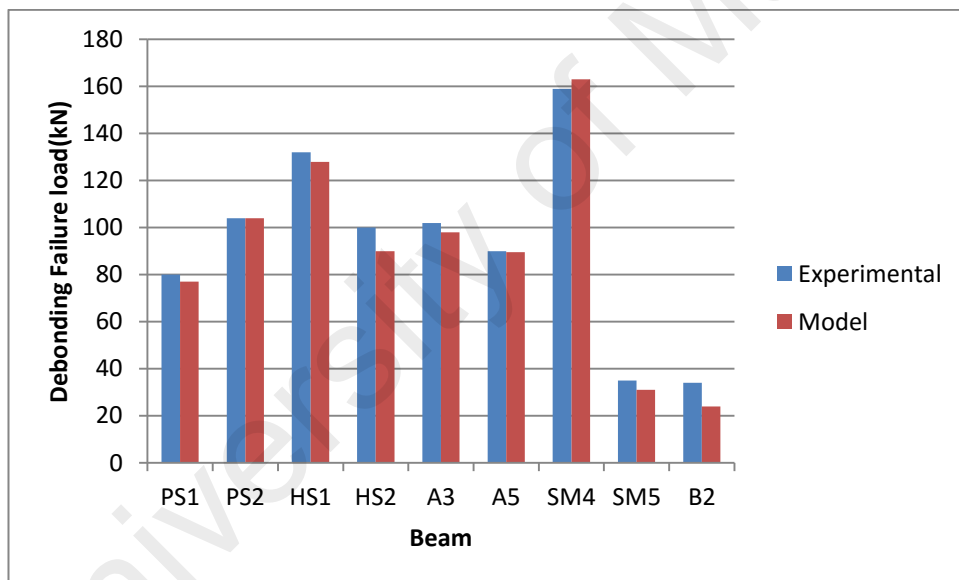


Figure 4.64: Predicted and experimental debonding failure load

4.3.4 Parametric Study using Debonding Strength Model

The effect of steel plate thickness and length using the debonding strength model is shown in Figure 4.65 and Figure 4.66. According to Figure 4.65, debonding failure load decreased with plate thickness and this trend is also similar to the experimental trend.

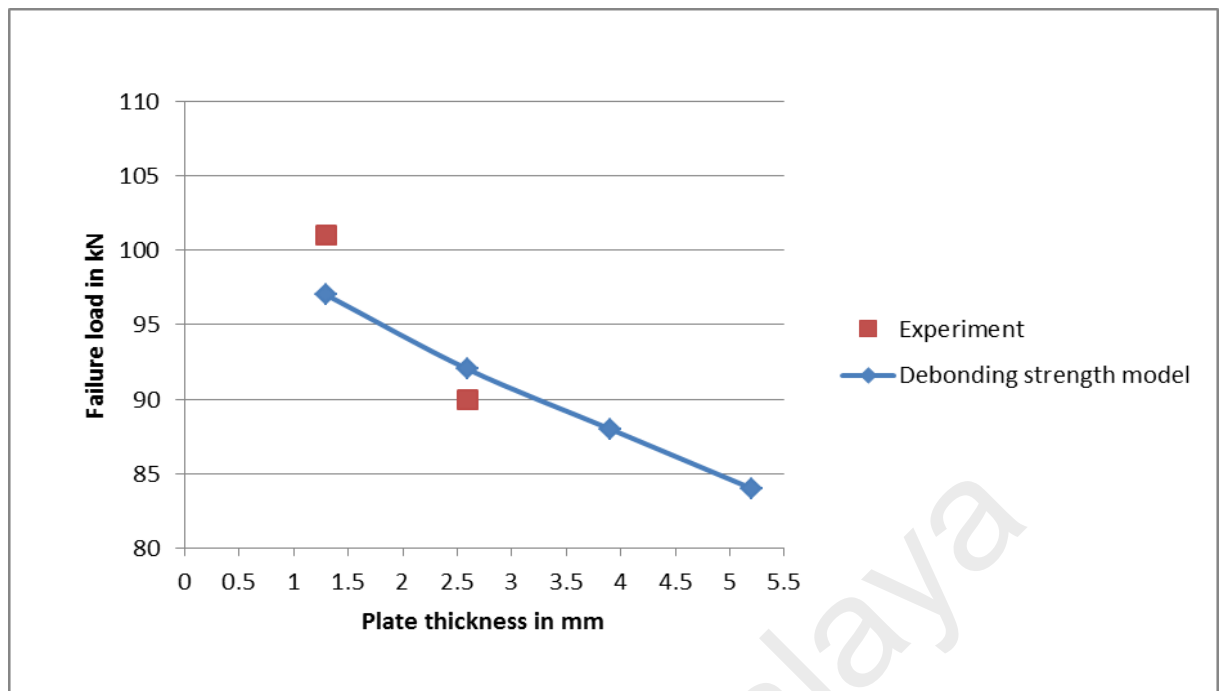


Figure 4.65: The effect of plate thickness using the debonding strength model

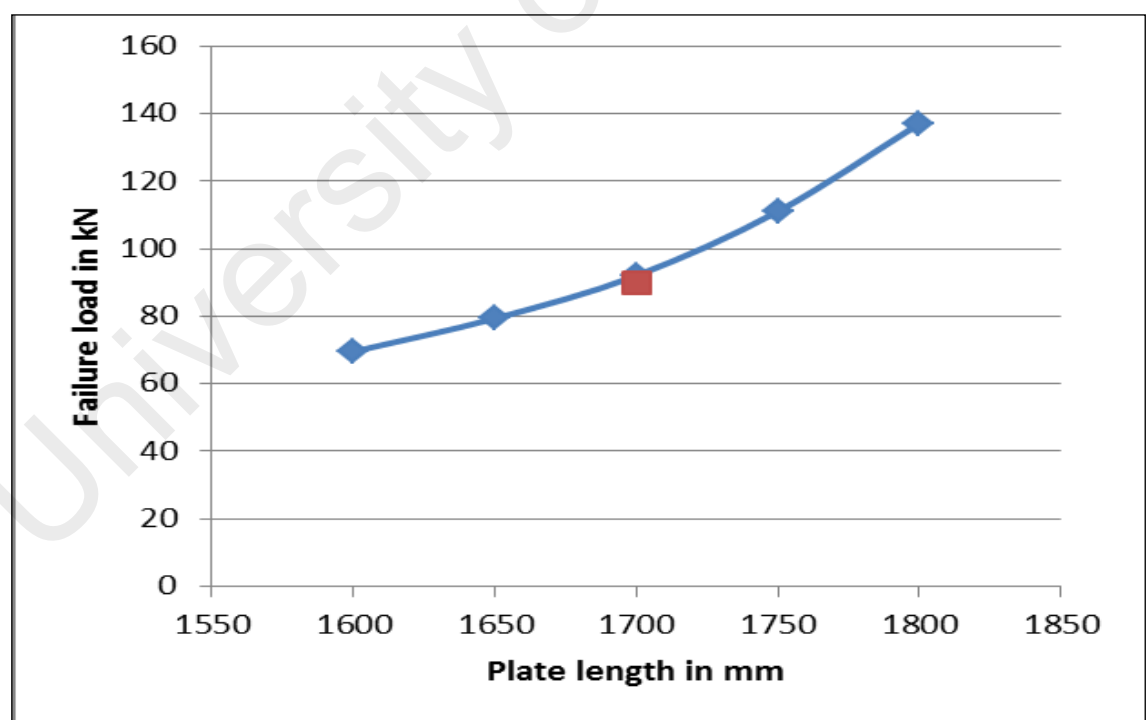


Figure 4.66: The effect of plate length using the debonding strength model

4.4 Finite Element Numerical Results

The numerical and experimental results in the following sections are presented in terms of the ultimate load carrying capacities, and deformational characteristics of the beams when using the presented model. The meshing with deflected shape of quarter of typical beam is shown in Figure 4.67 (3D).

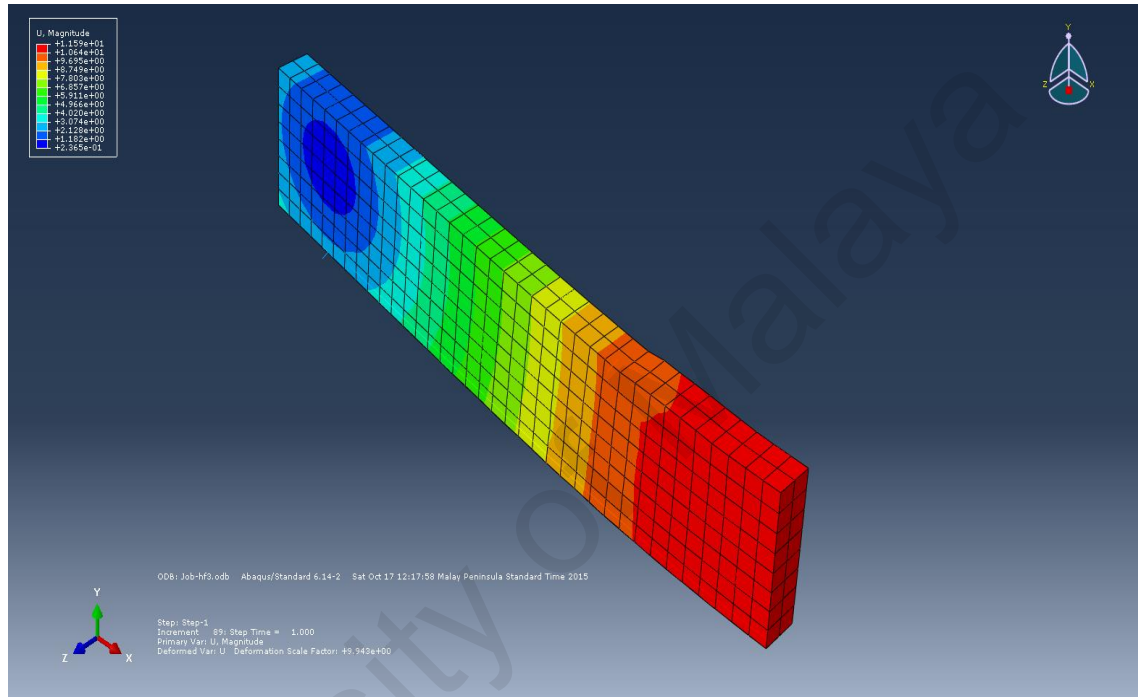


Figure 4.67: Meshing with deflected shape

4.4.1 Load Carrying Capacities

The comparison between finite element numerical and experimental results for the steel HSM strengthened beam specimens in terms of load at first crack, yield load and ultimate load is summarized in Table 4.13. As shown in Table 4.13, there is a good agreement between the predicted load carrying capacities and the experiment result of most of the test specimens.

Table 4.13: The comparison between numerical and experimental results

Beam Id	Load at first crack (kN)		Yield load (kN)		Ultimate load (kN)		Ratio
	Num.	Exp.	Num.	Exp.	Num.	Exp.	Num./exp.
CB	16	12.5	74	72	82	80.0	-
H1B8S19L73W2T	38	40.0	-	-	132	132.0	1.05
H1B8S16L73W2T	35	58.0	85	82	110	106.0	1.03
H1B6S16L73W2T	35	60.0	80	80	98	102.0	0.97
H2B8S19L73W2T	30	48.0	-	-	105	109.0	0.97
H2B6S19L73W2T	35	30.0	-	-	108	110.0	0.98
H2B6S19L73W2.76T	60	40.0	120	122	132	130.0	1.02
H2B6S19L125W1.5T	35	53.0	125	128	138	135.0	1.02
H1B8S19L73W2TAS	30	40.0	127	126	138	137.0	1.03
H1B8S19L73W2TAF	30	40.0	110	115	138	137.3	1.03
SH2S61900L100W2T	35	60.0	112	116	126	123.0	1.02

Two types of failure mode have been observed. The flexural failure modes were observed in control and NSM strengthened beams as shown in Figure 4.68 and Figure 4.69 and debonding failure modes in the form of concrete cover separation were observed in most of the HSM strengthened beams as shown in Figure 4.70 (Full scale).

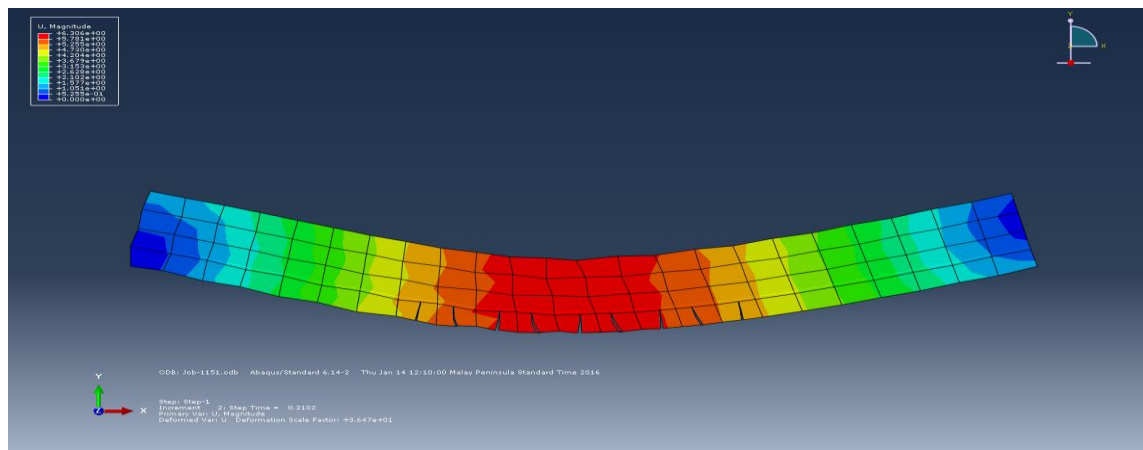


Figure 4.68: Typical flexure failure mode of control beams (2D)

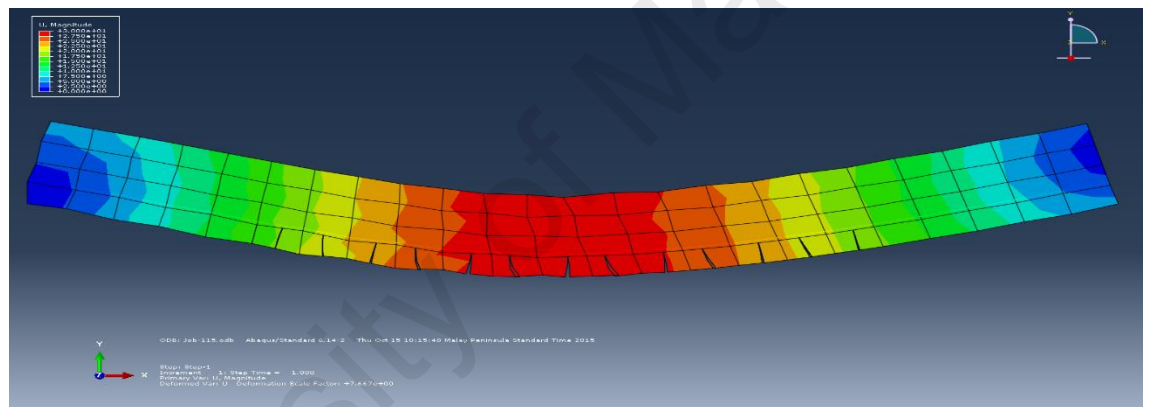


Figure 4.69: Typical flexure failure mode of NSM strengthened beams (2D)

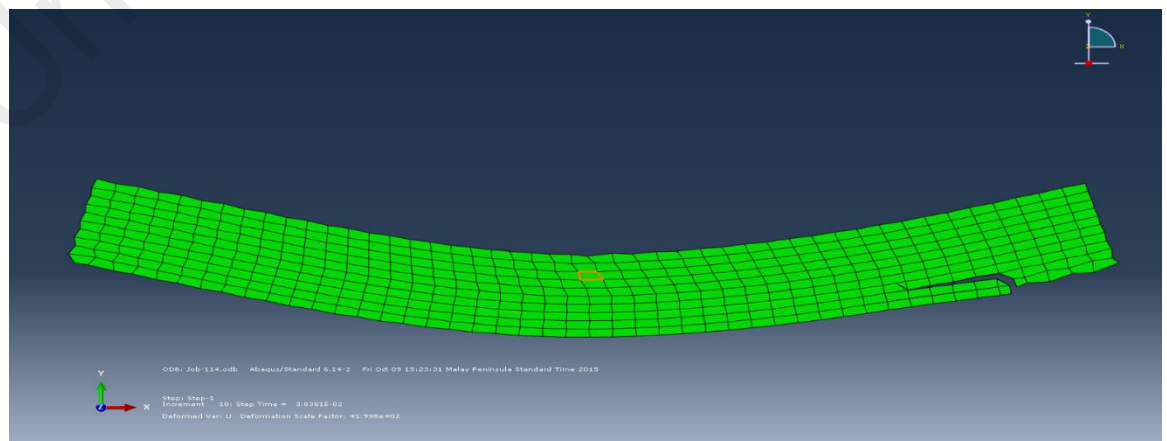


Figure 4.70: Typical debonding failure mode of HSM strengthened beam (2D)

4.4.2 Load-Deflection Relationship

The validity of the model to simulate the behaviour of RC beams strengthened using HSM was examined by comparing the experimental test results presented in Chapter 4. Figure 4.71, Figure 4.72 and Figure 4.73 show the load-deflection curves for the control specimen, the typical steel NSM and HSM strengthened beam specimens, respectively. The dotted and firm lines represents the curve of experimental and numerical results respectively. It can be observed that the correlation is reasonably good between the numerical result and the experimental data.

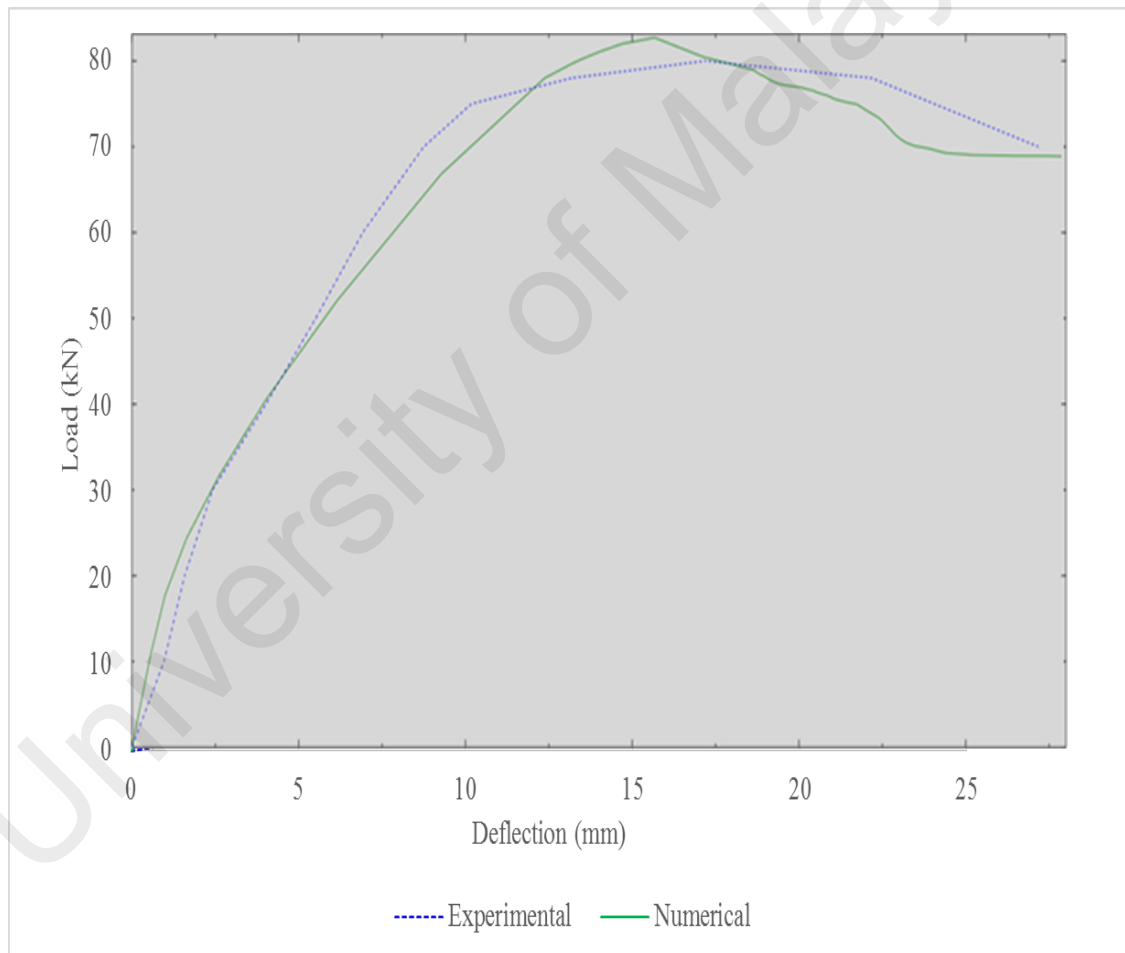


Figure 4.71 : Load deflection diagram of control beam

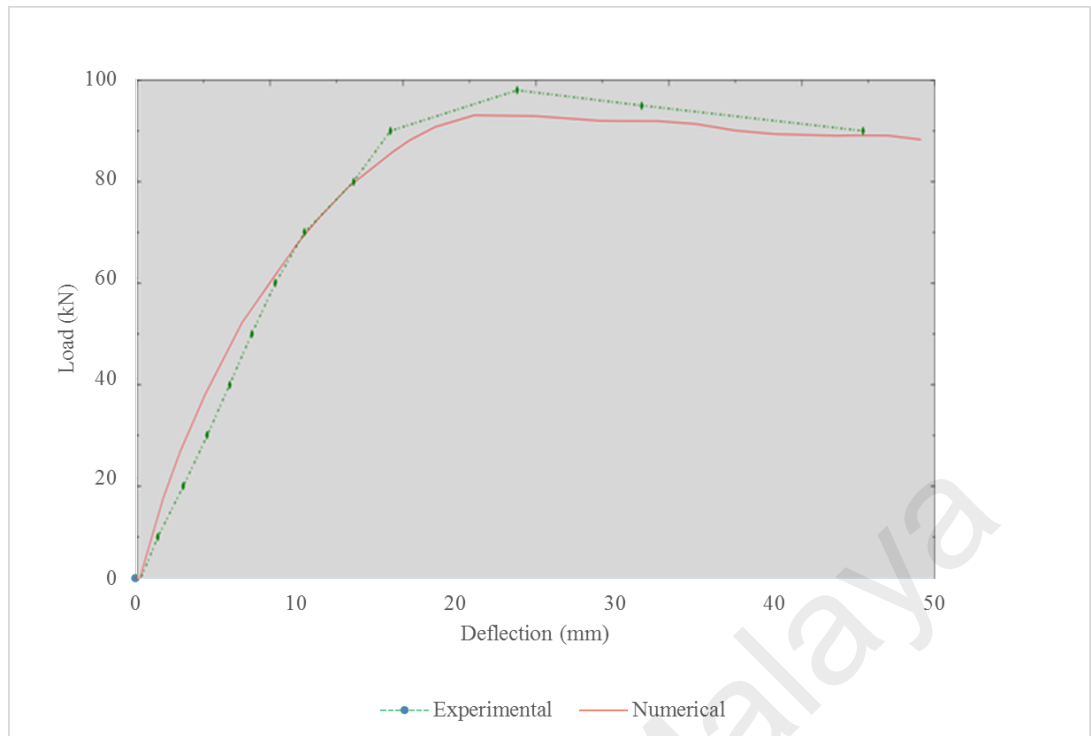


Figure 4.72 : Typical Load deflection diagram of NSM strengthened beam

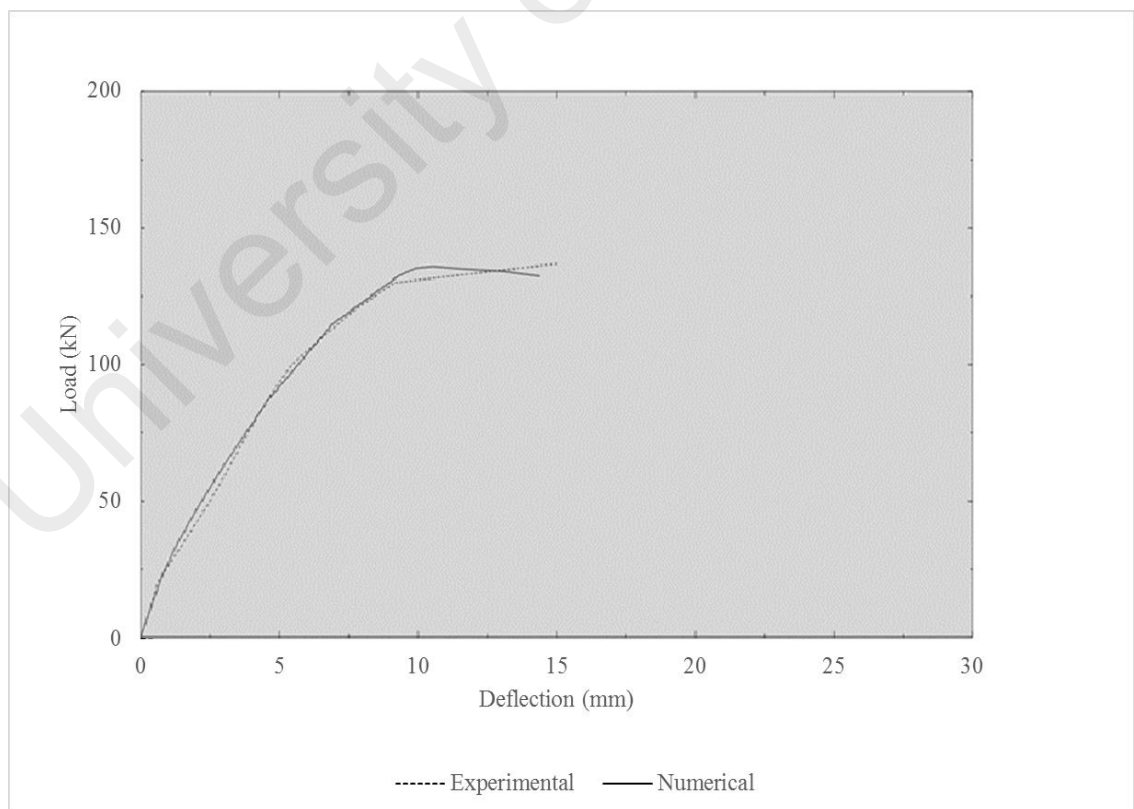


Figure 4.73: Typical Load deflection diagram of HSM strengthened beam

During the first stage, the displacement increases almost linearly with the load, the slopes of the curves are similar and Young's modulus has its greatest value. In the second stage, the cracks propagate and steel bars take the traction and there is noticeable non-linearity and irreversibility in beam property. Furthermore, the displacement increases faster than load which means a drop in Young's modulus. In other words, a reduction in the beam stiffness occurs.

To increase the bending resistance of the reinforced concrete beams, the steel or FRP plate is attached to the bottom of the beams in this section. The thickness of each FRP layer is 1.2 mm. The predicted ultimate load of 82 kN is in reasonable agreement with the experimental ultimate load 80 kN. Hence, the material constitutive models have been proven to be able to simulate the composite behaviour of reinforced concrete beams strengthened by FRP correctly.

4.4.3 Parametric Study using Finite Element Modelling

The effect of FRP plate thickness and length using finite element analysis is shown in Figure 4.74 and Figure 4.75. According to Figure 4.74, debonding failure load decreased with plate thickness and this trend is also similar to the experimental trend.

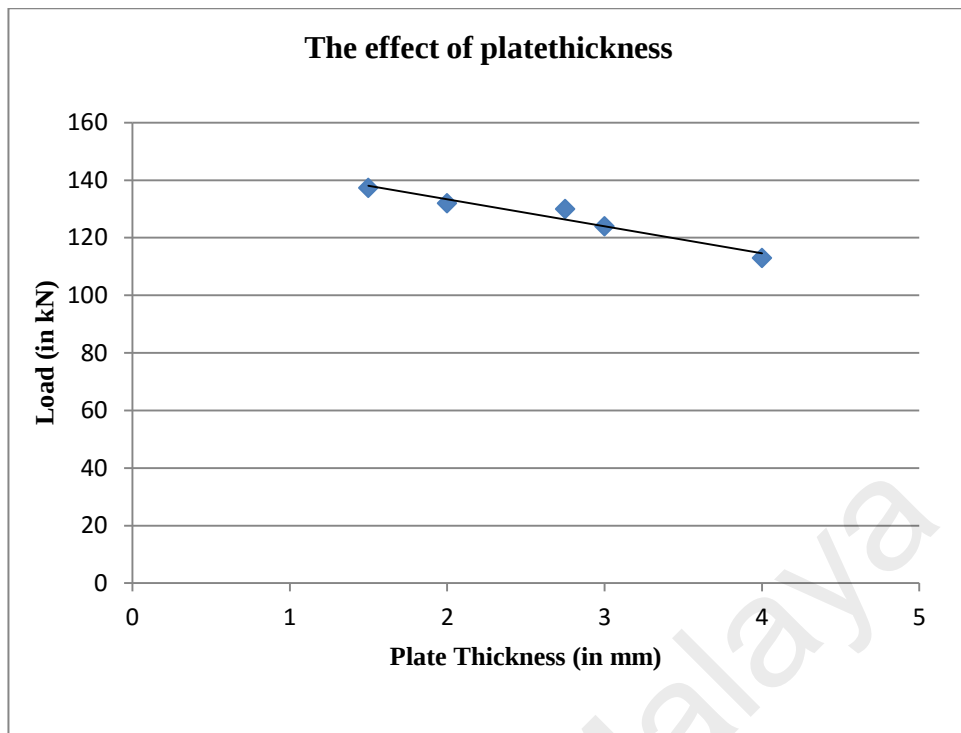


Figure 4.74: The effect of plate thickness using FEA

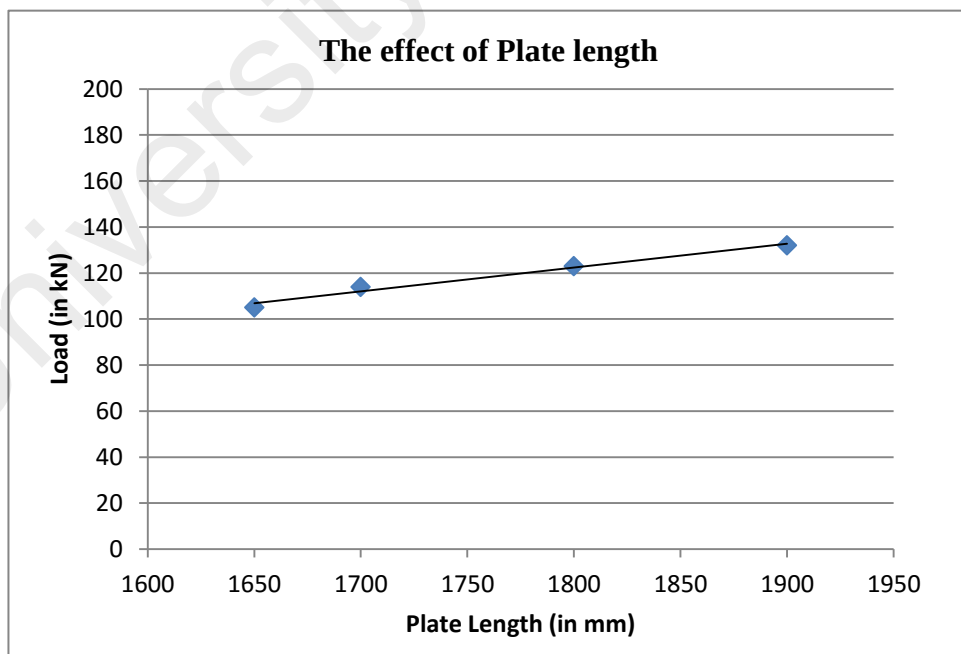


Figure 4.75: The effect of plate length using FEA

The ultimate load increased with plate length according to Figure 4.75, and this trend is also similar to the experimental observation and the trend in published literature. The findings of this investigation will be of interest to researchers and engineers looking to apply FRP composites in civil engineering applications, and may provide some implications for future design codes. All strengthened beams exhibited a higher load capacity and a lower ductility compared with their respective control beams. The non-linear three-dimensional finite element model proposed herein provides researchers and designers a computational tool for the design of FRP strengthened beams. Through FEA modelling, the reduction in deflection and the maximum improvement in strength due to different configurations of FRP can be obtained. With the proposed FEA modelling, it is possible to do trial and error to find an effective and reasonable retrofit scheme.

4.5 Solution of Mathematical Optimization

4.5.1 Non-linear Programming Solutions

The present study considers an example of optimization done by Arya et al. (2002). This study optimized the flexural strengthening (using CFRP) of a simply supported 9 m span RC beam. The beam was designed with a 350 mm wide (b) and 700 mm deep (h) concrete section and with 2532 mm² internal reinforcement to support characteristic dead (g_{initial}) and imposed loads (q_{initial}) of 15 kN/m and 21 kN/m, respectively. The beam was then strengthened to carry additional dead (g_{ad}) and imposed loads (q_{ad}) of 5 kN/m and 7 kN/m respectively. To achieve this, an externally bonded CFRP plate was attached to soffit of the beam. The common data used in this example are presented in Table 4.14.

Table 4.14 : The common data used for calculation

Materials properties and partial safety factors					
Concrete		Steel reinforcement		CFRP	
f_{cu}	40 MPa	f_y	460 MPa	f_{fk}	2500 MPa
E_c	31MPa	E_s	200 GPa	E_{fk}	65 GPa
ϵ_{cu}	0.0035	ϵ_y	0.002	ϵ_{fk}	0.013
γ_{mc}	1.5	γ_{ms}	1.15	γ_{mf}	1.4
				γ_{mm}	1.1

The problem in this example was solved using the non-linear programming approach used in the present research with previously mentioned parameters. From the analysis the optimum dimensions for the CFRP plate are given in Table 4.15 and 7.18 m length. The cost of this option (RM 5333) is RM 451 less than the option (RM 5785) made by Arya et al. (2002). Thus, the cost savings of the proposed optimization method would be 8.5% over the previous optimization model of Arya et al. (2002).

Table 4.15 : Result of FRP strengthening using non-linear programming

Design variable	Optimum value of continuous variables	Tradition value of the variable
Plate width, b (mm)	214.6	240.0
Plate height, h (mm)	1.44	1.40
Length of FRP plate (mm)	7180	7160
Total cost(RM)	5333	5785
Saving in cost	8.5%	0%

4.5.2 Genetic Algorithm Solutions

Applying genetic algorithms to the example problem from Arya et al. (2002) was solved using the previously mentioned parameters. The optimum dimensions for the CFRP plate are given Table 4.16 and corresponding length of 7.18 m. The cost of this option (RM 5357) is RM 428 less than the option (RM 5785) made by Arya et al. (2002).

The cost savings made from using the proposed genetic algorithms for optimization would be 8%. As the example from Arya et al. was taken from an academic journal that has been evaluated by a number of reviewers, the cost savings made from this optimization method using genetic algorithms would probably be even greater in the professional or practical field.

Table 4.16 : Result of FRP strengthening using the genetic algorithm

Design variable	Optimum value of continuous variables	Tradition value of the variable
Plate width, b (mm)	217	240
Plate height, h (mm)	1.42	1.40
Length of FRP plate (mm)	7180	7160
Total cost(RM)	5357.00	5785.00
Saving in cost	8.0%	0%

Regarding the technical performance criteria of the optimized plate dimensions, the resistance bending moment capacity of the optimized plate is equal to 737.31 kN-m which is slightly greater than the external moment (733.71 kN-m). The calculated longitudinal shear stress is equal to 0.78 N/mm² which is less than the allowable limit (0.8N/mm²) set against premature separation failure. In terms of serviceability, the estimated concrete stress (18.54 N/mm²) and steel stress (319 N/mm²) are significantly less than the allowable limits (24 N/mm² for concrete and 368 N/mm² for steel).

An important finding in this research is that if the cost of adhesive and surface preparation is ignored, the optimum dimensions of the CFRP plate is 256 mm wide and 1.21 mm deep with a corresponding length of 6.88 m long. It has been demonstrated that

the cost of the strengthening materials is an important consideration in any structural design process. This optimization procedure based on genetic algorithms can be an effective tool to make the design process more efficient and therefore lead to the proper and efficient use of structural strengthening materials.

4.6 Summary of the Results and Discussion

This section demonstrates how the results of this chapter are used to achieve the objective of this study. Most of objectives were successfully achieved through experimental investigation, analytical study, finite element modelling and mathematical optimization, as revealed in Table 4.17.

Since the cost of the steel bar and cement mortar is significantly lower than that of FRP bar and epoxy adhesive, the use of these material certainly reduces the cost of strengthening. On the other hand, the application of HSM increases the performance. Both reduction of cost and enhancement of performance help to achieve the goal of the research, i.e increase the efficiency of the structural strengthening system. Mathematical optimization further increases the efficiency by reducing the cost of the material. Therefore, the current goal of this study has been successfully achieved.

Table 4.17: Achievement of Objectives

Sl. No.	Objectives	Beam	How to achieve
1	Develop a strategy for eliminating premature failures of strengthened beams using hybrid strengthening method (HSM).	Beam Sl. No 10-15, 21-27 (13beams)	Effectiveness: The ultimate load capacity of all hybrid strengthened beam increased by 26%-72% respectively, compared to the control beam Efficiency: The ultimate load capacity of all hybrid strengthened beam increased by 6%-36% respectively, compared to the RC beam strengthened with EBR
		Beam Sl. No 10,11,21,22 (4 Beam) Compare to PS1, PS2, PF1, PF2	
		Beam Sl. No. 16 17 20, 26 19 28(6 beams)	i) Increase of internal shear strength: Not eliminated ii) Increase of plate width: Not eliminated iii) Use of End Anchor: Eliminated iv) Increase of external shear strength: Eliminated v) Use of side hybrid bond: Eliminated
2	Study the effectiveness of using cement mortar to replace epoxy and steel bar to replace FRP in NSM strengthening method.	Beam Sl. No 2-9 (8beams)	The ultimate load capacity of all NSM strengthened beam increased by 22.5%-46.8% respectively, compared to the control beam
3	investigate the fatigue performance of RC beams strengthened with HBR, EBR, and NSM	Beam Sl. No. 30-34 (5 beams)	Fatigue performance of strengthened beams are used to achieve this objective.
5	Develop a semi-numerical and finite element model (FEM) to predict flexural strength and deflection of RC beams strengthened using the HSM.	Beam Sl. No 10,11 (2 Beams)	The correlation between the experimental and predicted results from semi numerical mode is within a reasonable agreement that support objective four
		Beam Sl. No 10-28	The correlation between the experimental and predicted results from finite element model is within a reasonable agreement that support objective
7	Propose an economical approach for flexural strengthening of RC beams with CFRP plate based on non-linear and genetic algorithm.		Significant cost savings from optimization task of the efficient design method proved the achievement.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The study presents the results of the research under in developing a strategy for eliminating premature failure of strengthened RC beams, which was called HSM and in proposed in the study. This technique was shown to have helped in reducing the possibility of premature failure and more efficient compared to the existing strengthening techniques. In the study of replacing epoxy by cement mortar and FRP by steel bar promising results were also obtained. The fatigue performance of the hybrid strengthened RC beams were shown to be better than the other techniques. Semi-numerical and finite element models were developed to predict the flexural strength and deflection of RC beams strengthened with different techniques. Developed an easy, efficient, and direct closed-form solution model for optimization of design of steel and FRP strengthened RC beams using non-linear and genetic algorithms. Based on study carried out, the following conclusions can be drawn:

- i. Experimental result shows that strengthening with the HSM has been proven to be an effective alternative to the current strengthening techniques under monotonic and fatigue loadings.
- ii. The load carrying capacity of the HSM strengthened RC beam specimens increased by up to 65% for RC beam strengthened with steel plate and steel bar and 104% for RC beam strengthened with CFRP plate and steel bar.
- iii. The performance to increase load carrying capacity of the HSM strengthened beam to increase load carrying capacity was up to 36% higher than the corresponding EBR when the same amount of strengthening materials were used.
- iv. The performance of the bond between the concrete and the plate improved by 25% in the hybrid strengthening technique, even for the same plate thickness.

Separation or delamination of CFRP or steel plate from concrete substrate was successfully prevented due to this improved bond performance.

- v. The number of grooves adversely affected the performance of the HSM because of availability of sufficient beam width for providing enough space to make the grooves. Similarly, the effect of diameter of NSM bar on the strengthening performance of HSM beams is considerable.
- vi. The ductility of the HSM strengthened beams was found to be very similar to that of the un-strengthened control beams. Interestingly, the energy absorption capacity the HSM strengthened beams was significantly higher than that of the un-strengthened control beams due to higher ultimate and failure load.
- vii. The premature failure, i.e. delamination or concrete cover separation of HSM strengthened beams were successfully eliminated through decreasing the plate thickness, proper external shear strengthening, especially in cases of concrete cover separation, providing traditional end anchorage using CFRP wrapping, and changing the location of the bars and plates from soffit to sides.
- viii. Using NSM steel bars to strengthen RC beams is an economical alternative to strengthening with NSM FRP bars. The beams where 50% of the epoxy adhesive was replaced with cement mortar in the middle part of the NSM groove gave flexural performances almost similar to the performances of the beams using 100% epoxy adhesive.
- ix. The fatigue performance of the HSM strengthened beam was found to be higher than that of the NSM strengthened beam. In addition, the fatigue failure of the HSM strengthened beam was not found to be brittle or sudden compared with the NSM strengthened beam.
- x. The proposed semi-numerical model was shown to be an alternative computational method to the trial and error procedure for the design of an effective

and reasonable retrofit scheme. The results of the finite element models were found to be consistent with the experimental test results.

- xi. The non-linear programming and genetic algorithms provided a procedure that can be applied to produce economical solutions when designing FRP strengthening systems and this design process may lead to significant savings in the quantity of strengthening materials to be used in comparison to traditional design methods.

5.2 Recommendations

The present study illustrates the hybrid strengthening method (HSM) for strengthening RC beams and its practical suitability. HSM has huge potential for applications in structural strengthening. The following important recommendations are to be considered for future work in this area:

- i. The structural performance of RC beams flexurally strengthened with prestressed HSM using steel and FRP should be explored.
- ii. The flexural performance of prestressed beams strengthened with HSM using FRP and steel reinforcement should be studied.
- iii. The flexural behaviour of pre-cracked beams strengthened with HSM should also be investigated for their performance.
- iv. The fatigue performance of RC or prestressed beams strengthened with HSM using FRP or steel reinforcement should be tested.
- v. In this research, cement mortar used to replace 50% of epoxy adhesive. Future investigations are required to investigate the different percentages of replacement of epoxy adhesive by cement mortar.
- vi. Design guidelines need to be developed for the practical application of HSM.

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LIST OF PUBLICATIONS AND PAPERS PRESENTED

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- ii. Md. Moshiur Rahman , Mohd Zamin Jumaat, Md. Akter Hosen, A. B. M Saiful Islam (2015), “The effect of Replacement of Adhesive with Cement Mortar on the Performance of RC Beam with Near Surface Mounted Steel Bar” Accepted for publication in Accepted for publication in Revista de la Construcción. In press, corrected Proof.
- iii. Md. Moshiur Rahman , Mohd Zamin Jumaat, Md. Akter Hosen(2012), “Genetic Algorithm for Material Cost Minimization of External Strengthening System with Fiber Reinforced Polymer” Accepted for publication in Advanced Materials Research Journal.
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