Rehabilitation and Flexural Strengthening of Reinforced Concrete Beams using External Steel Reinforcement

by

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A thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering

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Declaration

This is to certify that the thesis work entitled "Rehabilitation and Flexural Strengthening of Reinforced Concrete Beams using External Steel Reinforcement" has been carried out by Md. Omar Ali Mondal in the Department of Civil Engineering, Khulna University of Engineering & Technology, Khulna, Bangladesh. The above thesis work or any part of this work has not been submitted anywhere for the award of any degree or diploma.

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Abstract

Rehabilitation of existing structures in the form of structural strengthening may be required due to decrease of load carrying capacity with aging, improper design or to accommodate with increased load requirements with time or codal change. Structural demolition can be reduced by applying strengthening technique to improve the capacity. This research investigated the flexural behavior of reinforced concrete (RC) beams rehabilitated with different strengthening techniques involving external steel reinforcement. The main focus of the study was to apply those strengthening techniques under service load condition. Twelve half-scaled beams were prepared and divided into six groups. The first group was used as control specimens while the other five groups were strengthened with different strengthening techniques. The control specimens were tested by 3rd point loading to find the ultimate load carrying capacity in flexure. Then all other beams from each of the five groups were preloaded with 65%-75% of the ultimate load to simulate the service load condition. Some initial cracks were formed in each beam due to preload before strengthening. After observation of the crack patterns, the preloading was released and the beams were ready for strengthening. Two groups were strengthened with 3mm thick external steel plate bonded with epoxy adhesive, of which the steel plate in one group was anchored by steel bolts in addition to the adhesive. Two different types of epoxy adhesive were used in two separate beams of each group. The 4th and 5th groups were strengthened with near surface mounted (NSM) external steel bars. The NSM bars in the 4th group were attached by epoxy adhesives while those in the 6th group were welded with the original bottom stirrups after removing the bottom concrete cover. The remaining group was strengthened by using external steel angle welded with the bottom stirrups after removal of the required concrete cover. Finally, the bottom concrete cover was cast again.

The average ultimate load carrying capacity of control beams in flexure was found to be 49.1kN. The ultimate capacity of strengthened beams with steel plate was observed to be 92.4kN for a specific adhesive type, which was as much as 88% higher than the control beams. The capacity was greatly influenced by the type of adhesives and the bond strength between steel plate and concrete. Anchoring the steel plate by steel bolts at both ends in addition to the epoxy adhesive further increased the capacity to 104% higher than the control beams and mode of failure switched from a brittle to a ductile nature. The capacity of beams

strengthened with NSM bars varied from 101.9kN to 115.0kN depending on the type of the adhesives. Flexural failure occurred either by the separation of NSM steel bars by bond failure or by pure bending. The beams strengthened with external steel angles welded with bottom stirrups showed an ultimate flexural load capacity of 124.4kN and 116.8kN, which were 153% and 138% higher, respectively than the control beams. A ductile behavior was obtained when the welded connections in this type of strengthening did not fail. The ultimate capacity was 127.5kN and 134kN for the beams strengthened with external steel bars welded with bottom stirrups which were 160% and 173% higher than the control beams and 95% and 100% of their designed strength. The initial flexural failure pattern in the unstrengthened beams was transformed to an obvious shear failure pattern in that case. After normalizing the experimental results with respect to external steel ratio and grade, it could be concluded that the NSM method of flexural strengthening was more effective in comparison to the other methods. On the other hand, strengthening with external steel plate was more convenient and easier to apply although its capacity is slightly lower than the NSM method.

Contents

		PAGE
	Cover Page	i
	Declaration	ii
	Approval	iii
	Acknowledgement	iv
	Abstract	v
	Contents	vii
	List of Tables	xi
	List of Figures	xii
	Nomenclature	xvi
CHAPTER I	INTRODUCTION	1
	1.1 General	1
	1.2 Objectives of the Study	2
	1.3 Scope of the Study	3
	1.4 Organization of Report	4
CHAPTER II	LITERATURE REVIEW	5
	2.1 General	5
	2.2 Rehabilitation	5
	2.2.1 Necessity of Structural Rehabilitation	5
	2.2.2 Repairing	6
	2.2.3 Retrofitting	6
	2.2.4 Strengthening	7
	2.2.5 Factors Affecting Selection of Rehabilitation Method	8
	2.2.6 Repairing Techniques	8
	2.2.6.1 Epoxy injection	9
	2.2.6.2 Routing and sealing	13
	2.2.6.3 Stitching	15
	2.2.6.4 Additional reinforcement	16
	2.2.6.5 Drilling and plugging	18

	2.2.6.6 Gravity filling	19
	2.2.6.7 Grouting	19
	2.2.6.8 Dry-packing	20
	2.2.6.9 Crack arrest	21
	2.2.6.10 Polymer impregnation	22
	2.2.6.11 Overlay and surface treatments	23
	2.2.6.12 Autogenous healing	24
	2.2.7 Retrofitting and Strengthening Techniques	24
	2.2.7.1 Section enlargement	24
	2.2.7.2 External plate bonding	25
	2.2.7.3 External post-tensioning	26
	2.2.7.4 Ferrocement laminates	27
	2.2.7.5 Sprayed concrete	28
	2.2.7.6 Strengthening using fiber reinforced polymer	28
	2.2.8 Strengthening of Concrete Structures	29
	2.2.8.1 Foundations	29
	2.2.8.2 Columns	29
	2.2.8.3 Load bearing walls	30
	2.2.8.4 Beams	30
	2.2.8.5 Masonry walls	30
	2.2.8.6 Slabs	31
	2.3 Strengthening of Reinforced Concrete Beams	32
	2.3.1 Strengthening with External Steel Plate	32
	2.3.2 Strengthening with Near Surface Mounted Bars	32
	2.3.3 Strengthening with External Steel Bars & Steel Angles	33
	2.3.4 Strengthening with HPFRCC	34
	2.3.5 Strengthening with CFRP Laminates Configurations	34
CHAPTER III	METHODOLOGY	35
	3.1 General	35
	3.2 Materials	35
	3.3 Specimen Preparation	35
	3.4 Background of Preloading	37

3.5 Design of Control Beams 37
3.6 Design of Strengthened Beams 39
3.6.1 Strengthening with External Steel Plate 40
3.6.2 Strengthening with External Steel Plate Anchorage by Steel Bolts 42
3.6.3 Strengthening with Near Surface Mounted Steel Bars 43
3.6.4 Strengthening with External Steel Angles 44
3.6.5 Strengthening with External Steel Bars 45
3.7 Determination of Capacity of Beams 46

CHAPTER IV EXPERIMENTAL PROGRAM 48
4.1 General 48
4.2 Materials Properties 48
4.2.1 Coarse Aggregate 48
4.2.2 Fine Aggregate 49
4.2.3 Binder 50
4.2.4 Reinforcing Steel 50
4.2.5 Steel Plate 51
4.2.6 Steel Angle 51
4.2.7 Adhesive 51
4.3 Sample Preparation 51
4.4 Concrete Mix Design 52
4.5 Casting and Curing 52
4.6 Determination of Compressive Strength 53
4.7 Flexural Strength Test of Control Beams 55
4.8 Preloading of Beams Before Strengthening 57
4.9 Strengthening of Beams 57
4.9.1 Strengthening with External Steel Plate 57
4.9.2 Strengthening with External Steel Plate Anchorage by Steel Bolts 58
4.9.3 Strengthening with Near Surface Mounted Steel Bars 58
4.9.4 Strengthening with External Steel Angles 59
4.9.5 Strengthening with External Steel Bars 60

	4.10 Test of Strengthened Beams	60
CHAPTER V	RESULTS & DISCUSSION	61
	5.1 General	61
	5.2 Beams Test Results	61
	5.2.1 Control Beams	62
	5.2.2 Application of Pre-load	64
	5.2.3 Beams Strengthened with External Steel Plate	66
	5.2.4 Beams Strengthened with External Steel Plate Anchorage by Steel Bolts	69
	5.2.5 Beams Strengthened with Near Surface Mounted Bars	72
	5.2.6 Beams Strengthened with External Steel Angles	75
	5.2.7 Beams Strengthened with External Steel Bars	78
	5.3 Comparison of Results	81
CHAPTER VI	CONCLUSION & RECOMMENDATION	90
	6.1 General	90
	6.2 Conclusions	90
	6.3 Recommendations	91
References		92
Appendix A		100

List of Tables

Table No	Description	Page
3.1	Design summary of all beams	36
4.1	Properties of aggregate	48
4.2	Different properties of binder	50
4.3	Tensile strength of steel bar, steel angle & steel plate	50
4.4	Properties of adhesives	51
4.5	Ingredients for concrete mixture	52
5.1	Summary of beam test results	61
5.2	Summary of preloading	66

List of Figures

Figure No	Description	Page
2.1	Structure prepared for epoxy injection through the ports shown	11
2.2	Repair of crack by sealing	13
2.3	Effect of bond breaking	14
2.4	Example of surface sealing for cracks subject to movement	15
2.5	Repair of crack by stitching	16
2.6	Reinforcing bar orientation used to effect the repair	17
2.7	Examples of external pre-stressing	17
2.8	Repair by drilling and plugging	18
2.9	Crack arrest method	22
3.1	Reinforcement detailing of prototype	37
3.2	Schematic diagram of control beams	39
3.3	Schematic diagram of beams strengthened with steel plate	40
3.4	Equivalent stress-strain diagram of beams strengthened with steel plate	41
3.5	Schematic diagram of beams strengthened with steel plate anchorage by bolts	42
3.6	Schematic diagram of beams strengthened with NSM bars	43
3.7	Schematic diagram of beams strengthened with steel angles	44
3.8	Schematic diagram of beams strengthened with steel bars	45
3.9	Applied load, shear force and bending moment diagram	46
4.1	Grain size distribution curve of coarse aggregate	49
4.2	Grain size distribution curve of fine aggregate	49
4.3	Typical steel case detailing	52
4.4	Casting of reinforced concrete beams	53

4.5	Curing of reinforced concrete beams	53
4.6	Illustration of compressive strength test	54
4.7	Compressive strength at different ages	55
4.8	Schematic diagram of the test setup	56
4.9	Experimental setup for the flexural test	56
4.10	Strengthening with external steel plate	57
4.11	Strengthening with external steel plate and bolts	58
4.12	Strengthening with NSM steel bars	59
4.13	Strengthening with external steel angles	59
4.14	Strengthening with external steel bars	60
5.1	Load versus deflection curve of control beam (B1-1)	62
5.2	Failure mode and crack pattern of control beam (B11)	63
5.3	Load versus deflection curve of control beam (B1-2)	63
5.4	Failure mode and crack pattern of control beam (B12)	64
5.5	Load versus mid-span deflection curve for application of 75% load	65
5.6	Load verses mid-span deflection curve for application of 65% load	65
5.7	Load versus deflection of beam strengthened with steel plate (B2-1)	67
5.8	Failure mode of beam strengthened with steel plate (B2-1)	67
5.9	Load versus deflection of beam strengthened with steel plate (B2-2)	68
5.10	Failure mode of beam strengthened with steel plate (B2-2)	69
5.11	Load versus deflection of beam strengthened with steel plate and bolts (B3-1)	70
5.12	Failure mode of beam strengthened with steel plate and bolts (B3-1)	70
5.13	Load versus deflection of beam strengthened with steel plate and bolts (B3-2)	71
5.14	Failure mode of beam strengthened with steel plate and bolts (B3-2)	72

5.15	Load versus deflection of beam strengthened with NSM bars (B4-1)	73
5.16	Failure mode of beam strengthened with NSM bars (B4-1)	73
5.17	Load versus deflection of beam strengthened with NSM bars (B4-2)	74
5.18	Failure mode of beam strengthened with NSM bars (B4-2)	75
5.19	Load versus deflection of beam strengthened with steel angles (B5-1)	76
5.20	Failure mode of beam strengthened with steel angles (B5-1)	76
5.21	Load versus deflection of beams strengthened with steel angles (B5-2)	77
5.22	Failure mode of beam strengthened with steel angles (B5-2)	78
5.23	Load versus deflection of beam strengthened with steel bars (B6-1)	79
5.24	Failure mode of beam strengthened with steel bars (B6-1)	79
5.25	Load versus deflection of beam strengthened with steel bars (B6-2)	80
5.26	Failure mode of beam strengthened with steel bars (B6-2)	81
5.27	Load versus mid-span deflection of control beams	82
5.28	Load versus mid-span deflection of beams strengthened with steel plate	83
5.29	Load versus mid-span deflection of beams strengthened with NSM bar	84
5.30	Load versus mid-span deflection of beams strengthened with steel angle	85
5.31	Load vs mid-span deflection of beams strengthened with steel bars	86
5.32	Relationship between calculated and experimental capacity	86
5.33	Capacity increased of strengthened beams	87
5.34	Experimental capacity of beams with calculated capacity	88
5.35	Comparison of load versus mid-span deflection of all six groups	88
5.36	Normalize results depending on external steel ratio	89
5.37	Normalize results depending on external steel ratio and steel grade	89
A-1	Load versus deflection of beam for preloading (B2-1)	100
A-2	Crack pattern for preloading (B2-1)	100
A-3	Load versus deflection of beam for preloading (B2-2)	101

A-4	Crack pattern for preloading (B2-2)	101
A-5	Load versus deflection of beam for preloading (B3-1)	102
A-6	Crack pattern for preloading (B3-1)	102
A-7	Load versus deflection of beam for preloading (B3-2)	103
A-8	Crack pattern for preloading (B3-2)	103
A-9	Load versus deflection of beam for preloading (B4-1)	104
A-10	Crack pattern for preloading (B4-1)	104
A-11	Load versus deflection of beam for preloading (B4-2)	105
A-12	Crack pattern for preloading (B4-2)	105
A-13	Load versus deflection of beam for preloading (B5-1)	106
A-14	Crack pattern for preloading (B5-1)	106
A-15	Load versus deflection of beam for preloading (B5-2)	107
A-16	Crack pattern for preloading (B5-2)	107
A-17	Load versus deflection of beam for preloading (B6-1)	108
A-18	Crack pattern for preloading (B6-1)	108
A-19	Load versus deflection of beam for preloading (B6-2)	109
A-20	Crack pattern for preloading (B6-2)	109

Nomenclature

a	Depth of equivalent rectangular stress block.
A_s	Flexural reinforcement area of beams.
$A_{s(c)}$	Flexural reinforcement area of control beams.
$A_{s(ext)}$	External flexural reinforcement area.
$A_{s(max)}$	Maximum area of flexural reinforcement.
$A_{s(min)}$	Minimum area of flexural reinforcement.
b	Width of beams.
c	Distance from extreme compression fiber to the neutral axis.
d	Distance from extreme compression fiber to centroid of main flexural reinforcement.
d_1	Distance from extreme compression fiber to centroid of external flexural reinforcement.
d_{ave}	Average distance from extreme compression fiber to centroid of flexural reinforcement.
f_c'	Compressive strength of concrete at 28 days curing.
f_y	Yield strength of flexural reinforcement.
$f_{y(c)}$	Yield strength of flexural reinforcement of control beams.
$f_{y(ext)}$	Yield strength of external reinforcement.
F_s	Tension in flexural reinforcement developed by a given bending moment (tension is positive).

F_{es} Tension in external flexural reinforcement developed by a given bending moment (tension is positive).

F_{is} Tension in internal flexural reinforcement developed by a given bending moment (tension is positive).

L Span length of beams.

M_n Nominal moment capacity of beams.

$M_{n(c)}$ Moment capacity of control beams in tension.

$M_{n(max)}$ Maximum moment capacity of strengthened beams in tension.

$M_{n(total)}$ Moment capacity of strengthened beams in tension.

$M_{u(c)}$ Moment capacity of beams in compression.

P Ultimate load bearing capacity of beams.

V Maximum shear force.

w Self-eight of beams per unit length.

β_1 Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth.

$\Delta\varepsilon_c$ Change in strain level in concrete substrate developed by a given bending moment (tension is positive).

$\Delta\varepsilon_s$ Change in strain level in main flexural reinforcement developed by a given bending moment (tension is positive).

ρ Flexural reinforcement ratio.

$\rho_{balance}$ Balance reinforcement ratio in flexure.

ρ_{max} Maximum reinforcement ratio in flexure.

ε_c	Strain level in concrete substrate developed by a given bending moment (tension is positive).
ε_{co}	Strain level in concrete substrate developed by a given bending moment (tension is positive) by preloading.
ε_s	Strain level in flexural reinforcement of control beams developed by a given bending moment (tension is positive).
ε_{so}	Strain level in flexural reinforcement developed by a given bending moment (tension is positive) by preloading.
ε_{es}	Strain level in internal flexural reinforcement developed by a given bending moment (tension is positive).
ε_{is}	Strain level in internal flexural reinforcement developed by a given bending moment (tension is positive).

CHAPTER I

INTRODUCTION

1.1 General

Concrete has been used practically since primitive times. Since the innovation of reinforced concrete in the middle of the 19^{th} century, it was greatly influenced by the development of new structures (Bosc et al., 2001). As a result, reinforced concrete has become a very popular material and nowadays most constructions are made of this composite material.

Generally, structural elements are designed to carry various types of loadings. Strengthening of RC structures are required due to overloading, corrosion of the steel reinforcement, inadequate maintenance, change in use or change in the code of practice, and/or exposure to unfavorable conditions like earthquakes and blasts. Structural strengthening allows existing underperforming structures to survive against additional service load requirements, design, or construction error and structural deterioration due to age or the surrounding adverse environment.

Beam is a structural element that primarily resists loads applied laterally to the beam's axis. Its mode of deflection is primarily by bending (Beam, 2018). The loads applied to the beam result in reaction forces at the beam's support points. The total effect of all the forces acting on the beam is to produce shear forces and bending moments within the beam, that in turn induce internal stresses, strains and deflections of the beam. Beams are characterized by their manner of support, profile (shape of the cross-section), length, and material.

Structural members, however may require upgrading or strengthening due to various reasons including, human error, structural design and/or construction, amendments in practicing design standards/codes, structural deterioration due to ageing and environmental exposures, abusive use of buildings in the form of change in the utility of the structure resulting in an increase in the live load and stress concentration in structural members (Khan, Rafeeqi & Ayub, 2013).

Natural disasters such as earthquakes, tornados, and tsunamis threaten the integrity of civil infrastructure and the safety of their users. A large number of reinforced concrete buildings and bridges built in early ages typically do not have sufficient capacity to resist the forces during such catastrophes. In order to guarantee the safety of the people; older, existing structures need to be repaired and strengthened to prevent their collapse. Strengthening of beams required to eliminate structural problems or distress which results from unusual loading or exposure conditions, inadequate design or poor construction practices.

Different methods are available for the strengthening of RC beams such as strengthening with Fiber Reinforced Polymer (FRP), strengthening with High-Performance Fiber Reinforced Cementitious Composites (HPFRCC), strengthening with external plate bonding, strengthening with Near Surface Mounted (NSM) steel bar / FRP rod and strengthening with external steel reinforcement. Fiber Reinforced Polymer (FRP) composites are efficient strengthening materials having their several properties, such as high strength and good bonding quality with concrete (Tankut & Arslan, 1992). However, their brittle stress-strain behavior limits the overall ductility of beams. Furthermore, FRP composites are costlier compared to other strengthening materials (steel, RC, Shotcrete, etc.) and their fire resistance is low.

Different researchers investigate different methods for flexural strengthening of RC beams with different parameter. Most of those researches are done with newly constructed RC beams also, they are not analysis the capacity of strengthened beams. Most of the structures or structural elements are required strengthening in its service life after a certain period after construction. To investigate the effect of service load on the flexural strengthening of RC beams, preloading is applied before the application of different strengthening methods. To find the effectiveness of the strengthening method an analytical analysis is carried out in this study. This study aims to investigate different methods for rehabilitation and flexural strengthening of RC beams using external steel reinforcement (steel plate, steel angle and steel bar).

1.2 Objectives of the Study

The main objectives of this research work are outlined below:

- To investigate the flexural performance of RC beams after strengthening by using external steel reinforcement after preloading.
- To compare the load carrying capacity of the beams strengthened by different strengthening techniques.

The objectives are attained by the following tasks:

- Flexural strengthening of RC beams by using external steel plate bonded with epoxy.
- Flexural strengthening of RC beams by using external steel plate anchored by steel bolts.
- Flexural strengthening of RC beams by using Near Surface Mounted (NSM) steel bars.
- Flexural strengthening of RC beams by using external steel angles.
- Flexural strengthening of RC beams by using external steel bars.

1.3 Scope of the Study

Bangladesh is suffering from disasters such as Cyclones, Floods, Storm Surges and Tornados including Earthquakes regularly. Out of 5,000 public buildings in Bangladesh, around 3,000 were constructed before 1993 when Bangladesh National Building Code (BNBC 1993) was enacted (Bangladesh, PWD, MHPW, 2015). These buildings have low resistant ability against earthquakes. According to the report of PWD, if an earthquake of M7.5 at Madhupur Fault in the Dhaka suburb occurs, the damage estimation for the Dhaka city became VIII of MMI seismic intensity scale, and out of the total 326,000 buildings, 72,000 buildings will be damaged beyond repair. About 50% of them would be reinforced concrete and about 30 percent would be brick masonry buildings. In addition, moderately damaged buildings are estimated to be 49%. Further, if the earthquake occurs at 2:00 am, about 90 thousand people will die. Also, most of the buildings in Bangladesh are low-rise buildings. With the rapid development of construction, the land becomes more and more scarce and the construction of the new structure is quite expensive. Under such situations, maintenance of the buildings construction quality and improvement of the safety of the buildings are absolutely necessary for Bangladesh. To overcome this upcoming hazard, the structures are required modification or strengthening to satisfy current building code requirements. As a structural element beams are also required to strengthening. This research can be help to find a cost-effective strengthening technique for the strengthening of reinforced concrete beams in flexure.

1.4 Organization of Report

This study comprises of five chapters including Introduction in **Chapter I**. A review of related previous studies presented in **Chapter II**. In **Chapter III** methodology of different strengthening techniques, the design of specimens, and mathematical investigation procedure are described. Experimental program including preparation of specimens, the test of control specimens, preloading before strengthening, strengthening procedure and finally the test of strengthened beams are described in **Chapter IV.** Findings of this study (Results & Discussions) are presented in **Chapter V**. Summary of the findings of the study and some recommendations for future research are presented in **Chapter VI**.

CHAPTER II

LITERATURE REVIEW

2.1 General

Literature review is the most important work before an experimental study to know the limitation and scope of the research. In this chapter, a detailed literature review on the strengthening of reinforced concrete beams using external steel reinforcement is presented. Firstly, some important definitions are described related to rehabilitation. Then a brief introduction of different available external strengthening methods for RC beams is given. Finally, a detailed review of previous researches on the behavioral characteristics of RC beams strengthened with different techniques under flexure is presented.

2.2 Rehabilitation

Structural rehabilitation represents an important aspect of the construction industry and its significance is increasing (Júlio, Branco & Silva, 2003). Structural rehabilitation may be defined as an upgrade to meet the present requirements being sensitive to building features and a sympathetic matching of the original construction or the process of repairing or modifying a structure to a desired useful condition. Structural rehabilitation can be done by three different aspects known as repair, retrofitting and strengthening.

2.2.1 Necessity of Structural Rehabilitation

Reinforced concrete (RC) structures show excellent performance in terms of structural behavior and durability except for the zones that are exposed to severe environmental influences and high mechanical loading. Rehabilitation of deteriorated concrete structures is a heavy burden from the socio-economic viewpoint since it leads to significant user costs. As a consequence, novel concepts for the rehabilitation of concrete structures must be developed (Brühwiler & Denarié, 2013). Structural rehabilitation may be required at any time from the beginning of the construction phase until the end of the service life. According to Júlio et al. (2003) during the construction phase, it may occur because of

- design errors
- deficient concrete production

- bad execution processes

During the service life, it may arise on account of:

- an earthquake
- an accident, such as collisions, fire, explosions
- situations involving changes in the structure functionality
- the development of more demanding code requirements.

2.2.2 Repairing

The purpose of repair is to rectify the observed defects and bring the building to reasonable architectural shape so that all services start functioning. This enables the use of building for its desired purpose. Repair does not improve structural strength or stability. In fact, a repaired building may be deceptive. It may hide the structural defects. Outwardly it may appear good but may suffer from structural weakness. Repairs include the following interventions:

- Patching cracks and plastering.
- Fixing doors, windows, broken glass panes, etc.
- Rebuilding non-structural walls, partition walls, plastering, etc.
- Providing decorative finishes, whitewashing.
- Re-fixing roof tiles
- Painting woodwork, attending to roof leakage during rain, etc.

The need for structural repairs can arise from any of the following:

- Faulty design of the structure
- Improper execution and bad workmanship
- Extreme weathering and environmental conditions
- High degree of chemical attack
- Aging of the structure

2.2.3 Retrofitting

Retrofitting is the process to restitution of the strength of the structures or structural elements had before the damage occurred. The retrofitting is performed to regain the strength of the existing building to the original strength. This type of action should be undertaken once there's proof that the structural damage may be attributed to exceptional phenomena that don't seem to be seemingly to happen. The main purpose is to structurally treat the building with an aim to regain its original strength (Sarkar, 2006). This intervention is undertaken for a damaged building. The action will involve cutting portion of walls and rebuilding them, inserting supports, underpinning foundation, strengthening a weak component, etc. The retrofitting of a structure involves improving its performance under earthquake loadings through one or more of these following measures (Shrestha, Pribadi, Kusumastuti & Lim, 2009).

- Increasing its strength and/or stiffness
- Increasing its ductility
- Reducing the seismic forces.

2.2.4 Strengthening

Strengthening of concrete structures is a crucial task within the field of structural engineering. The aim of strengthening is to extend the capability of an existing structural component. Structural elements need to be strengthened because of a number of factors including the increase in loads as a result of functional changes of the structures, overloading, under-designed of existing structural elements or to the lack of quality control (Jumaat & Alam, 2009). In a broad sense, it may be defined as:

- An improvement over the original strength.
- Increase in lateral strength in one or both directions.
- Eliminating features that are sources of weakness or that produce a concentration of stress in some members.
- Avoiding the likelihood of brittle modes of failure by necessary reinforcement and association of resisting members.
- Repair refers to restoring but not increasing original performance after damage.

This work is very important since many civil structures are no longer evaluate safe which can be due to increased load requirements in the design codes, overloading, under-design of existing structural elements or to the lack of quality control. In order to keep efficient serviceability, structures which made a long time ago must be repaired or strengthened so that they can meet the same requirements of structures built today and in the future. It is also becoming both environmentally and economically preferable to repair or strengthening the structures rather than to replace them totally, particularly if rapid, effective and simple strengthening methods are available (Jumaat & Alam, 2007). So, a lot of reasons may be claimed for strengthening existing structural members. It is summarized as follows:

- To eliminate structural problems or distress which results from unusual loading or exposure conditions, inadequate design or poor construction practices.
- To conform to current codes and standards.
- To allow the feasibility of changing the structure to accommodate a different use from the present one.
- Durability problem due to poor and inappropriate construction materials.
- Design or construction errors.
- Aggressive environment not properly understood during the design stages.
- Increased life-span demands made on aging infrastructures.
- Exceptional or accidental loading.
- Varying life span of different structural or non-structural components.

2.2.5 Factors Affecting Selection of Rehabilitation Method

- Magnitude of Rehabilitation
- Size of project
- Environmental conditions
- Dimensional/clearance constraints
- Accessibility
- Operational constraints
- Availability of materials, equipment, and qualified contractors.
- Construction cost, maintenance costs, and life-cycle costs

2.2.6 Repairing Techniques

There are many techniques available for repairing the concrete structure. Issa (2009) described various technique for repairing a crack that’s are described below.

2.2.6.1 Epoxy injection

Cracks as narrow as 0.05mm can be bonded by the injection of epoxy. The technique generally consists of establishing entry and venting ports at close intervals along the cracks, sealing the crack on exposed surfaces, and injecting the epoxy under pressure.

Epoxy injection has been successfully used in the repair of cracks in buildings, bridges, dams, and other types of concrete structures (ACI Committee 503R). However, unless the cause of the cracking has been corrected, it will probably recur near the original crack. If the cause of the cracks cannot be removed, then two options are available. One is to rout and seal the crack, thus treating it as a joint. The second is to establish a joint that will accommodate the movement and then inject the crack with epoxy or other suitable material. Epoxy materials used for structural repairs should conform to ASTM C 881 (Type IV). ACI Committee 504R describes practices for sealing joints, including joint design, available materials, and methods of application.

With the exception of certain moisture-tolerant epoxies, this technique is not applicable if the cracks are actively leaking and cannot be dried out. Wet cracks can be injected using moisture-tolerant materials, but contaminants in the cracks (including silt and water) can reduce the effectiveness of the epoxy for structurally repairing the cracks. The use of a low-modulus, flexible adhesive in a crack will not allow significant movement of the concrete structure. The effective modulus of elasticity of a flexible adhesive in a crack is substantially the same as that of a rigid adhesive (Adams and Wake, 1984) because of the thin layer of material and high lateral restraint imposed by the surrounding concrete. Epoxy injection requires a high degree of skill for satisfactory execution, and application of the technique may be limited by the ambient temperature. The general procedures involved in epoxy injection are as follows (ACI Committee 503R).

❖ ***Clean the cracks***

The first step is to clean the cracks that have been contaminated, to the extent that this is possible and practical. Contaminants such as soil, grease, dirt, or fine particles of concrete prevent epoxy penetration and bonding and reduce the effectiveness of repairs. Preferably, contamination should be removed by vacuuming or flushing with water or other particularly effective cleaning solutions.

The solution is then flushed out using compressed air and a neutralizing agent or adequate time is provided for air drying. It is important, however, to recognize the practical limitations of accomplishing complete crack cleaning. A reasonable evaluation should be made of the extent, and necessity, of cleaning. Trial cleaning may be required.

❖ ***Seal the surfaces***

Surface cracks should be sealed to keep the epoxy from leaking out before it has gelled. Where the crack face cannot be reached, but where there is backfill, or where a slab-on-grade is being repaired, the backfill material or sub-base material is sometimes an adequate seal; however, such a condition can rarely be determined in advance, and uncontrolled injection can cause damage such as plugging a drainage system. Extreme caution must therefore be exercised when injecting cracks that are not visible on all surfaces. A surface can be sealed by applying an epoxy, polyester, or other appropriate sealing material to the surface of the crack and allowing it to harden. If a permanent glossy appearance along the crack is objectionable and if high injection pressure is not required, a strippable plastic surface sealer may be applied along the face of the crack. When the job is completed, the surface sealer can be stripped away to expose the gloss-free surface. Cementitious seals can also be used where the appearance of the completed work is important. If extremely high injection pressures are needed, the crack can be cut out to a depth of 13mm and width of about 20mm in a V-shape, filled with an epoxy, and struck off flush with the surface.

❖ ***Install the entry and venting ports***

Typical settings for entry and venting ports are shown in Figure 2.1. Three methods are in general use:

Fittings inserted into drilled holes - This method was the first to be used and is often used in conjunction with V-grooving of the cracks. The method entails drilling a hole into the crack, approximately 20mm in diameter and 13–25mm below the apex of the V-grooved section, into which a fitting such as a pipe nipple or tire valve stem is usually bonded with an epoxy adhesive. A vacuum chuck and bit, or a water-cooled core bit, is useful in preventing the cracks from being plugged with drilling dust.

Bonded flush fitting - When the cracks are not V-grooved, a method frequently used to provide an entry port is to bond a fitting flush with the concrete face over the crack. The flush fitting has an opening at the top for the adhesive to enter and a flange at the bottom that is bonded to the concrete.

Figure 2.1: Structure prepared for epoxy injection through the ports shown

An interruption in seal - Another system of providing entry is to omit the seal from a portion of the crack. This method can be used when special gasket devices are available that cover the unsealed portion of the crack and allow injection of the adhesive directly into the crack without leaking.

❖ ***Mix the epoxy***

This is done either by batch or continuous methods. In batch mixing, the adhesive components are premixed according to the manufacturer's instructions, usually with the use of a mechanical stirrer, like a paint mixing paddle. Care must be taken to mix only the amount of adhesive that can be used prior to the commencement of gelling of the material. When the adhesive material begins to gel, its flow characteristics begin to change, and pressure injection becomes more and more difficult. In the continuous mixing system, the two liquid adhesive components pass through metering and driving pumps prior to passing through an automatic mixing head. The continuous mixing system allows the use of fast-setting adhesives that have a short working life.

- ***Inject the epoxy***

Hydraulic pumps, paint pressure pots, or air-actuated caulking guns may be used. The pressure used for injection must be selected carefully. Increased pressure often does little to accelerate the rate of injection. In fact, the use of excessive pressure can propagate the existing cracks, causing additional damage. If the crack is vertical or inclined, the injection process should begin by pumping epoxy into the entry port at the lowest elevation until the epoxy level reaches the entry port above. The lower injection port is then capped, and the process is repeated until the crack has been completely filled and all ports have been capped. For horizontal cracks, the injection should proceed from one end of the crack to the other in the same manner. The crack is full if the pressure can be maintained. If the pressure cannot be maintained, the epoxy is still flowing into unfilled portions or leaking out of the crack.

- ***Remove the surface seal***

After the injected epoxy has cured, the surface seal should be removed by grinding or by other means as appropriate.

- ***Alternative procedure***

For massive structures, an alternative procedure consists of drilling a series of holes (usually 20–100mm diameter) that intercepts the crack at a number of locations. Typically, holes are spaced at 1.5m intervals. Another method recently being used is a vacuum or vacuum assist method. There are two techniques: one is to entirely enclose the cracked member with a bag and introduce the liquid adhesive at the bottom and to apply a vacuum at the top. The other technique is to inject the cracks from one side and pull a vacuum from the other.

Typically, epoxies are used; however, acrylics and polyesters have proven successful. Stratton and McCollum (1974) describe the use of epoxy injection as an effective intermediate-term repair procedure for delaminated bridge decks. As reported by Stratton and McCollum, the first, second, and sixth steps are omitted and the process is terminated at a specific location when epoxy exits from the crack at some distance from the injection ports. This procedure does not arrest on-going corrosion. The procedure can also be attempted for other applications, and is available as an option, although is not accepted universally. The success of the repair depends on the absence of bond-inhibiting contaminants from the crack plane. Epoxy resins and injection procedures should be carefully selected when attempting to inject delamination. Unless there is sufficient depth or anchorage to surrounding concrete, the injection process can be unsuccessful or increase the extent of delamination.

2.2.6.2 Routing and sealing

Routing and sealing of cracks can be used in conditions requiring remedial repair and where structural repair is not necessary. This method involves enlarging the crack along its exposed face and filling and sealing it with a suitable joint sealant (Figure 2.2). This is a common technique for crack treatment and is relatively simple in comparison with the procedures and the training required for epoxy injection. The procedure is most applicable to relatively flat horizontal surfaces such as floors and pavements. However, routing and sealing can be accomplished on vertical surfaces with a non-sag sealant as well as on curved surfaces (pipes, piles, and poles).

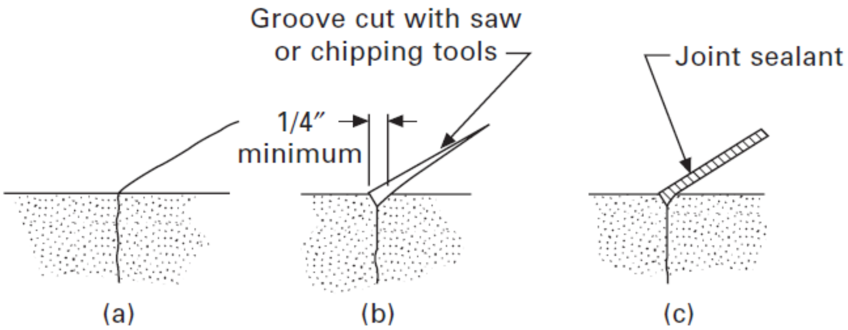

Figure 2.2: Repair of crack by sealing: (a) original crack; (b) routing; (c) sealing (Johnson, 1965)

Routing and sealing are used to treat both fine pattern cracks and larger, isolated cracks. A common and effective use is for waterproofing by sealing cracks on the concrete surface where water stands, or where hydrostatic pressure is applied. This treatment reduces the ability of moisture to reach the reinforcing steel or pass through the concrete, causing surface stains or other problems.

The sealants may be any of several materials, including epoxies, urethanes, silicones, polysulfide's, asphaltic materials, or polymer mortars. Cement grouts should be avoided due to the likelihood of cracking. For floors, the sealant should be sufficiently rigid to support the anticipated traffic. Satisfactory sealants should be able to withstand cyclic deformations and should not be brittle.

The procedure consists of preparing a groove at the surface ranging in depth, typically, from 6–25mm. A concrete saw hand tools or pneumatic tools may be used. The groove is then cleaned by air blasting, sandblasting, or water blasting, and dried. A sealant is placed into the dry groove and allowed to cure.

A bond breaker may be provided at the bottom of the groove to allow the sealant to change shape, without a concentration of stress on the bottom (Figure 2.3). The bond breaker may be a polyethylene strip or tape which will not bond to the sealant. Careful attention is required when detailing the joint so that its width to depth aspect ratio will accommodate anticipated movement (ACI Committee 504R).

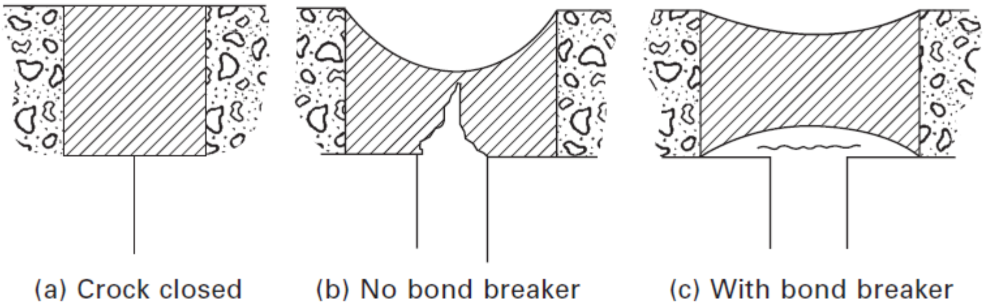

Figure 2.3: Effect of bond breaking (ACI Committee 224.1R)

In some cases, over-banding (strip coating) is used independently of, or in conjunction with, routing and sealing. This method is used to enhance protection against edge spalling and for aesthetic reasons to create a treatment with a more uniform appearance. A typical procedure for over-banding is to prepare an area approximately 25–75mm on each side of the crack by sandblasting or other means of surface preparation and to apply a coating (such as urethane) 1–2mm thick in a band over the crack. Before over-banding in non-traffic areas, a bond breaker is sometimes used over a crack that has not been routed, or over a crack previously routed and sealed. In traffic areas, a bond breaker is not recommended. Cracks subject to minimal movement may be over-banded (Figure 2.4) but, if the significant movement can take place, routing and sealing must be used in conjunction with over-banding to ensure a waterproof repair.

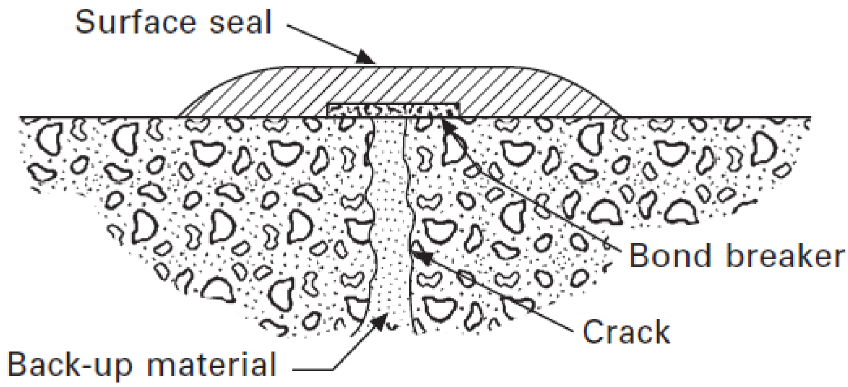

Figure 2.4: Example of surface sealing for cracks subject to movement (ACI Committee 224.1R).

2.2.6.3 Stitching

Stitching involves drilling holes on both sides of the crack and grouting in inverted U-shaped metal units with short legs (staples or stitching dogs) that span the crack as shown in Figure 2.5 (Johnson, 1965). Stitching may be used when tensile strength must be re-established across major cracks (Hoskins, Fowler, & McCullough, 1991). Stitching a crack tends to stiffen the structure, and the stiffening may increase the overall structural restraint, causing the concrete to crack elsewhere. Therefore, it may be necessary to strengthen the adjacent section or sections using appropriate reinforcing methods. Because stresses are often concentrated, using this method in conjunction with other methods may be necessary. The stitching procedure consists of drilling holes on both sides of the crack, cleaning the holes, and anchoring the legs of the staples in the holes, with either a non-shrink grout or an epoxy resin-based bonding system. The staples should be variable in length, orientation, or both and they should be located so that the tension transmitted across the crack is not applied to a single plane within the section but is spread over an area.

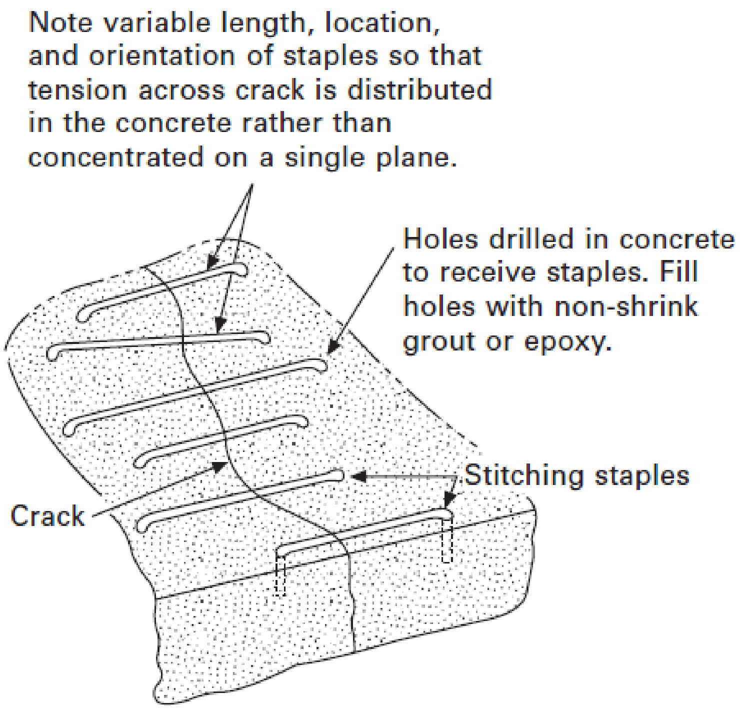

Figure 2.5: Repair of crack by stitching (Johnson, 1965)

2.2.6.4 Additional reinforcement

❖ ***Conventional reinforcement***

Cracked reinforced concrete bridge girders have been successfully repaired by inserting reinforcing bars and bonding them in place with epoxy (Stratton et al., 1978, 1982; Stratton, 1980). This technique consists of sealing the crack, drilling holes that intersect the crack plane at approximately 90° (Figure 2.6), filling the hole and crack with injected epoxy, and placing a reinforcing bar into the drilled hole. Typically, No. 4 or 5 (10 M or 15 M) bars are used, extending at least 18 in. (0.5m) each side of the crack. The reinforcing bars can be spaced to suit the needs of the repair. They can be placed in any desired pattern, depending on the design criteria and the location of the in-place reinforcement. The epoxy bonds the bar to the walls of the hole fills the crack plane, bonds the cracked concrete surfaces back together in one monolithic form, and thus reinforces the section. The epoxy used to re-bond the crack should have a very low viscosity and conform to ASTM C 881 Type IV.

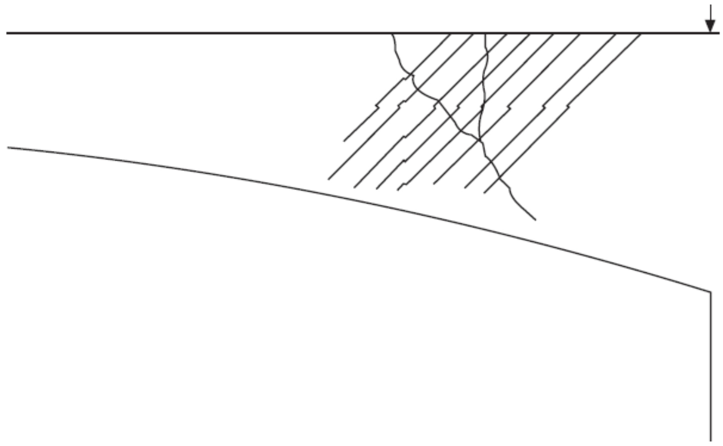

Figure 2.6: Reinforcing bar orientation used to effect the repair (Stratton Et Al., 1978)

❖ ***Pre-stressing steel***

Post-tensioning is often the desirable solution when a major portion of a member must be strengthened or when the cracks that have formed must be closed (Figure 2.7). This technique uses pre-stressing strands or bars to apply a compressive force. Adequate anchorage must be provided for the pre-stressing steel, and care is needed so that the problem will not merely migrate to another part of the structure.

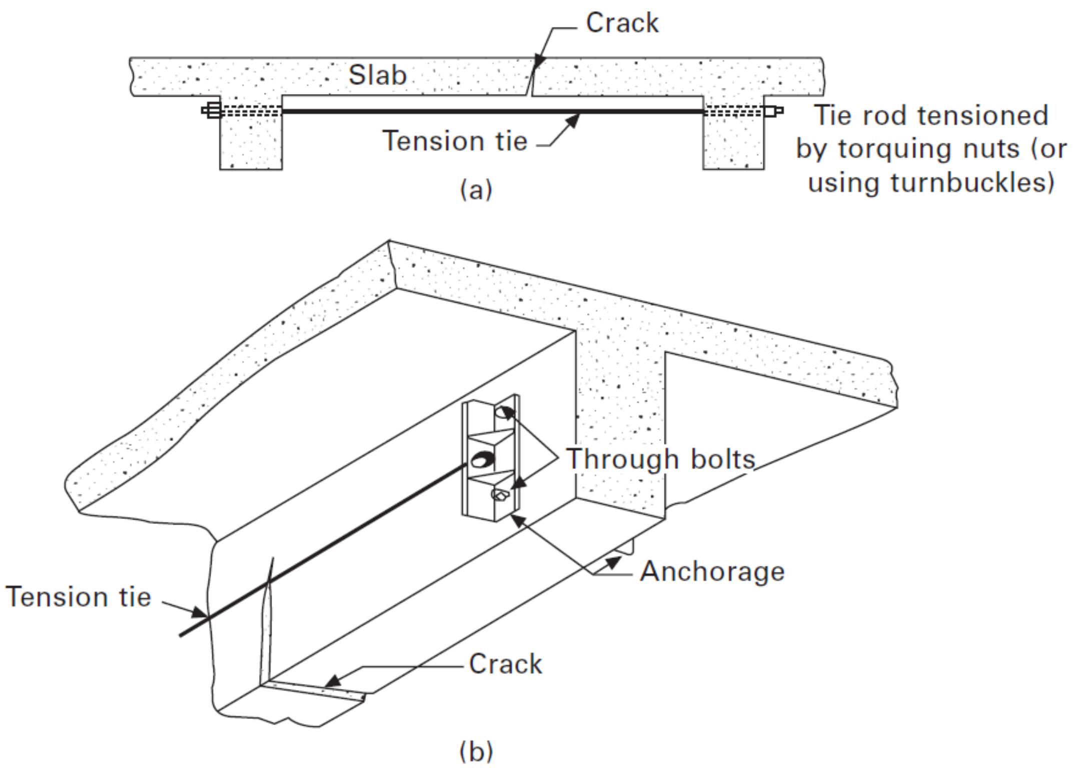

Figure 2.7: Examples of external pre-stressing: (a) to correct cracking of slab; (b) to correct cracking of beams (Johnson, 1965).

The effects of the tensioning force (including eccentricity) on the stress within the structure should be carefully analyzed. For indeterminate structures post-tensioned using this procedure, the effects of secondary moments and induced reactions should be considered (Lin and Burns, 1981; Nilson, 1987).

2.2.6.5 Drilling and plugging

Drilling and plugging a crack consists of drilling down the length of the crack and grouting it to form a key (Figure 2.8). This technique is only applicable when cracks run in reasonably straight lines and are accessible at one end. This method is most often used to repair vertical cracks in retaining walls. A hole [typically 50–75mm in diameter] should be drilled, centered

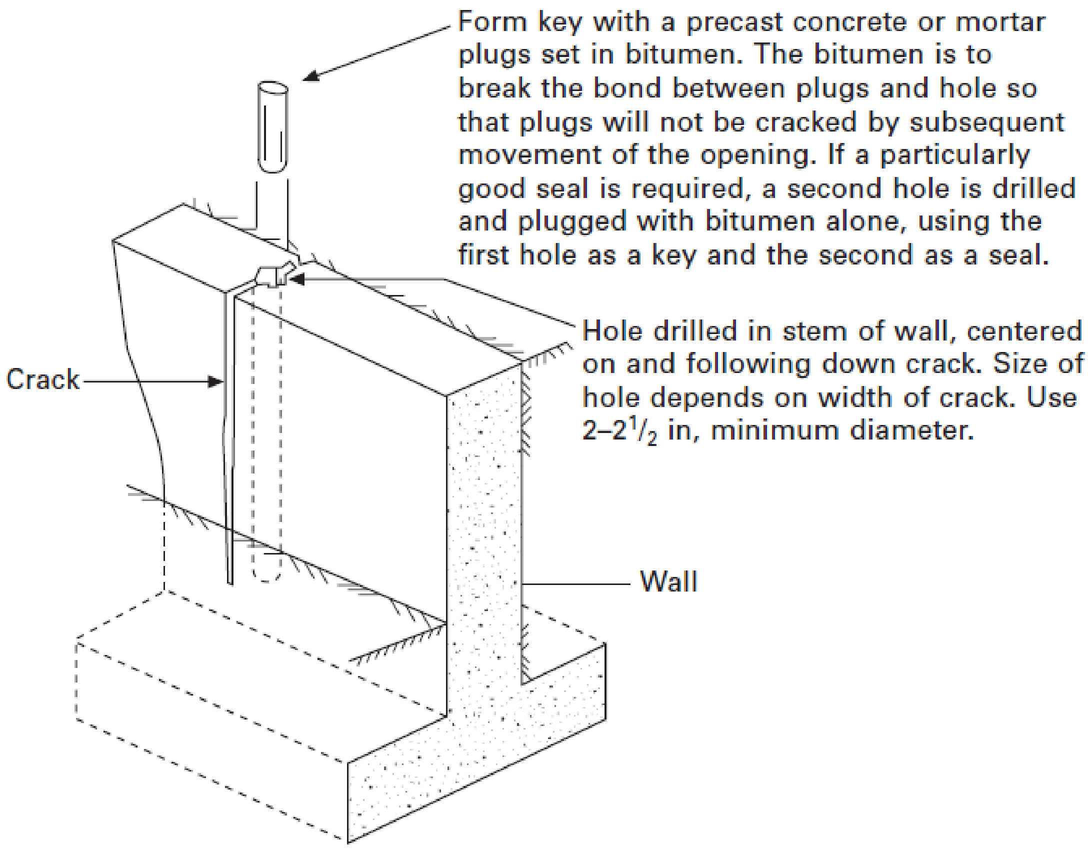

Figure 2.8: Repair by drilling and plugging (ACI Committee 224.1R).

on and following the crack. The hole must be large enough to intersect the crack along its full length and provide enough repair material to structurally take the loads exerted on the key. The drilled hole should then be cleaned, made tight, and filled with grout. The grout key prevents transverse movements of the sections of concrete adjacent to the crack. The key will also reduce heavy leakage through the crack and loss of soil from behind a leaking wall.

If water tightness is essential and structural load transfer is not, the drilled hole should be filled with a resilient material of low modulus in lieu of grout. If the keying effect is essential, the resilient material can be placed in a second hole, the first being grouted.

2.2.6.6 Gravity filling

Low-viscosity monomers and resins can be used to seal cracks with surface widths of 0.03-2mm by gravity filling (Rodler, Whitney, Fowler & Wheat, 1989). High-molecular-weight methacrylate's, urethanes, and some low-viscosity epoxies have been used successfully. The lower the viscosity of the monomers and resins, the finer the cracks that can be filled.

The typical procedure is to clean the surface by air blasting and/or water blasting. Wet surfaces should be permitted to dry several days to obtain the best crack filling. The monomer or resin can be poured onto the surface and spread with brooms, rollers, or squeegees. The material should be worked back and forth over the cracks to obtain maximum filling since the monomer or resin penetrates slowly into the cracks. Excess material should be removed from the surface to prevent slick, shining areas after curing. If surface friction is important, sand should be broadcast over the surface before the monomer or resin cures.

If the cracks contain significant amounts of silt, moisture, or other contaminants, the sealant cannot fill them. Water blasting followed by a drying time may be effective in cleaning and preparing these cracks. Cores taken at cracks can be used to evaluate the effectiveness of the crack filling. The depth of penetration of the sealant can be measured. Shear (or tension) tests can be performed with the load applied in a direction parallel to the repaired cracks (as long as reinforcing steel is not present in the core in or near the failure area). For some polymers, the failure crack will occur outside the repaired crack.

2.2.6.7 Grouting

❖ ***Portland cement grouting***

Wide cracks, particularly in gravity dams and thick concrete walls, may be repaired by filling with Portland cement grout. This method is effective in stopping water leaks, but it will not structurally bond cracked sections. The procedure consists of cleaning the concrete along the crack; installing built-up seats (grout nipples) at intervals astride the crack to provide a pressure-tight connection with the injection apparatus; sealing the crack between the seats with a cement paint, sealant, or grout; flushing the crack to clean it and test the seal; and then grouting the whole area.

Grout mixtures may contain cement and water or cement plus sand and water, depending on the width of the crack. However, the water-cement ratio should be kept as low as practical to maximize the strength and minimize shrinkage. Water reducers or other admixtures may be used to improve the properties of the grout. For small volumes, a manual injection gun may be used; for larger volumes, a pump should be used. After the crack is filled, the pressure should be maintained for several minutes to ensure good penetration.

❖ ***Chemical grouting***

Chemical grouts consist of solutions of two or more chemicals (such as urethanes, sodium silicates, and acrylamides) that combine to form a gel, a solid precipitate, or a foam, as opposed to cement grouts that consist of suspensions of solid particles in a fluid. Cracks in concrete as narrow as 0.05mm have been filled with chemical grout. The advantages of chemical grouts include applicability in moist environments (excess moisture available), wide limits of control of gel time, and ability to be applied in very fine fractures. Disadvantages are the high degree of skill needed for satisfactory use and lack of strength.

2.2.6.8 Dry-packing

Dry-packing is the hand placement of a low water content mortar followed by tamping or ramming of the mortar into place, producing intimate contact between the mortar and the existing concrete (US Bureau of Reclamation, 1975). Because of the low water-cement ratio of the material, there is little shrinkage, and the patch remains tight and can have good quality with respect to durability, strength, and water tightness.

Dry-pack can be used for filling narrow slots cut for the repair of dormant cracks. The use of dry-pack is not advisable for filling or repairing active cracks. Before a crack is repaired by dry-packing, the portion adjacent to the surface should be widened to a slot about 25mm wide and 25mm deep. The slot should be undercut so that the base width is slightly greater than the surface width. After the slot has been thoroughly cleaned and dried, a bond coat, consisting of cement slurry or equal quantities of cement and fine sand mixed with water to a fluid paste consistency, or an appropriate latex bonding compound (ASTM C 1059), should be applied.

Placing of the dry-pack mortar should begin immediately. The mortar consists of one-part cement, one to three parts sand passing a No. 16 (1.18mm) sieve, and just enough water so that the mortar will stick together when molded into a ball by hand. If the patch must match the color of the surrounding concrete, a blend of grey Portland cement and white Portland cement may be used. Normally, about one-third of white cement is adequate, but the precise proportions can be determined only by trial.

To minimize shrinkage in place, the mortar should stand for 1/2 hour after mixing and then should be remixed prior to use. The mortar should be placed in layers about 10mm thick. Each layer should be thoroughly compacted over the surface using a blunt stick or hammer, and each underlying layer should be scratched to facilitate bonding with the next layer. There must be no time delays between layers. The mortar may be finished by laying the flat side of a hardwood piece against it and striking it several times with a hammer. Surface appearance may be improved by a few light strokes with a rag or sponge float. The repair should be cured by using either water or a curing compound. The simplest method of moist curing is to support a strip of folded wet burlap along the length of the crack.

2.2.6.9 Crack arrest

During construction of massive concrete structures, cracks due to surface cooling or other causes may develop and propagate into new concrete as construction progresses. Such cracks may be arrested by blocking the crack and spreading the tensile stress over a larger area (US Army Corps of Engineers, 1995). A piece of bond-breaking membrane or a grid of steel mat may be placed over the crack as concreting continues. A semicircular pipe placed over the crack may also be used (Figure 2.9). A description of installation procedures for semicircular pipes used during the construction of a massive concrete structure follows: (i) the semicircular pipe is made by splitting 200mm, 16 gauge pipe and bending it to a semicircular section with about 75mm flange on each side; (ii) the area in the vicinity of the crack is cleaned; (iii) the pipe is placed in sections so as to remain centered on the crack; (iv) the sections are then welded together; (v) holes are cut in the top of the pipe to receive grout pipes; and (vi) after setting the grout pipes, the installation is covered with concrete placed concentrically over the pipe by hand. The installed grout pipes are used for grouting the crack at a later date, thereby restoring all or a portion of the structural continuity.

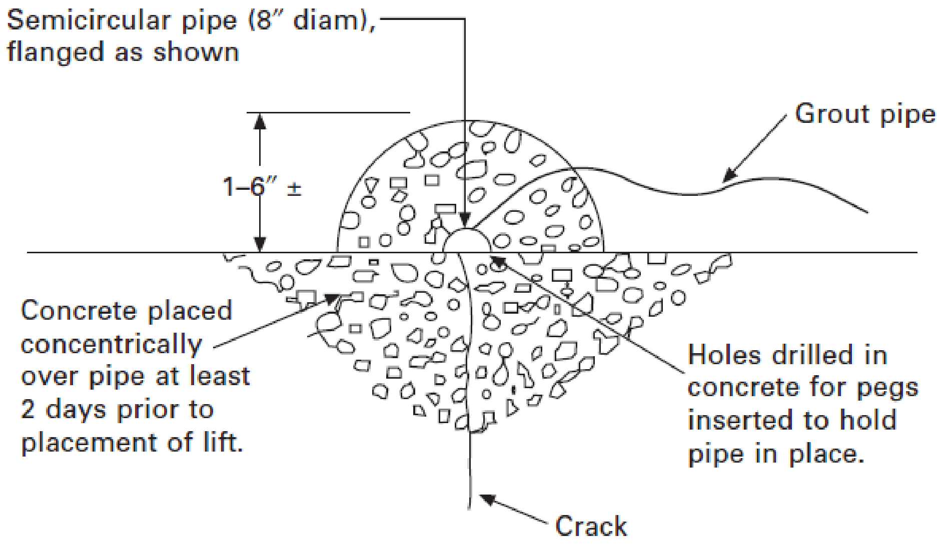

Figure 2.9: Crack arrest method (ACI Committee 224.1R).

2.2.6.10 Polymer impregnation

Monomer systems can be used for effective repair of some cracks. A monomer system is a liquid consisting of monomers which will polymerize into a solid. Suitable monomers have varying degrees of volatility, toxicity, and flammability. They do not mix with water. They are very low in viscosity and will soak into dry concrete, filling the cracks, much as water does. The most common monomer used for this purpose is methyl-methacrylate. Monomer systems used for impregnation contain a catalyst or initiator plus the basic monomer (or combination of monomers). They may also contain a cross-linking agent. When heated, the monomers join together or polymerize, creating a tough, strong, durable plastic that greatly enhances a number of concrete properties.

If a cracked concrete surface is dried, flooded with the monomer, and polymerized in place, some of the cracks will be filled and structurally repaired. However, if the cracks contain moisture, the monomer will not soak into the concrete at each crack face and, consequently, the repair will be unsatisfactory. If a volatile monomer evaporates before polymerization, it will be ineffective. Polymer impregnation has not been used successfully to repair fine cracks. Polymer impregnation has primarily been used to provide more durable, impermeable surfaces (Hallin, 1978; Webster et al., 1978). Badly fractured beams have been repaired using polymer impregnation. The procedure consists of drying the fracture, temporarily encasing it in a watertight (monomer proof) band of sheet metal, soaking the fractures with the monomer, and polymerizing the monomer. Large voids or broken areas in compression zones can be filled with fine and coarse aggregate before being flooded with the monomer, providing a polymer concrete repair.

2.2.6.11 Overlay and surface treatments

Fine surface cracks in structural slabs and pavements may be repaired using either a bonded overlay or surface treatment if there will not be further significant movement across the cracks. Un-bonded overlays may be used to cover, but not necessarily repair a slab. Overlays and surface treatments can be appropriate for cracks caused by one-time occurrences and which do not completely penetrate the slab. These techniques are not appropriate for repair of progressive cracking, such as that induced by reactive aggregates, and D-cracking.

Slabs-on-grade in freezing climates should not be repaired by an overlay or surface treatment that is a vapor barrier. An impervious barrier will cause condensation of moisture passing from the sub-grade, leading to critical saturation of the concrete and rapid disintegration during cycles of freezing and thawing.

❖ ***Surface treatments***

Low solids and low-viscosity resin-based systems have been used to seal the concrete surfaces, including treatment of very fine cracks. They are most suited for surfaces not subject to significant wear. Bridge decks and parking structure slabs, as well as other interior slabs, may be coated effectively after cracks are treated by injecting with epoxy or by routing and sealing. Materials such as urethanes, epoxies, polyesters, and acrylics have been applied in a thickness of 1–50mm, depending on the material and purpose of the treatment. Skid-resistant aggregates are often mixed into the material or broadcast onto the surface to improve traction.

❖ ***Overlays***

Slabs containing fine dormant cracks can be repaired by applying an overlay, such as a polymer, modified portland cement mortar or concrete, or by silica fume concrete. Slabs with working cracks can be overlaid if joints are placed in the overlay directly over the working cracks. In highway bridge applications, an overlay thickness as low as 30mm has been used successfully (NCHRP, 1970). Suitable polymers include styrene butadiene or acrylic latexes. The resin solids should be at least 15% by weight of the Portland cement, with 20% usually being optimum (Clear and Chollar, 1978).

2.2.6.12 Autogenous healing

A natural process of crack repair known as 'autogenous healing' can occur in concrete in the presence of moisture and the absence of tensile stress (Lauer, 1956). It has practical application for closing dormant cracks in a moist environment, such as may be found in mass concrete structures.

Healing occurs through the continued hydration of cement and the carbonation of calcium hydroxide in the cement paste by carbon dioxide, which is present in the surrounding air and water. Calcium carbonate and calcium hydroxide crystals precipitate, accumulate, and grow within the cracks. The crystals interlace and twine, producing a mechanical bonding effect, which is supplemented by a chemical bonding between adjacent crystals and between the crystals and the surfaces of the paste and the aggregate. As a result, some of the tensile strength of the concrete is restored across the cracked section, and the crack may become sealed.

Healing will not occur if the crack is active and is subjected to movement during the healing period. Healing will also not occur if there is a positive flow of water through the crack, which dissolves and washes away the lime deposits unless the flow of water is so slow that complete evaporation occurs at the exposed face causing re-deposition of the dissolved salts.

Saturation of the crack and the adjacent concrete with water during the healing process is essential for developing any substantial strength. Submergence of the cracked section is desirable. Alternatively, water may be ponded on the concrete surface so that the crack is saturated. The saturation must be continuous for the entire period of healing. A single cycle of drying and re-immersion will produce a drastic reduction in the amount of healing strength. Healing should be commenced as soon as possible after the crack appears. Delayed healing results in less restoration of strength than does the immediate correction.

2.2.7 Retrofitting and Strengthening Techniques

2.2.7.1 Section enlargement

Enlargement is the placement of additional concrete and reinforcing steel on an existing structural member. Beams, slabs, columns, and walls, if necessary, can be enlarged to add stiffness or load- carrying capacity in most cases, the enlargement is bonded to the existing concrete to create a monolithic member.

This method of strengthening involves placing additional “bonded” reinforced concrete to an existing structural member in the form of an overlay or a jacket. With section enlargement, columns, beams, slabs, and walls can be enlarged to increase their load-carrying capacity or stiffness. A typical enlargement is approximately 50 to 75mm for slabs and 75 to 125 mm for beams and columns (Keys to Success, 2018).

Concrete jacketing is typically applied methods of repair and strengthening of concrete members. Jacketing is one of the most generally used techniques to strengthen reinforced concrete (RC) columns. The size of the jacket, the number and diameter of the steel bars used in the jacketing process depend on the structural analysis that was made to the column., If it is required, beams, slabs, and walls can be enlarged to add stiffness or load - carrying capacity. Extensive longitudinal and transverse reinforcement is added in the new layer of concrete, enhancing the shear and flexural strength and ductility. It is necessary to provide a good bond between new and old concrete in reinforced concrete jacketing. Reinforced concrete jacketing can be applied one, two, three or four side of the column, depending on the state of the implementation and space around the column. In order to made jacketing to a required level (Aktas and Erdemli 2017);

- The strength of the new material must be greater than the existing one
- Min 10 cm thickness of the jacket for the cast-in-situ concrete
- New reinforcement and concrete must collaborate with existing concrete and reinforcement

This method application is difficult to construct in some active buildings such as hospitals, schools, industry etc. because of the implementation is taken time, made noise and many other limitations. In most cases, size of the columns or reinforced concrete members increased by concrete jacketing. Reinforced concrete jackets are built by enlarging the existing cross-section with a new layer of concrete and reinforcement (Aktas and Erdemli 2017).

2.2.7.2 External plate bonding

Steel plate is one of the most common materials for the strengthening of reinforced concrete structures. It is very much effective for increasing the flexural and shear capacity of the reinforced concrete beam. Strengthening with reinforcing steel plate is a popular system due to its availability, cheapness, uniform materials properties (isotropic), easy to work, high ductility and high fatigue strength. Investigations into the performance of members strengthened by this method were started in the 1960s. This method had been used to strengthen not only buildings but also bridges in many countries such as Belgium, France, Japan, Poland, South Africa, Switzerland and the United Kingdom. However, the most usual form of plating is to glue steel plates to the tension faces of beams. In this position, reinforcing plate is at its furthest extremity from the compression region and, as a result, the composite flexural action is at its maximum (Oehlers & Ali, 1997). Furthermore, the combined action between the plate, glue, and concrete will be maintained until failure/

2.2.7.3 External post-tensioning

The use of external pre-stressing as a means of strengthening or rehabilitating existing bridges has been used in many countries and has been found to provide an efficient and economical solution for a wide range of bridge types and conditions (Daly & Witarnawan, 1997). The technique is growing in popularity because of the speed of installation and the minimal disruption to traffic flow which can, in many cases, be the critical factor in decisions regarding strengthening. In spite of its obvious advantages, there is a lack of general information on how it can be applied and there are no specific guidelines available on this method of strengthening.

The principle of external post-tensioning is the same as that of pre-stressing, i.e., the application of an axial load combined with a hogging bending moment to increase the flexural capacity of a beam and improves the cracking performance. It can also have a beneficial effect on shear capacity. Precise evaluation of flexural and shear capacity of beams with unbonded tendons, either internal or external to the section, is difficult. This is because the load in the tendons is a function of the overall behavior of the beam, rather than just depending on the strain distribution at a particular critical section.

Taniguchi, Mutsuyoshi, Kita, & Machida, (1997) conducted fatigue tests on three T-beams internally pre-stressed with CFRP tendons and post-tensioned with external AFRP tendons. The internal and external tendons were initially pre-stressed at 22% and 40% of the tendon’s ultimate capacity, respectively. All of the beams survived two million cycles without failure, although a 12–15% decrease in the pre-stressing force was recorded during fatigue loading, attributed to either relaxation of the tendons or slippage of the anchors.

Grace and Abdel-Sayed (1998) used a combination of bonded internal CFRP tendons with un-bonded external double-draped carbon fiber cables in the construction of four bridge models having double-tee cross sections. The post-tensioning forces in the external tendons varied between 57% and 78% of their ultimate capacity. The four models were tested under fatigue loading at different load ranges within the working load limit (less than the cracking loads), and infinite fatigue lives were reported for all models. Insignificant losses in the pre-stressing forces were encountered in the externally draped tendons (approximately 3% of the initial force).

Braimah, Green & Campbell (2006) tested three beams post-tensioned with internally un-bonded CFRP tendons under fatigue loading. The CFRP tendons were post-tensioned to 60% of their ultimate capacity. Only one CFRP post-tensioned beam survived two million cycles of fatigue loading. Failure of the other post-tensioned beams was initiated by the fracture of the tendons at the tendon-anchor junction after surviving a few thousand cycles. The available literature reveals only

2.2.7.4 Ferrocement laminates

Ferrocement is a type of thin composite materials made of cement mortar reinforced with uniformly distributed layers of continuous, relatively small diameter, wire meshes. Ferrocement, being of the same cementitious material as reinforced concrete (RC), is ideally suited as an alternative strengthening component for the rehabilitation of RC. structures. The ferrocement laminate possesses higher tensile strength to weight ratio and a degree of toughness, ductility, durability and cracking resistance that is considerably greater than those found in other conventional cement-based materials. The use of ferrocement proper in repair was first introduced by Iorns (1987) in the early 1980s mainly as relining membranes for the repair of liquid retaining structures, such as pools, sewer lines, tunnels, etc. For flexural strengthening, the ferrocement laminates were cast onto the soffits (tension face) of the beams without any change in the width of the beams, while in shear strengthening the ferrocement laminates were formed onto the three exposed faces of the beams, except for the top compression face. Before placing the ferrocement laminates proper surface preparation should be ensured.

2.2.7.5 Sprayed concrete

Sprayed concrete is one of the oldest materials and the most common techniques of repairing and strengthening of reinforced concrete structures. Sprayed concrete has been used in the field of structural repair and strengthening for almost 90 years.

Diab (1998) described the technique of strengthening of the reinforced concrete beam by using sprayed concrete. There are two processes for applying sprayed concrete. American Concrete Institute defines dry mix sprayed concrete as sprayed concrete in which most of the mixing water is added at the nozzle, and wet mix sprayed concrete as sprayed concrete in which the ingredients, including water, are mixed before introduction into the delivery hose, is normally added at the nozzle. Both dry mix and wet mix sprayed concrete is used in concrete repair/strengthening work, but the use of dry mix sprayed concrete is more common.

2.2.7.6 Strengthening using fiber reinforced polymer (FRP)

Using fiber reinforced polymer (FRP) for the strengthening of reinforced concrete structures is also effective due to its high strength to weight ratio. Garden and Hollaway (1998) were described FRP materials had mechanical and physical properties superior to those of steel, particularly with respect to tensile and fatigue strengths, and these qualities are maintained under a wide range of temperatures. FRP composite materials were first introduced in the early 1940s. The U.S. Navy and Air-Force capitalized on the exceptional strength to weight ratio and inherent resistance to corrosion of these materials in a variety of applications (Harries, Porter, & Busel, 2003). In 1986, the world’s first highway bridge using FRP reinforcing tendons was built in Germany. The first FRP pedestrian bridge was erected in 1992 in Aberfeldy, Scotland. In the U.S., the first FRP concrete bridge deck was built in 1996 at Mc Kinleyville (Harries et al., 2003). A more sustainable market has developed around the use of FRP materials to strengthen and repair concrete structures. Several fiber reinforced polymer (FRP) systems are now commercially available for the external strengthening of concrete structures. Grace, Ragheb, & Abdel-Sayed (2004) mentioned fibers commonly used in these systems include glass, aramid, and carbon and they are available in many forms such as pultruded plates, uniaxial fabrics, woven fabrics and sheets.

2.2.8 Strengthening of Concrete Structures

2.2.8.1 Foundations

Columns foundations need strengthening in the case of applying additional loads. Widening and strengthening of existing foundations may be carried out by constructing a concrete jacket to the existing footings. The new jacket should be properly anchored to the existing footing and column neck in order to guarantee the proper transfer of loads. This can be accomplished by drilling holes into existing concrete of footing and epoxy grouting the longitudinal reinforcement of jacket. Another possibility is to provide full anchorage length for longitudinal reinforcement by extending the column jacket at the top of the footing (Olatayo & Oladeji, n.d).

When the bearing area of the footing is not sufficient, the size of the footing should be increased. If the column is also being jacketed, the transfer of load from column to footing becomes easy. The size of the "jacket" shall be selected such that the average maximum foundation pressure does not exceed the recommended allowable value. Attention shall be given during construction in order that the excavations for the new "jackets" do not affect the existing adjacent foundations.

There can be a split of new concrete from the old concrete surface under the action of loads. To avoid this splitting of concrete, a sufficient number of closed rings with sufficient overlap or welded connection should be provided around the footing.

2.2.8.2 Columns

Columns, along with load-bearing walls, are the main members for transferring loads vertically downwards from one end to another a structure. The resistance of a column can either be determined by buckling or by its crushing. These two cases must always be considered during design. When strengthening a column, its slenderness is therefore of interest. Some approaches might be more or less appropriate depending on how large the bending moment is in comparison with the normal force. One of the most straightforward ways to strengthen a slender column is to brace it and simply reduce its buckling length.

According to Saraswathi & Saranya (2016) strengthening of reinforced concrete columns is needed when,

- The load carried by the column is increased due to either increase the number of floors or due to mistakes in the design.
- The compressive strength of the concrete or the percent and type of reinforcement are not according to the codes' requirements.
- The inclination of the column is more than the allowable. The settlement in the foundation is more than the allowable.

There are several techniques for the strengthening of reinforced concrete columns like reinforced concrete jacketing, steel jacketing, and FRP confining or jacketing

2.2.8.3 Load bearing walls

In many existing RC buildings, shear walls constitute the seismic force resisting system. Shear walls that were designed according to older design codes may now be seismically deficient according to modern seismic design codes due to their insufficient strength and/or ductility (El-Sokkary & Galal 2013). Shear walls designed according to modern seismic design codes may experience higher demands at upper stories arising from the effects of higher modes of vibrations (Priestley and Amaris, 2002; Panneton, Léger, & Tremblay, 2006). The aforementioned situations necessitate upgrading the seismic performance of many existing RC shear walls to meet the requirements of modern seismic design codes. Different strengthening methods are available that's are strengthening against crushing and buckling of walls by section enlargement, strengthening against crushing and buckling of walls by external struts and strengthening against buckling of walls by vertical CFRP.

2.2.8.4 Beams

The need for strengthening a reinforced concrete beam or a number of beams in a structure is usually caused by problems due to degradation of characteristics of materials with time, reduction in cross-section, corrosion, wrong initial design or the increase in the load demand on the building when utilized for a new purpose other than it was intended to. These problems may lead to the existing steel bars in the beam to become unsafe or insufficient. In such cases, there are a number of solutions to be applied to make them safe or sufficient enough to bear the load (Strengthening of RC Beam, 2018)

2.2.8.5 Masonry walls

Brick masonry fails mainly due to shear failure, bending failure and sliding failure (Priyadarshani, Sanjeewa, De Silva, & Mendis, 2013). Strengthening of masonry walls is required to prevent failure and collapse during major earthquake or addition of extra load on buildings. Strengthening of masonry walls also may be required during the rehabilitation of buildings. Unreinforced masonry walls have good compressive strength, but they are brittle and very weak under the action of lateral loads which causes tension in walls. Whenever tension force act on a masonry wall, it tends to crack. Cracking of masonry walls may occur due to the settlement of foundation, during earthquakes, application of lateral loads. There can be several causes for masonry wall cracks, but occurrence these cracks may lead to complete collapse of the wall (How to Strengthen Masonry Walls, 2018). Masonry walls can be strengthened by providing reinforced concrete jackets on one or both faces of walls or Using FRP Structural Repointing for the strengthening of masonry walls (Tumialan, Huang, Nanni & Silva, 2001).

2.2.8.6 Slabs

When an existing slab (of timber, composite steel beam, concrete or hollow clay block construction) is reinstated it often needs to be strengthened to reduce its deformability, increase its load-bearing capacity, and bring it into compliance with new regulatory requirements or support increased live loads required by the new use of the building (Slab strengthening, n.d.). The reason for which reinforced concrete slabs require the intervention for repairs or strengthening are the following (Taly, GangaRao & Vijay, 2006);

- Repairing damaged/deteriorated concrete slabs to restore their strength and stiffness.
- Corrosion of the reinforcement.
- Limiting crack width under increased (design/service) loads or sustained loads.
- Retrofitting concrete members to enhance the flexural strength and strain to failure of concrete elements requested by increased loading conditions such as earthquakes or traffic loads.
- Rectifying design and construction errors such as undersized reinforcement.
- Enhancing the service life of the RC slabs.
- Shear strengthening around columns for increasing the perimeter of the critical section for punching shear.
- Changes in the structural system such as cut-outs in the existing RC slabs.
- Changes in the design parameters.

- Optimization of structure regarding the reduction of deformations and of the stresses in the reinforcing bars.

2.3 Strengthening of Reinforced Concrete Beams

2.3.1 Strengthening with External Steel Plate

The technique of strengthening of concrete structures by bonding thin steel plate with the concrete surface with epoxy adhesive was first used in mid of 1960 in South Africa and France (Roberts & Haji-Kazemi, 1989). Systematic research of the various factors which cause the structural behavior of strengthened concrete beams, such as the thickness of the plate and the thickness of the epoxy adhesive layer, has been conducted by Jones, Swamy, Bloxham & Bouderbalah (1980) and Swamny & Jones (1983). Those researches showed that the structural performance of reinforced concrete beams increased significantly by using externally bonded steel plate to the tension faces.

An experimental investigation was conducted by Jones, Swamy & Ang (1982), on the strength and deformation characteristics in flexure of under and over reinforced concrete beams with glued steel plates. They investigated the composite behavior of the beams, the interaction of the plate, glue and concrete, and the influence of the glued plate on stiffness, cracking, plate slip, interface shear stress and ultimate strength of the beams. Significant increase of capacity 42% to 105% for the under-reinforced beams and 22% to 44% for the over-reinforced beams. The failure either by yielding or plate separation, act as composite action between the reinforced concrete beams and the glued steel plate. Thicker plates generally had a greater effect, but the mode of failure changed from yielding to separation as the thickness increased. To prevent the separation problem, an experimental investigation was carried out by Jones, Swamy, & Charif (1988) in which the plate was anchorage by bolts or L-shaped anchoring plates extending to the side faces of the beams. By this technique, the flexural capacity was improved but the separation of steel plates cannot be prevented.

2.3.2 Strengthening with Near Surface Mounted (NSM) Bars

In the year of 1940 NSM technique was first used in Finland where steel bars were placed into grooves for strengthening a bridge deck slab (Asplund, 1949). The first experimental study on NSM technique carried out by Blaschko & Zilch (1999) using CFRP strips.

Numerous experimental researches has investigated the flexural characteristics of Reinforced Concrete beams strengthened using NSM bars or FRP strips.

An experimental and numerical investigation has carried out by Almusallam, Elsanadedy, Al-Salloum, & Alsayed (2013) on RC beams strengthened in flexure by using NSM steel bars and GFRP bars. The steel and GFRP bars have shown excellent bond behavior in all case of the beam tested. The flexural capacity was increased with the increase of NSM reinforcement ratio. The flexural capacity of RC beams was increased for GFRP bars and steel bars strengthening up to 26.2% and 94.9% respectively for experimental study which was 46.2% and 115.9% for FEM.

An investigation carried out by Hosen, Jumaat, Darain, Obaydullah & Islam (2014) on Flexural Strengthening of RC Beams with near surface mounted (NSM) Steel Bars. The NSM steel reinforcement enhanced the load-deflection response of the RC beams. The deflections of the strengthened beams were meaningfully less than that of the control beam at any load level. The first cracking load was increased up to 39.68% and the ultimate capacity was increased up to 53.85% for the beams strengthened by NSM technique respectively control beam.

Jumaat, M. Z., et al. (2016) investigated the strengthening of RC beams using externally bonded reinforcement combined with near-surface mounted technique. This study investigates the flexural behavior of reinforced concrete (RC) beams strengthened through the combined externally bonded and near-surface mounted (CEBNSM) technique. The load was increased (118-230%), (38-120%) and (82-170%) for the first crack, yield and ultimate state respectively with the control beam. A considerable reduction of the deflection and increased stiffness of all strengthened beams were observed in this investigation. The average crack spacing of strengthened beams was less than the control beam.

2.3.3 Strengthening with External Steel Bars & Steel Angles

Gul, Alam, Khan, Badrashi & Shahzada (2015) investigated the flexural strengthening of reinforced concrete beams using external steel. External steel bars & steel angles were used by removing bottom concrete cover and welding with the stirrups for the strengthening of different reinforced concrete beams. The flexural capacity of reinforced concrete beams strengthened by external steel members was greatly enhanced and showed a uniform distribution of flexure cracks. The failure of the strengthened beam resulted in a very favorable mode as compared to the control specimen. The flexural strength was increased up to 80% and 110% for the use of steel bars and steel angles respectively. In most cases, the ductility was decrease and better ductility was shown for the strengthening with steel angles than the strengthening with external steel bars. The experimental result slightly varied from the computed results.

2.3.4 Strengthening with HPFRCC

Martinola, G., et al. (2007) investigated an application of high-performance fiber reinforced cementitious composites (HPFRCC) for RC beam strengthening. Concrete beams without reinforcement, reinforced concrete beams, strengthened concrete beams and strengthened reinforced concrete beams were used in this investigation. It can be noticed as the HPFRCC use allows increasing the bearing capacity of the beam (2.15 times), even if the post-peak behavior becomes softening. In any case, at the end of the softening branch, the load stabilizes with a plastic branch, with a value higher than that obtained in the RC beam without a jacket.

2.3.5 Strengthening with CFRP Laminates Configurations

Sobuz, Ahmed, Uddin & Sadiqul (2011) investigated the structural strengthening of RC beams externally bonded with different CFRP laminates configurations. The CFRP layer was attached to the bottom layer of beams with epoxy Adhesive. The result of the experimental study indicated that externally bonded CFRP laminates could be used effectively to strengthen the reinforced concrete beams. With the increase of CFRP layers, an increase in stiffness and flexural strength was achieved. The strengthening beams didn’t show any inter-layer delamination in any cases. From the experimental investigation, it was identified that the percentage increase of cracking load of 1, 2 and 3-layers CFRP strengthened beams were 25%, 50% and 75% respectively whereas the percentage increase of ultimate load was 54%, 73% and 85% respectively as compared to the control beam.

CHAPTER III

METHODOLOGY

3.1 General

The methodology is an important part of any successful work. The details of theoretical work procedure of this study are presented in this chapter including materials, specimen preparation, the design of controlled beams, the design of strengthened beams, and determination of the capacity of control and strengthened beams.

3.2 Materials

Materials are the most important element for any types of project. Good quality materials are always required for a successful experimental investigation. Different materials which were used in this thesis work are listed below.

- Coarse Aggregate (Stone chips)
- Fine Aggregate (Sylhet Sand)
- Binder (Ordinary Portland Cement)
- Steel Bar (6 mm, 10 mm & 12mm)
- Steel Plate (125mm × 3mm)
- Steel Angle (25mm × 25mm × 4mm)
- Adhesives (Epoxy-1 & Epoxy-2)

3.3 Specimen Preparation

Before preparation of specimens, a beam (prototype) was selected with a clear span of 4800mm and x-section of that beams was (300mm × 500mm). Twelve half scaled (Harris & Sabnis, 1999) reinforced concrete beams having rectangular x-section were constructed. The width, depth and length were 150mm, 250mm and 2700mm respectively. The compressive strength of concrete at 28 days was 33.3MPa. 500w deform bars were used for main reinforcement. Table 3.1 shows the design summary of all specimens. All beams were divided into six groups and each group consists of two beams. The first group (Group B1) was used as controlled specimens to find out the ultimate capacity of beams. Five other groups were used for Strengthening with different techniques after the application of preloading. Among which the 2nd and 3rd groups (Group B2 and B3) were strengthened by attaching steel plate on the bottom face of the beams. Two different types of epoxy adhesives were used as a bonding agent. Two ø12 mm steel bolts were used on each side of beams for the 3rd group. The 4th group (Group B4) was strengthened with Near Surface Mounted (NSM) steel bars. Two ø12 mm bars were attached in grooves with epoxy adhesive. Another two groups (Group B5 and B6) were strengthened with external steel angles and steel bars. The bottom concrete cover was removed and external steel angles and steel bars were attached by welding with bottom stirrups. Two steel angles (25mm × 25mm × 4mm) and 3-ø12 mm steel bars were used for the 5th and 6th group. After that, the bottom of the beams was cast again with new concrete.

Table 3.1: Design summary of all beams

Specimen ID	Types of Beams	Cross-section b × h (mm × mm)	Main Reinforcement	External Reinforcement	Bonding Mode / Adhesive	Fastening Mechanism
B1-1	Control Beams	150 × 250	2- ø10 mm	-	-	-
B1-2		150 × 250	2- ø10 mm	-	-	-
B2-1	Strengthened with Steel Plate	150 × 250	2- ø10 mm	125mm × 3mm steel plate	Epoxy-1	-
B2-2		150 × 250	2- ø10 mm		Epoxy-2	-
B3-1	Strengthened with Steel Plate & Bolts	150 × 250	2- ø10 mm	125mm × 3mm steel plate	Epoxy-1	Steel bolt
B3-2		150 × 250	2- ø10 mm		Epoxy-2	
B4-1	Strengthened with NSM Steel bars	150 × 250	2- ø10 mm	2-ø12 mm Rebar	Epoxy-1	-
B4-2		150 × 250	2- ø10 mm		Epoxy-2	-
B5-1	Strengthened with Steel Angles	150 × 250	2- ø10 mm	2- 25mm × 25mm × 4mm steel angle	-	Welding with stirrups
B5-2		150 × 250	2- ø10 mm		-	
B6-1	Strengthened with Steel bars	150 × 250	2- ø10 mm	3- ø12 mm Rebar	-	Welding with stirrups
B6-2		150 × 250	2- ø10 mm		-	

The reinforcement of the prototype would be forth times of the control beams. The moment capacity of the prototype would be eight times and the load-bearing capacity would be four times of the control beams in similar load arrangement. The reinforcement detailing of the prototype is shown in figure 3.1.

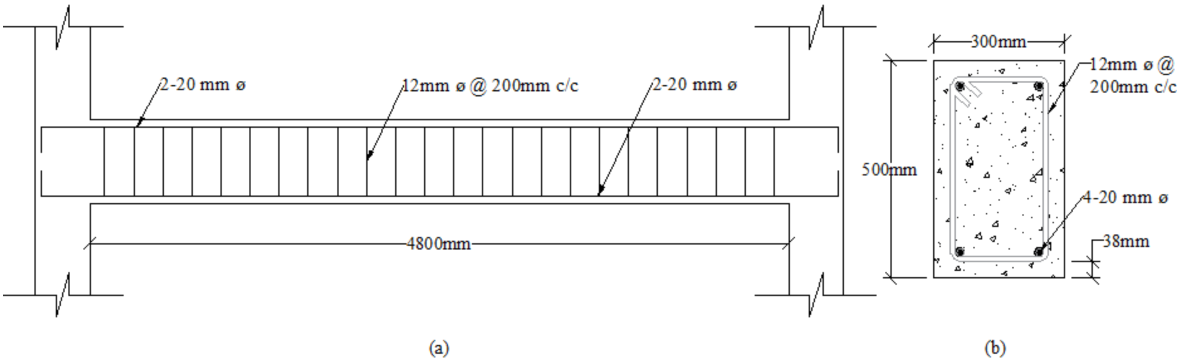

Figure 3.1: Reinforcement detailing of prototype (a) long section (b) x-section

3.4 Background of Preloading

Most of the cases rehabilitation and strengthening of structural elements are required after the application of service load. To investigate the effect of service load in the purpose of rehabilitation and strengthening preloading was applied to the beams before strengthening. Structural elements are normally designed for factored load suggested by BNBC and ACI (Dead Load factor 1.2, Live load factor 1.6). The allowable capacity is 83% & 62% of the ultimate load of the dead & live load according to BNBC and ACI. From these values, 65% and 75% of the ultimate load were applied as preloading before the strengthening of all beams.

3.5 Design of Control Beams

The moment capacity of the control specimens $M_{n(c)}$ was determined by using basic concepts for the rectangular reinforced concrete beams as given in the ACI code ACI-318-08 section 10.5. For the comparison of results, the beams were designed with minimum steel ratio.

$$A_{s(min)} = 3\frac{\sqrt{f_c'}}{f_y}bd \geq \frac{200}{f_y}bd \text{ - } (3.1)$$

As a requirement of minimum steel two ø10 mm bars were used as flexural reinforcement. To hold the stirrups, two ø10 mm bars were placed on top of the beam's web. The beams were designed strong enough against shear.

The maximum shear force was found on the basis of $A_{s(max)}$ calculated after the design of strengthened beams.

$$A_{s(max)} = \rho_{max} b d_{ave} \tag{3.2}$$

$$M_{n(c)} = A_s f_y \left(d - \frac{a}{2}\right) \tag{3.3}$$

$$a = \frac{A_s f_y}{0.85 f_c'} \tag{3.4}$$

$$M_{n(max)} = A_{s(max)} f_y \left(d_{ave} - \frac{a}{2}\right) \tag{3.5}$$

$$V = \frac{3M_{n(c)}}{L} \tag{3.6}$$

Where

$V =$ *Maximum shear force*

$M_{n(c)} =$ *Moment capacity of control beams*

$M_{n(max)} =$ *Maximum moment capacity of strengthened beams*

For the above maximum shear force, beams were designed and provided with ø6 mm 500w steel bars placed at 100mm on center throughout the length of the beams. The reinforcement detailing control beams is shown in Figure 3.2 (a) & (b). The strain diagram, nonlinear stress diagram and rectangular stress block of control beams are shown in Figure 3.2 (c), (d) & (e).

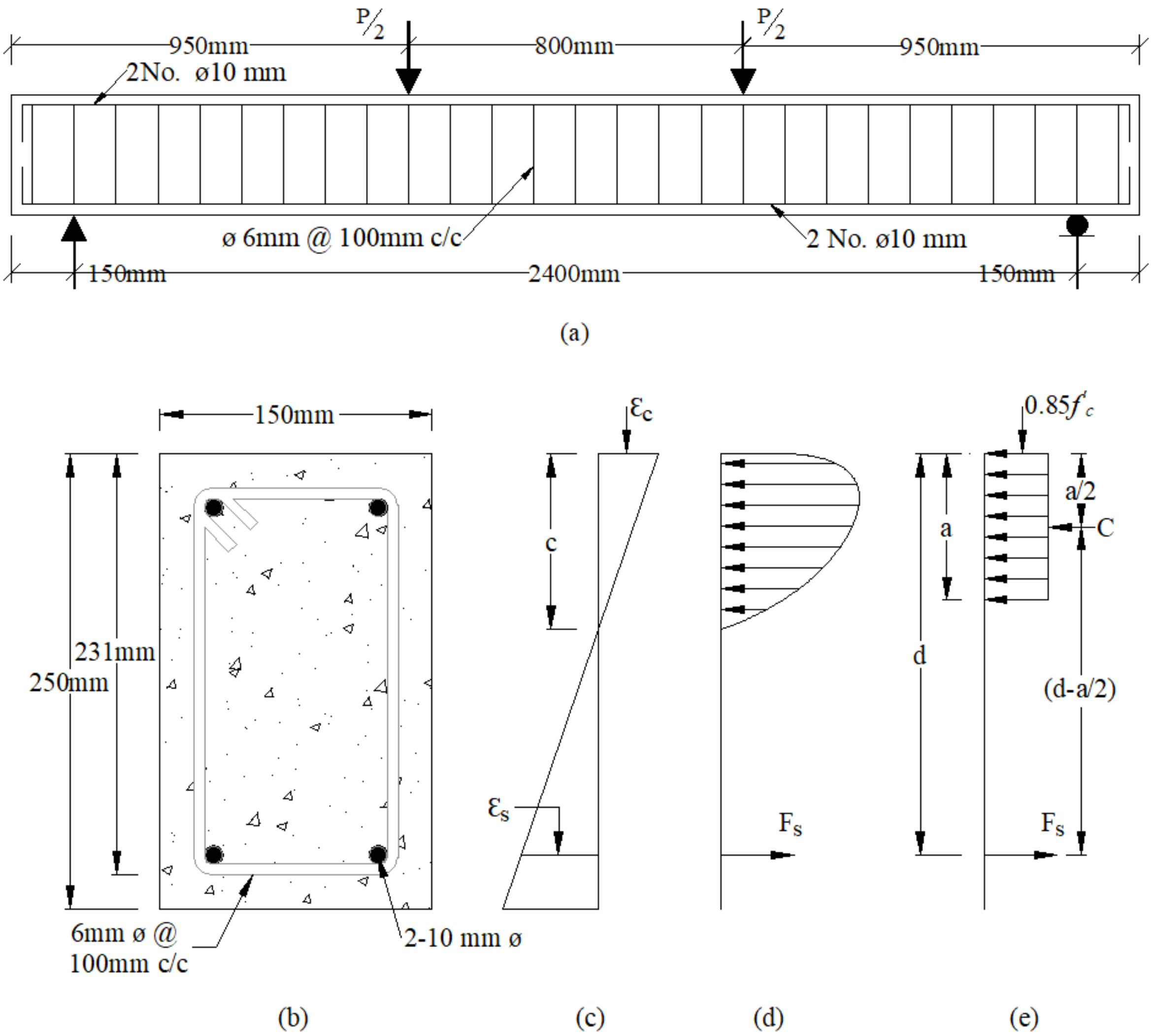

Figure 3.2: Schematic diagram of control beams (a) long section (b) x-section (c) strain diagram (d) nonlinear stress diagram (e) rectangular stress block

3.6 Design of Strengthened Beams

Two of the twelve beams were used as control specimens and the other ten beams were strengthened with different methods that are,

- Strengthening with external steel plate
- Strengthening with external steel plate anchorage by steel bolts
- Strengthening with Near Surface Mounted (NSM) steel bars
- Strengthening with external steel angles
- Strengthening with external steel bars

3.6.1 Strengthening with External Steel Plate

The same flexural design procedure was used for external strengthening of reinforced concrete beams as the regular rectangular beams. To find the steel area for the external reinforcement the strain compatibility and stress diagram were used which recommend by Gomes & Appleton (1999) are shown in Figure 3.3.

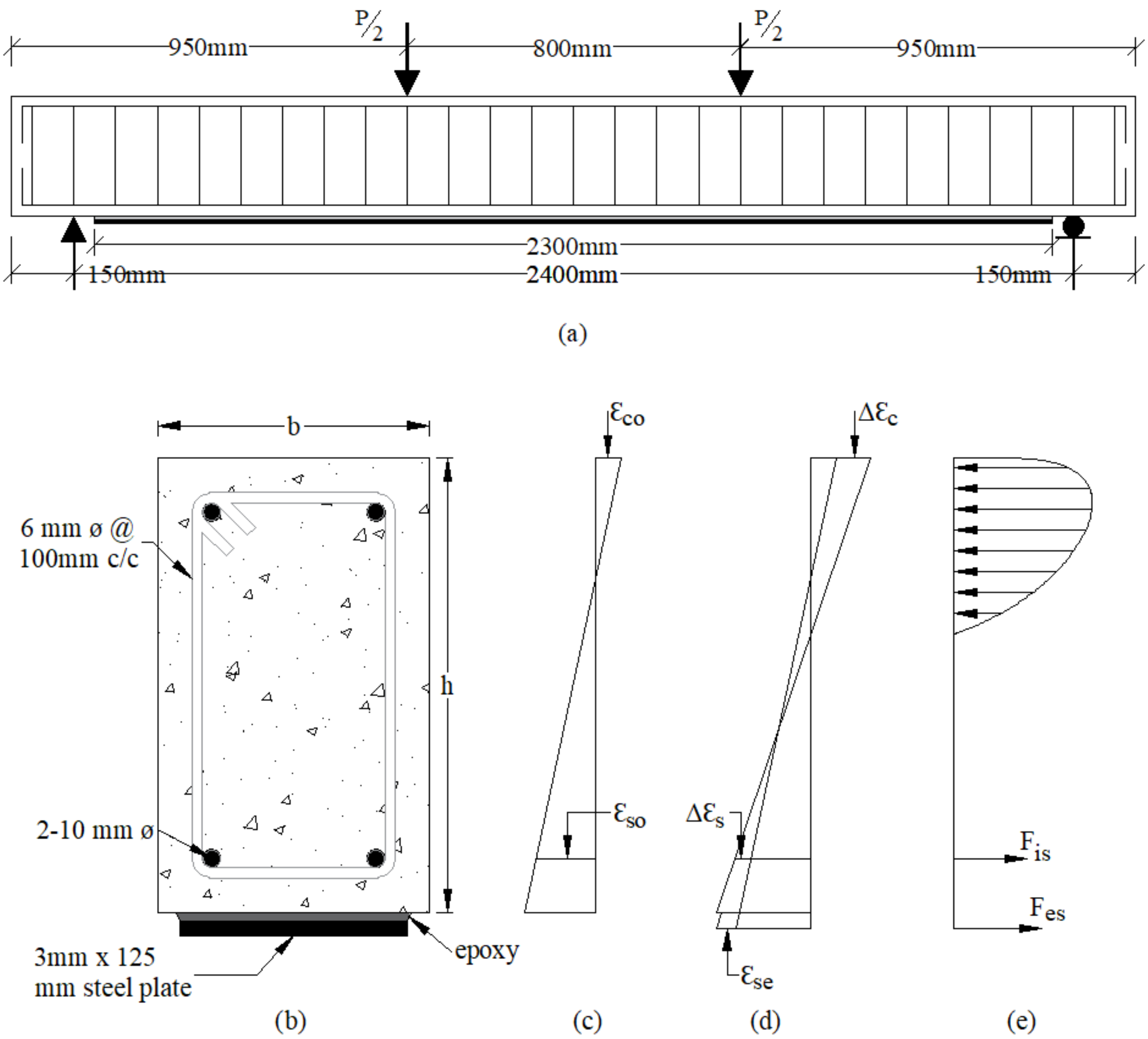

Figure 3.3: Schematic diagram of beams strengthened with steel plate (a) long section (b) x-section (c) strain diagram (d) nonlinear stress diagram

This procedure was also same as the ACI method for RC flexural members. The regular formulas for the design of RC beams were used to calculate the steel area for external reinforced concrete beams. The external steel area worked as the second flexural force acting member at the center of the external steel. The equivalent strain, nonlinear stress and rectangular stress diagram are shown in Figure 3.4.

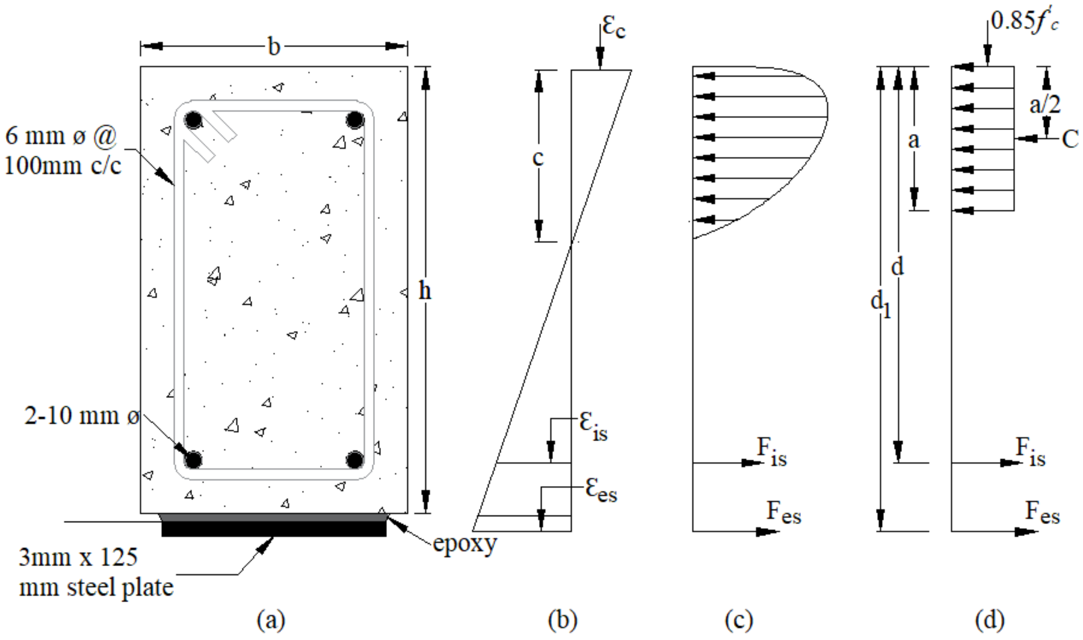

Figure 3.4: Equivalent stress-strain diagram of beams strengthened with steel plate (a) x-section (b) strain diagram (c) nonlinear stress diagram (d) rectangular stress block

The design procedure for externally strengthening of RC beams was based on maximum reinforcement ratio as given by ACI 318-08 Section 10.2 $\rho_{max} = \rho_{balance}$. The calculate steel area was divided in two parts. Internal steel area $A_{s(c)}$ and external steel area $A_{s(ext)}$.

$$A_{s(ext)} = A_{s(max)} - A_{s(c)} \quad \text{--} \quad (3.7)$$

Where:

$A_{s(c)} = \textit{steel area of control beams}$

$A_{s(ext)} = \textit{external steel area provided for strengthening purpose}$

The moment capacity of strengthened beams was calculated as

$$M_{n(total)} = A_{s(c)} f_{y(c)} \left(d - \frac{a}{2}\right) + A_{s(ext)} f_{y(ext)} \left(d_1 - \frac{a}{2}\right) \quad \text{----------------------} \quad (3.8)$$

Where:

$M_{n(total)} = \textit{moment capacity of strengthened beams}$

$f_{y(c)} = \textit{ yield strength of steel used in control beams}$

$f_{y(ext)} = \textit{ yield strength of external steel}$

125mm × 3mm steel plate was used at a distance of 200mm from the end of the beams and 50mm from the support. Two different types of adhesive were used for this purpose.

3.6.2 Strengthening with External Steel Plate Anchorage by Steel Bolts

The design of strengthened beams with external steel plate anchorage by steel bolts was similar to the beams strengthened with external steel plate bonded with epoxy adhesives (Figure 3.5). 2-ø12mm steel bolts were used for anchorage at a distance of 400mm and 600mm from the end of the beams. Figure 3.5 (a) & (b) show the position of the external steel plate and bolts. The strain diagram, nonlinear stress diagram and rectangular stress block are shown in Figure 3.5 (c), (d) & (e).

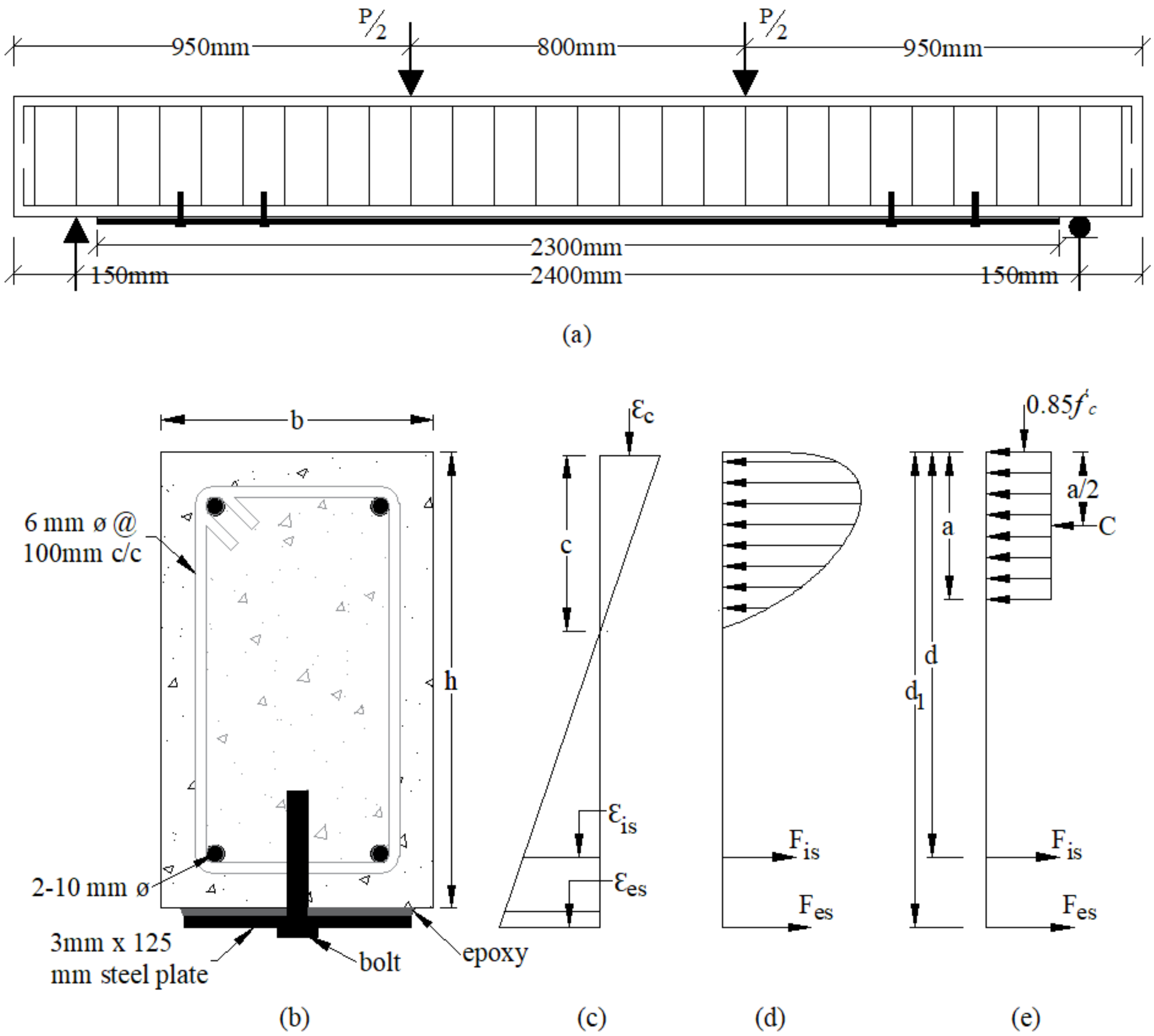

Figure 3.5: Schematic diagram of beams strengthened with steel plate anchorage by bolts (a) long section (b) x-section (c) strain diagram (d) nonlinear stress diagram (e) rectangular stress block

3.6.3 Strengthening with Near Surface Mounted (NSM) Steel Bars

Two grooves (19mm × 19mm) were drilled in the soffit of beams for the attachment of 2-ø12 mm steel bars. The steel bars were placed at a distance of 200mm from the end of the beams. Figure 3.6 (a) & (b) show the position of NSM steel bars. Two different types of adhesive were used for this purpose. The internal balance of load for a flexural member bearing internal reinforcing bars and external steel bars for the beams strengthened with NSM bars attached by epoxy adhesive are shown in Figure 3.6 (c), (d) & (e) according to ACI, A. 440.2.

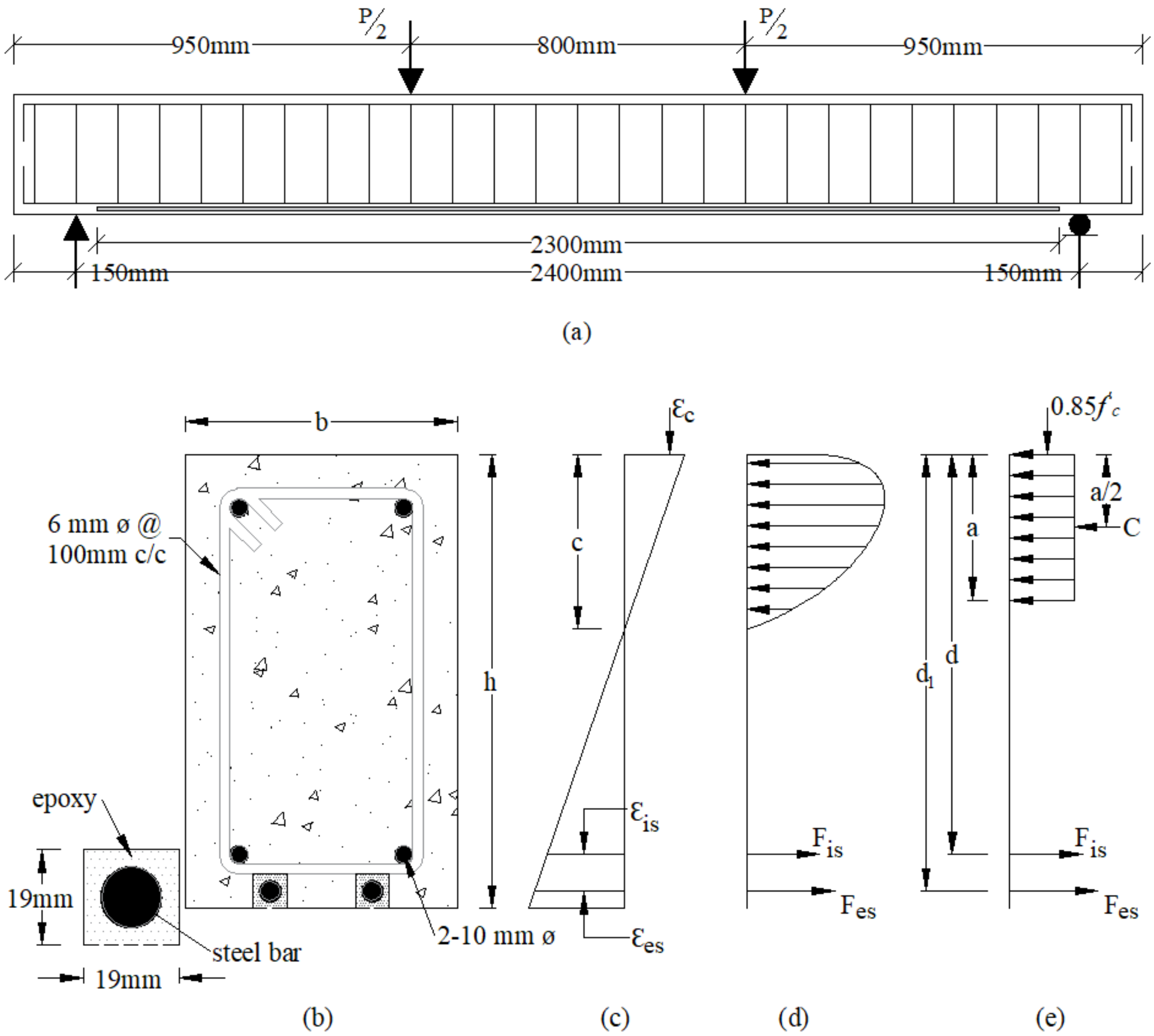

Figure 3.6: Schematic diagram of beams strengthened with NSM bars (a) long section (b) x-section (c) strain diagram (d) nonlinear stress diagram (e) rectangular stress block

3.6.4 Strengthening with External Steel Angles

The bottom and side (50mm) concrete cover was removed for the attachment of steel angles. Two steel angles (25mm × 25mm × 4mm) were attached with the bottom stirrups by welding. The steel angles were placed at a distance of 200mm from the side of the beams. The placement of external steel angles is shown in Figure 3.7 (a) & (b). The concrete cover was cast again with new concrete. To find the external steel area the strain compatibility and rectangular stress block was used by Gul et al. (2015) in case of Strengthening with external steel angles. The strain diagram, nonlinear stress diagram and rectangular stress block for the beams strengthened with external steel angles are shown in Figure 3.7 (c), (d) & (e).

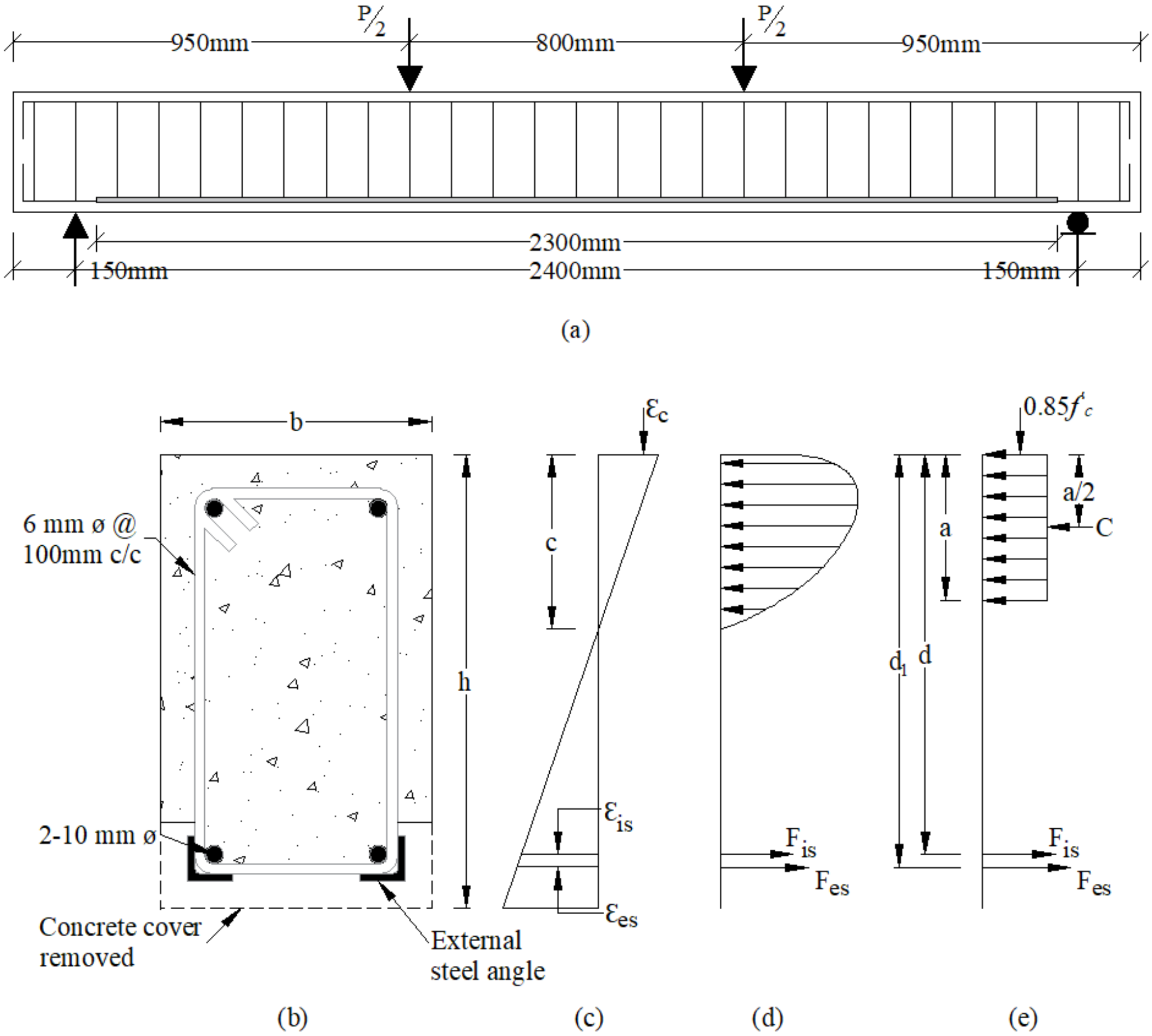

Figure 3.7: Schematic diagram of beams strengthened with steel angles (a) long section (b) x-section (c) strain diagram (d) nonlinear stress diagram (e) rectangular stress block

3.6.5 Strengthening with External Steel Bars

The bottom concrete cover was removed to attach external steel bars. Three (ø12 mm) steel bars were attached by welding with the bottom stirrups. Figure 3.8 (a) & (b) show the reinforcement detailing of the strengthened beams. The steel bars were placed at a distance of 200mm from the end of the beams. To find the external steel area the strain compatibility and rectangular stress block was used by Gul et al. (2015) in case of Strengthening with external steel bars. The internal balance of load for a flexural member bearing internal reinforcing bars and external steel bars can be represented as shown in Figure 3.8 (c), (d) & (e).

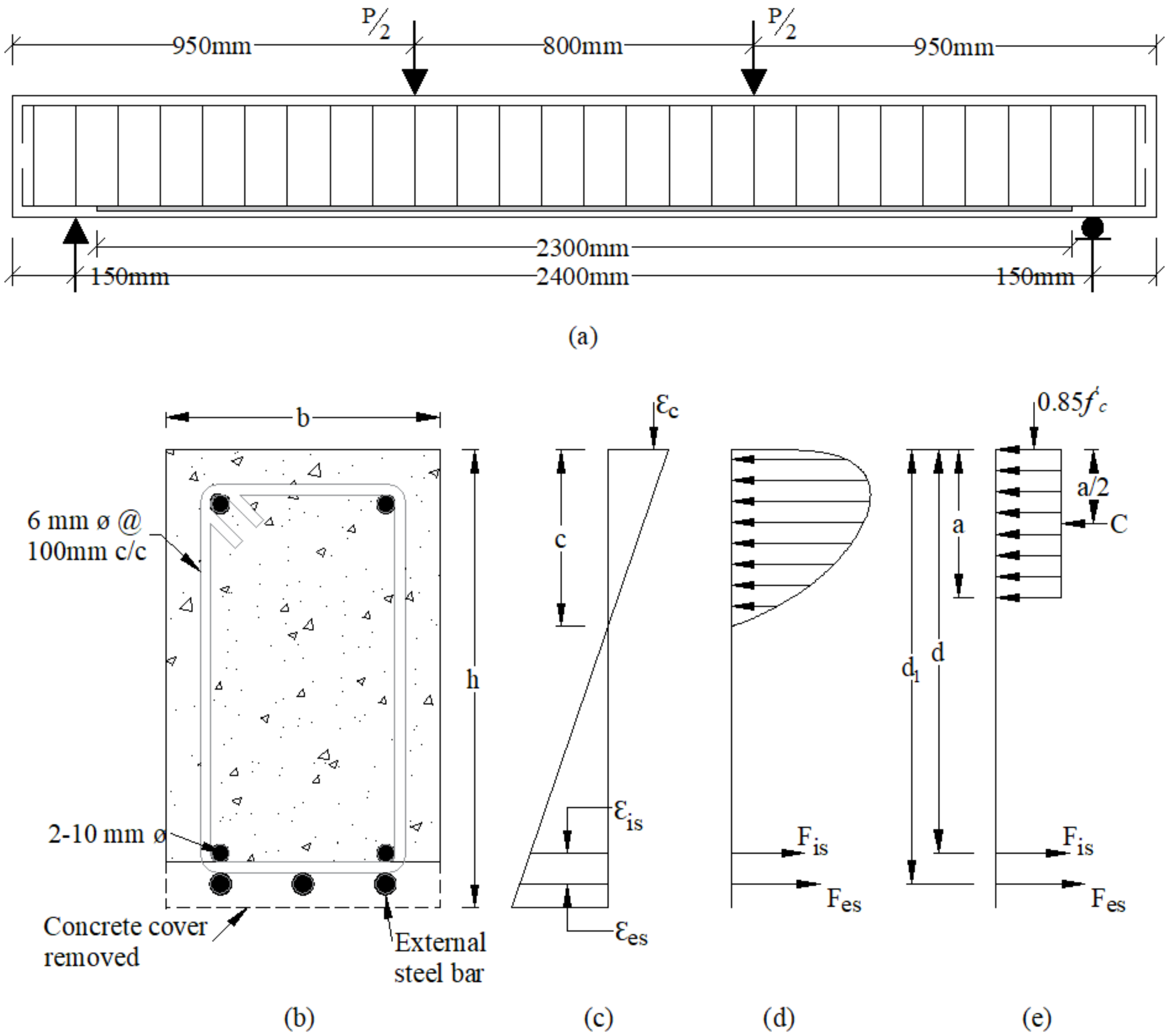

Figure 3.8: Schematic diagram of beams strengthened with steel bars (a) long section (b) x-section (c) strain diagram (d) nonlinear stress diagram (e) rectangular stress block

3.7 Determination of Capacity of Beams

The ultimate capacity of different beams (control beams and Strengthened beams) was calculated according to ACI 318-08. A beam can fail in two basic modes. Failure in compression and failure in tension. The internal and external reinforcement detailing was described in article 3.5 and 3.6 in this report. From the design section, the ultimate capacity of beams in compression was found by equation 3.9 and capacity in tension was found by equation 3.3 or equation 3.8.

$$M_{u(c)} = \rho_{max} f_y b d^2 \left(1 - 0.59 \rho_{max} \frac{f_y}{f_c'}\right) \quad (3.9)$$

$$\rho_{max} = 0.85 \beta_1 \frac{f_c'}{f_y} \frac{\varepsilon_u}{\varepsilon_u + 0.004} \quad (3.10)$$

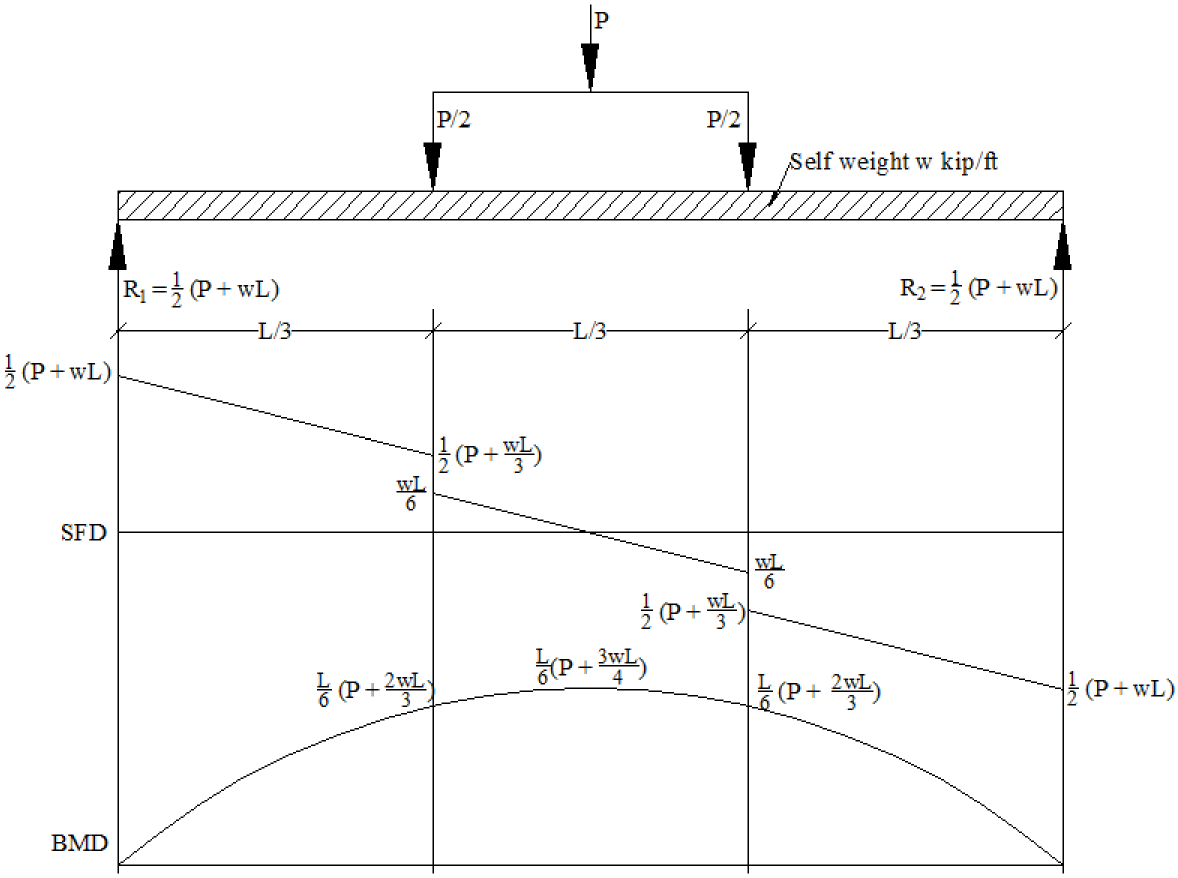

Figure 3.9: Applied load, shear force and bending moment diagram

As the beams were tension controlled the value of $M_{n(total)}$ was smaller for all beams. The applied load, shear and moment diagrams are shown in Figure 3.9. The maximum shear force was $\frac{1}{2}(P + wL)$ and the maximum bending moment was $\frac{L}{6}\left(P + \frac{3}{4}wL\right)$.

$$M_n = \frac{L}{6}\left(P + \frac{3}{4}wL\right) \text{-- } (3.11)$$

From equation 3.11 the maximum load bearing capacity of beams can be written by

$$P = \frac{6}{L}\left(M_n - \frac{1}{8}wL^2\right) \text{ --- } (3.12)$$

Where:

$P = \textit{ultimate load for beams}$

$L = \textit{span length}$

$w = \textit{self-weight of beam per unit length}$

The maximum moment capacity M_n was found from the minimum value of equation 3.8, equation 3.3 or equation 3.5 and using this value the maximum load was calculated from equation 3.12. The ultimate moment capacity of control beams was calculated 13.45 (kip-ft) in tension and 40.97 (kip-ft) in compression. The load bearing capacity of control beams was 43.2kN for the load arrangement which is shown in figure 3.9.

CHAPTER IV

EXPERIMENTAL PROGRAM

4.1 General

The experimental programs include the collection of materials, the test of materials, sample preparation, concrete mix design, casting and curing of reinforced concrete beams, the test of control specimens, preloading of beams before strengthening, strengthening of beams and finally testing of strengthened beams. The details of experimental work procedure of this study are described in this chapter.

4.2 Materials Properties

Properties of different materials which were required for the design and analysis were tested by the proper guideline of ASTM code.

4.2.1 Coarse Aggregate (Stone chips)

12.5 mm downgrade black stone chips were used as coarse aggregate. Different properties such as specific gravity and absorption capacity were determined by ASTM C 127, dry rodded unit weight was determined by ASTM C 29 and moisture content by ASTM C 556. Different properties of coarse aggregate are shown in Table 4.1. The grain size distribution curve is shown in Figure 4.1 according to ASTM C 136.

Table 4.1: Properties of aggregate

Types of Aggregate	Coarse Aggregate	Fine Aggregate
Specific Gravity (SSD)	2.85	2.56
Absorption Capacity (%)	3.09	3.39
Dry Rodded Unit Weight (Kg/m^3)	1510	1570
Moisture Content (%)	1.90	4.01
Fineness Modulus	-	2.70

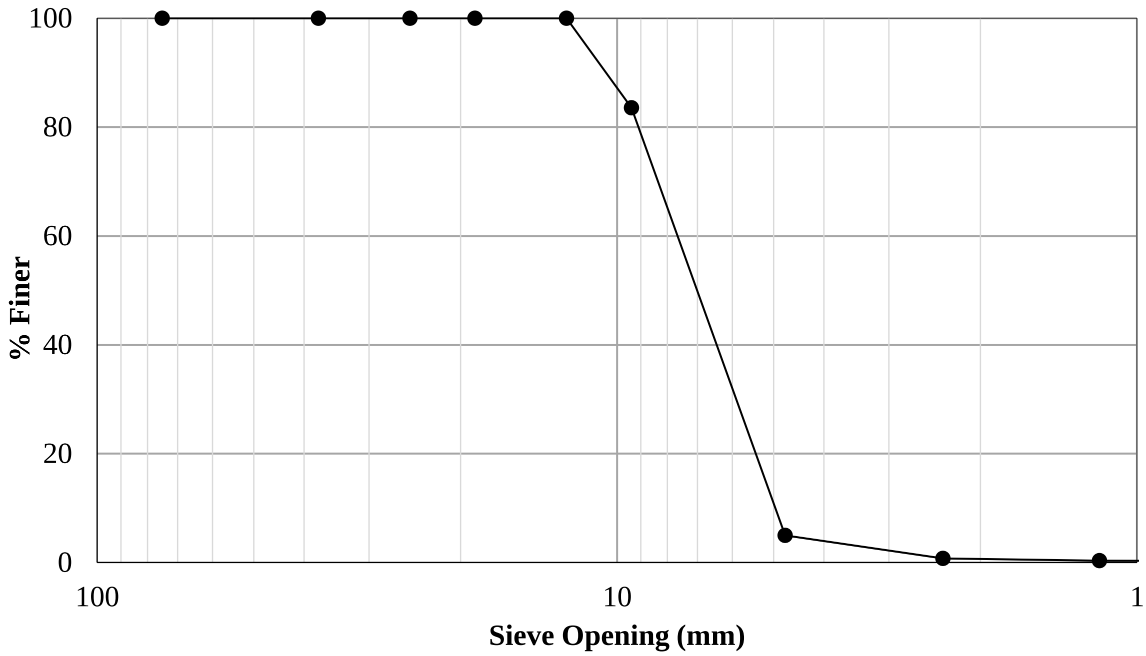

Figure 4.1: Grain size distribution curve of coarse aggregate

4.2.2 Fine Aggregate (Sylhet Sand)

River sand from Sylhet, Bangladesh was used as fine aggregate. Different properties such as specific gravity and absorption capacity were determined by ASTM C 128, dry rodded unit weight was determined by ASTM C 29, Fineness Modulus (F.M) was determined by

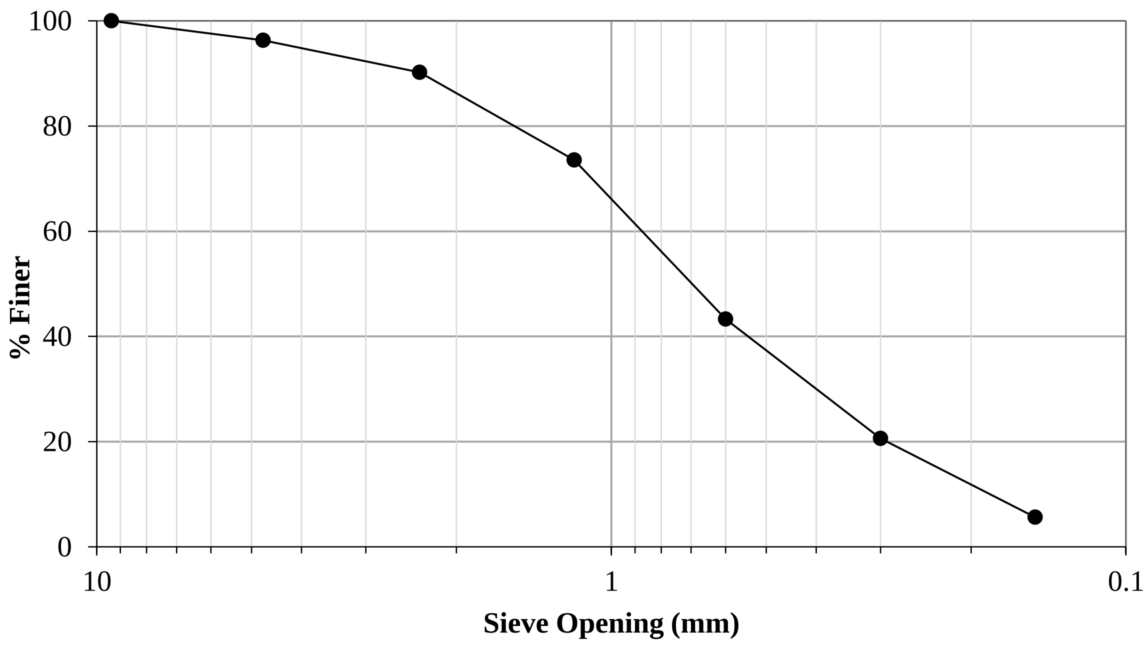

Figure 4.2: Grain size distribution curve of fine aggregate

ASTM C 136 and moisture content by ASTM C 556. Different properties of fine aggregate are shown in Table 4.1. The grain size distribution curve is shown in Figure 4.2 according to ASTM C 136.

4.2.3 Binder (Ordinary Portland Cement)

ASTM C 150 Type I cement (Ordinary Portland Cement) was used as a binder material. The properties of the cement were followed by (ASTM C 150). This cement was only two basic ingredients; clinker and gypsum (calcium sulfate). Generally, producers use at least 95% of cement clinker and rest gypsum. Various properties of binder (OPC) are shown in Table 4.2.

Table 4.2: Different properties of binder (OPC)

Compressive Strength (MPa)	3days	25.4
	7days	35.6
	28days	44.5
Setting Time (Min)	Initial Setting	140
	Final Setting	260

4.2.4 Reinforcing Steel

BSRM 6mm, 10mm and 12mm deformed bars were used for this experimental study. 6mm bars were used as shear reinforcement or stirrups, 10mm bars were used as main reinforcement of all beams and 12mm bars were used for flexural strengthening of beams. The tensile properties such that yield strength and ultimate strength were determined according to ASTM A 370. The properties of reinforcing steel bars are shown in Table 4.3.

Table 4.3: Tensile strength of steel bar, steel angle & steel plate

Description	Yield Strength (MPa)	Ultimate Strength (MPa)
Ø6 mm bar	565	668
Ø10 mm bar	552	643
Ø12 mm bar	547	647
Steel Angle	426	530
Steel Plate	278	364

4.2.5 Steel Plate

(125mm × 3mm) steel plate was used for externally strengthening of beams. The tensile properties were determined according to ASTM A 370. The properties of the steel plate are shown in Table 4.3.

4.2.6 Steel Angle

(25mm × 25mm × 4mm) steel angles were used for externally strengthening of beams. The tensile properties were determined according to ASTM A 370. The tensile test results of steel angles are shown in Table 4.3.

4.2.7 Adhesive

Two types of adhesive (Epoxy-1 & Epoxy-2) were used in this study. Epoxy-1 is a multipurpose epoxy adhesive and Epoxy-2 is a Thixotropic epoxy adhesive. Compressive strength, tensile strength and shear strength were provided by the manufacturer. Bond strength was determined according to ASTM C 1583. The properties of the adhesives are shown in Table 4.4.

Table 4.4: Properties of adhesives

Properties	Epoxy-1	Epoxy-2
Compressive Strength (MPa)	-	75
Tensile bending strength (MPa)	33	Concrete Failure
Shear Strength (MPa)	17.6	13
Bond Strength (MPa)	0.95	1.22

4.3 Sample Preparation

ø10 mm bar was cut into pieces for use as main reinforcement and made anchorage on each side of the main reinforcement. 112mm × 212mm stirrups were prepared by cutting ø6 mm bar. Twelve steel cases were prepared by using 4-ø10 mm main bars and 27-ø6 mm stirrups @ 100mm c/c. Main steel bars were tied with stirrups by using steel wire. Figure 4.3 shows the typical steel case detailing.

Figure 4.3: Typical steel case detailing

Formworks were prepared with wood for the casting of 150mm × 250mm × 2700mm beams as described in ACI 318-08 section 6.1. Polyethylene sheet was used on the inner side for preventing water absorption by the wood and for providing a smooth surface to the beams. Total twelve beams were cast in this manner.

4.4 Concrete Mix Design

Concrete was designed for a target strength of 34.5MPa according to ACI 211.1. The proportion of concrete (cement ∶ fine aggregate ∶ coarse aggregate) was (1∶2.24∶2.08) by volume and (1∶1.82∶1.88) by weight with a water to cement ratio of 0.48 by weight. The target slump was 75-100mm. Table 4.5 shows total ingredients for one beam.

Table 4.5: Ingredients for concrete mixture (one beam)

Cement (kg)	Fine Aggregate (kg)	Coarse Aggregate (kg)	Water (kg)	Total (kg)
50.19	91.36	94.37	24.09	260.00

4.5 Casting and Curing

Concrete was mixed according to ASTM C192 using a standard concrete mixer. Slump test was performed according to ASTM C143. ø100 mm × 200 mm concrete cylinders were cast and compacted according to ASTM C31 in two layers with 25 rods per layer for determination of compressive strength of concrete. The beams were cast and cured according to ASTM C192. Standard vibrator was used to compact the concrete properly. The compressive strength of concrete was determined at 3 days, 7 days, 14 days and 28 days of curing. Figure 4.4 & Figure 4.5 show the casting and curing of RC beams.

Figure 4.4: Casting of reinforced concrete beams

Figure 4.5: Curing of reinforced concrete beams

4.6 Determination of Compressive Strength

The compressive strength of concrete is one of the most important and useful properties of concrete. Compressive strength was measured by the resistance of concrete under compressive force. The compressive strength of concrete was determined by testing cylinders made in the laboratory according to ASTM C39. Figure 4.6 shows the procedure of compressive strength test of concrete.

Figure 4.6: Illustration of compressive strength test

The load was applied to the specimen by using a compressive strength testing machine and failure load was noted. Compressive strength was calculated by using equation 4.1.

$$C = P/A \text{ - } 4.1$$

Where,

$C = Compressive\ Strength$

$P = Failure\ Load$

$A = Contact\ Area$

The compressive strength of concrete at different age is shown in Figure 4.7. The compressive strength at 3days, 7days, 14days, and 28days was 14.7MPa, 23.3MPa, 30.3MPa and 33.3MPa respectively.

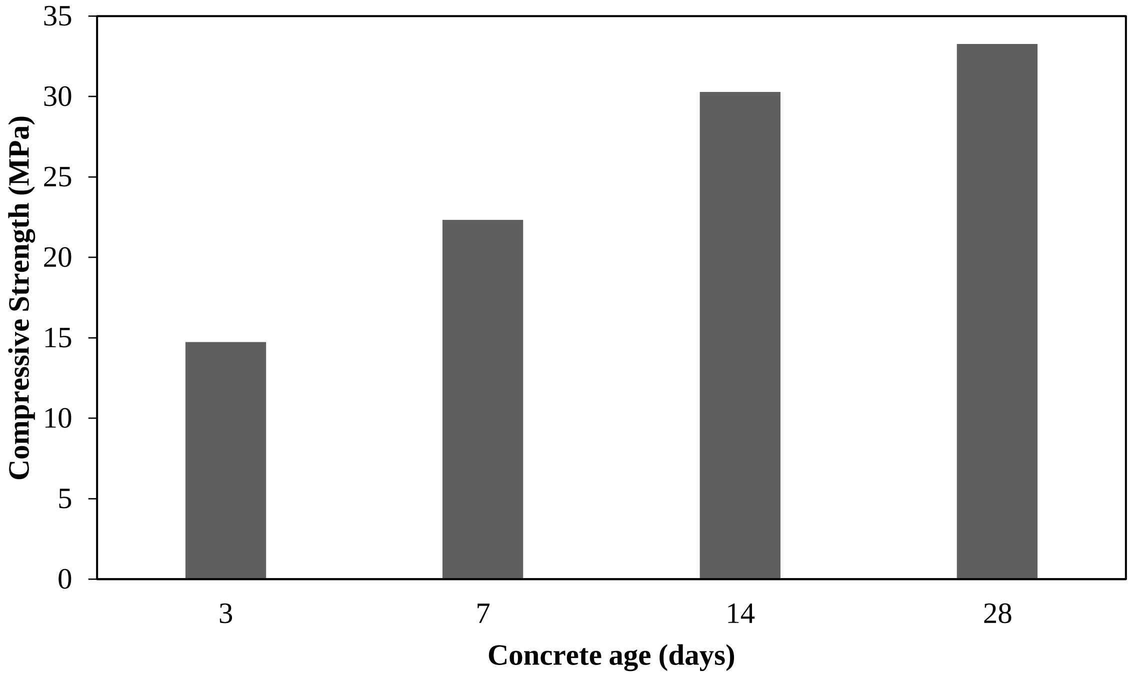

Figure 4.7: Compressive strength at different ages

4.7 Flexural Strength Test of Control Beams

After 28 days curing, two beams (B1-1 & B1-2) were tested up to the failure load to determine the ultimate flexural capacity of control beams by using 3rd point loading according to ASTM C78. The beams were simply supported at a clear span of 2400mm and loaded symmetrically in four-point bending with point loads 400mm on each side of the center line of the beam. A 300kN hydraulic jack was used for the application of load arranged vertically and divided into two equal point loads through a transfer beam. The deflection of transfer beams at maximum capacity of strengthened beams was calculated 0.5mm which was neglected on the test performed. The rate of loading was maintained at 4kN/min according to ASTM C78. A 500kN load cell and 5 LVDTs (Linear variable differential transformer) were used to collect data directly by TML TDS-303 data logger. The 1st LVDT was placed at the center of the beam, 2nd and 3rd LVDTs were placed under the beam to the point load. 4th and 5th LVDTs were placed in midpoint of support and the point load. A schematic diagram of the test setup is shown in Figure 4.8 and the picture taken during the test is shown in Figure 4.9. The accuracy of the Load cell and LVDTs were 0.01kN and 0.01mm respectively. The range of 3 LVDTs (LVDT-1, LVDT-2 & LVDT-3) was 100mm and for the other 2 LVDTs (LVDT-4 & LVDT-5), it was 50mm. The load cell & LVDTs were calibrated before the test was performed.

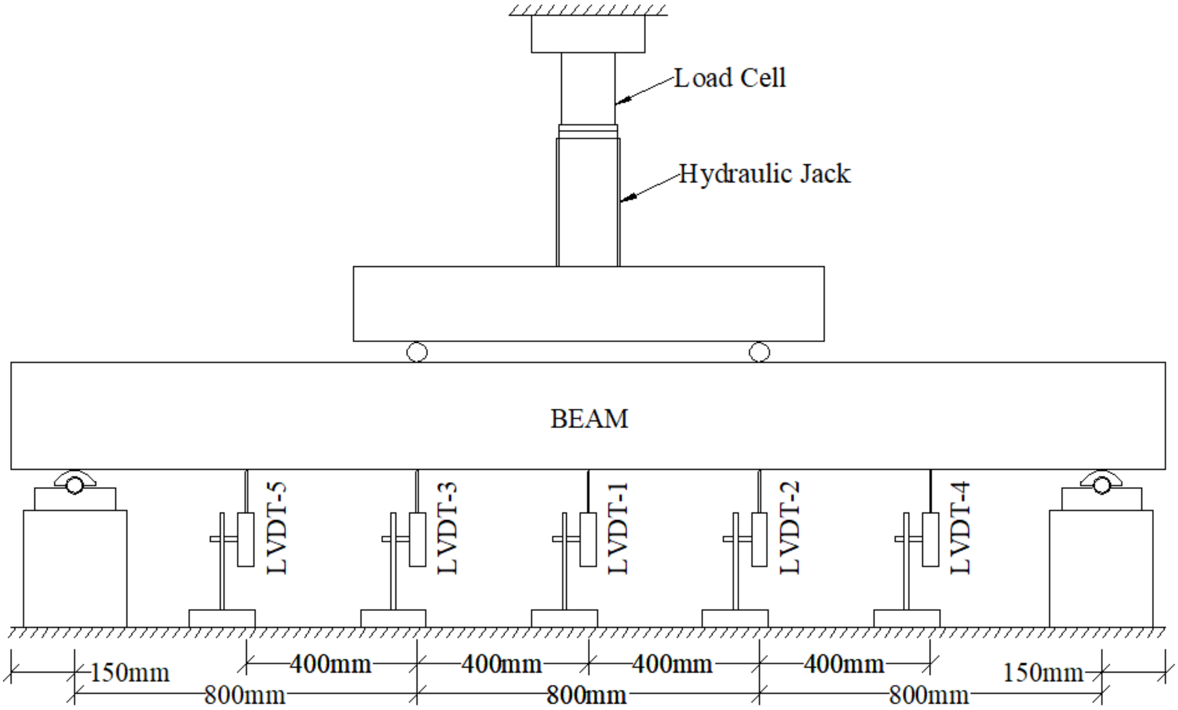

Figure 4.8: Schematic diagram of the test setup

Figure 4.9: Experimental setup for the flexural test

4.8 Preloading of Beams Before Strengthening

To investigate the effect of service load on the strengthening technique, all beams (except control beams) were preloaded before strengthening. One beam from each group was preloaded with 75% and the other one beam was preloaded with 65% of ultimate load bearing capacity of control beams. The load was applied in the same way as the test of control beams. By the application of preloading, some initial cracks were formed. After the observation of the initial cracks and mark it by a permanent black marker, the load was released and the permanent deformation was noted. All data were recorded by TML TDS 303 data logger automatically.

4.9 Strengthening of Beams

After the application and release of preloading, the beams were ready for strengthening. Five techniques were applied for the strengthening of beams. The strengthening techniques were applied after the beams turn in reverse (bottom face on top). As the beams designed with 2 ø10 mm steel bars so the effect of self-weight was negligible on the capacity of beams and strengthening techniques.

4.9.1 Strengthening with External Steel Plate

Two beams (B2-1 & B2-2) were strengthened with external steel plate bonded with epoxy adhesive. Firstly, the bottom surface was made rough enough by grinding for proper bonding. The surface was cleaned with a blower before the application of epoxy adhesive. Then epoxy adhesive was applied on the bottom face of the beam and one side of the steel plate. After that the steel plate was attached on the surface of the beam. Figure 4.10 shows the strengthening of the beam by external steel plate bonded with epoxy adhesive.

Figure 4.10: Strengthening with external steel plate

A corrosion protective coating is required to protect the steel plate from corrosion. Different types of corrosion protection techniques are available to protect the steel plate from corrosion. Corrosion of steel plate can be protected by passive barrier protection, active protection and sacrificial protection (Corrosion protection of Steel., n.d.).

4.9.2 Strengthening with External Steel Plate Anchorage by Steel Bolts

For other two beams (B3-1 & B3-2) two holes were drilled with a hammer drill on each side of the beam and also made two holes in the steel plate as like the beam for the use of bolts. The holes of each beam were cleaned by water jet and dried. After placing the steel plate, all the holes were filled with epoxy adhesive and the steel bolts were placed in the hole. All the steel plates were placed at a distance 200 mm from each side of the beams where the supports were placed at a distance of 150 mm from the side of the beam so that there remained a 50 mm clearance between the support and the end of steel plate. The length of each plate was 2300 mm. Two different types of epoxy adhesive were used in two different beams. Figure 4.11 shows the strengthening of the beam by external steel plate bonded with epoxy adhesive and anchored by steel bolts.

Figure 4.11: Strengthening with external steel plate and bolts

4.9.3 Strengthening with Near Surface Mounted (NSM) Steel Bars

Two beams (B4-1 & B4-2) were strengthened by using Near Surface Mounted (NSM) steel bars. Two grooves were made on the bottom concrete face for the placement of NSM bars in each beam. Then the grooves were cleaned by an air jet. After that, the grooves were filled with epoxy adhesive up to the half depth of the grooves and the steel bars were placed and filled the remaining depth of the grooves with epoxy adhesive. For the real-life NSM technique, the steel bars can be attached by using jacking in several places to initial setting.

After that, the jacking should remove and fill the place with epoxy. The steel bars were placed at a distance 200 mm from the side of the beam. The length of each bar was 2300 mm. Two different types of epoxy adhesive were used in two different beams. Figure 4.12 shows the strengthening procedure with NSM bars.

Figure 4.12: Strengthening with NSM steel bars

4.9.4 Strengthening with External Steel Angles

Two beams (B5-1 & B5-2) were strengthened with external steel angles. The bottom concrete cover and the side concrete cover (up to 50mm) were removed to add steel angles with the bottom corners of the stirrups. Steel angles (25mm × 25mm × 4mm) were cut in pieces of 2300mm. The bottom face was cleaned and added two steel angles in two sides of the beams with bottom stirrups by welding. The distance of steel angles was 200mm from the side of the beam. After attaching the steel angles, the beams were cleaned with a blower and the exposed part was covered with concrete again. The strengthening procedure with external steel angles is shown in Figure 4.13.

Figure 4.13: Strengthening with external steel angles

4.9.5 Strengthening with External Steel Bars

Rest two beams (B6-1 & B6-2) were strengthened with external steel bars. The bottom concrete cover was removed to add steel bars with the stirrups. The bottom face was cleaned and three steel bars (ø12 mm and 2300mm long) were attached with bottom stirrups by welding. The distance of each bar was 200mm from the side of the beam. After attaching the steel bars, the beams were cleaned with a blower and the exposed part was covered with new concrete. Figure 4.14 shows the strengthening procedure with external steel bars.

Figure 4.14: Strengthening with external steel bars

4.10 Test of Strengthened Beams

To evaluate the strengthening performance by different techniques all beams were tested by 3rd point loading according to ASTM C78 as the same process of the test of control beams. The supports were placed at a distance of 150mm from the side of the beam which was 50mm from the end of external steel (steel plate, steel bars, and steel angles).

CHAPTER V

RESULTS & DISCUSSION

5.1 General

This chapter summarizes the experimental and calculated results of all tests performed in this study. Test results of the control specimens, preloading, and test results of the strengthened beams are presented in this chapter. The comparison of results is also presented in this chapter.

5.2 Beams Test Results

3rd point loading was performed to investigate the flexural performance of control and strengthened beams. The summarizes results of all beams are shown in Table 4.2.

Table 5.1: Summary of beams test results

Series	Specimen ID	$P_{u(c)}$ (KN)	$P_{u(e)}$ (KN)	Δ_u (mm)	I_u (%)	$\frac{P_{u(c)}}{P_{u(e)}}$
Control	B1-1	43.2	47.9	69.22	N/A	1.11
	B1-2		50.4	53.32	N/A	1.17
Steel Plate	B2-1	105.0	59.5	4.12	21	0.57
	B2-2		92.4	6.33	88	0.88
Steel Plate & Bolts	B3-1		90.8	6.60	85	0.86
	B3-2		100.4	29.46	104	0.96
NSM bars	B4-1	107.3	101.9	12.64	108	0.95
	B4-2		115.0	25.70	134	1.07
Steel Angles	B5-1	118.2	124.4	39.20	153	1.05
	B5-2		116.8	14.76	138	0.99
Steel bars	B6-1	133.6	127.5	17.22	160	0.95
	B6-2		134.0	12.44	173	1.00

$P_{u(c)}$= calculated ultimate capacity; $P_{u(e)}$= ultimate experimental load bearing capacity; Δ_u= deflection at ultimate load; I_u Increase on ultimate load bearing capacity.

5.2.1 Control Beams

The reinforced concrete beam (B1-1) showed elastic behavior up to a load of 16.5kN and the corresponding mid-span deflection at elastic load was 1.38mm. The destruction of stiffness was started after the elastic load of 16.5kN and the relation was again linear up to a load of 44.5kN. The mid-span deflection was 10.55mm at that stage. The load-deflection relationship of the control beam (B1-1) is shown in Figure 5.1.

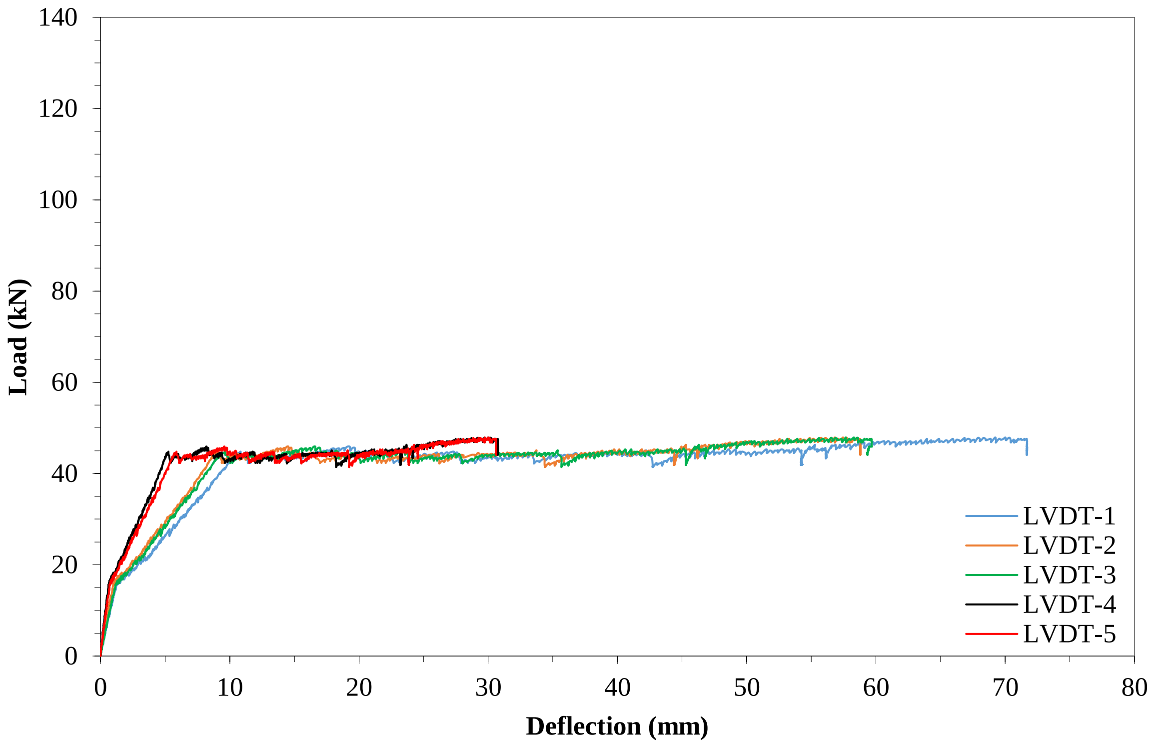

Figure 5.1: Load versus deflection curve of control beam (B1-1)

The first flexure crack formed at the bottom of the beam surface at a load of 23.9kN at a mid-span deflection of 4.26mm and propagated rapidly towards the upper part of the beam. These cracks were in the region of the maximum bending moment. It was total 11 major cracks and some minor cracks. The ultimate load was 47.9kN at a mid-span deflection of 69.22mm. The calculated load carrying capacity of the control beam was 43.2kN while the experimental ultimate load carrying capacity was found to be 11% higher than the calculated load carrying capacity. The ductility index was 6.59 for the control beam (B1-1). The beam failed by flexure after the yielding of steel reinforcement and showed a pure bending behavior. The crack pattern and failure mode of the control beam (B1-1) are shown in Figure 5.2.

Figure 5.2: Failure mode and crack pattern of control beam (B1-1)

The 2nd control beam (B1-2) showed the same behavior as to control beam (B1-1) in flexure. In this case, the elastic load was 14.5kN and the corresponding mid-span deflection at the elastic load was 1.30mm. The stiffness reduction was started after the elastic load. The load-deflection relationship was again linear up to a load of 45.8kN at a mid-span deflection of 9.82mm. Figure 5.3 shows the load versus deflection relationship of the control beam (B1-2).

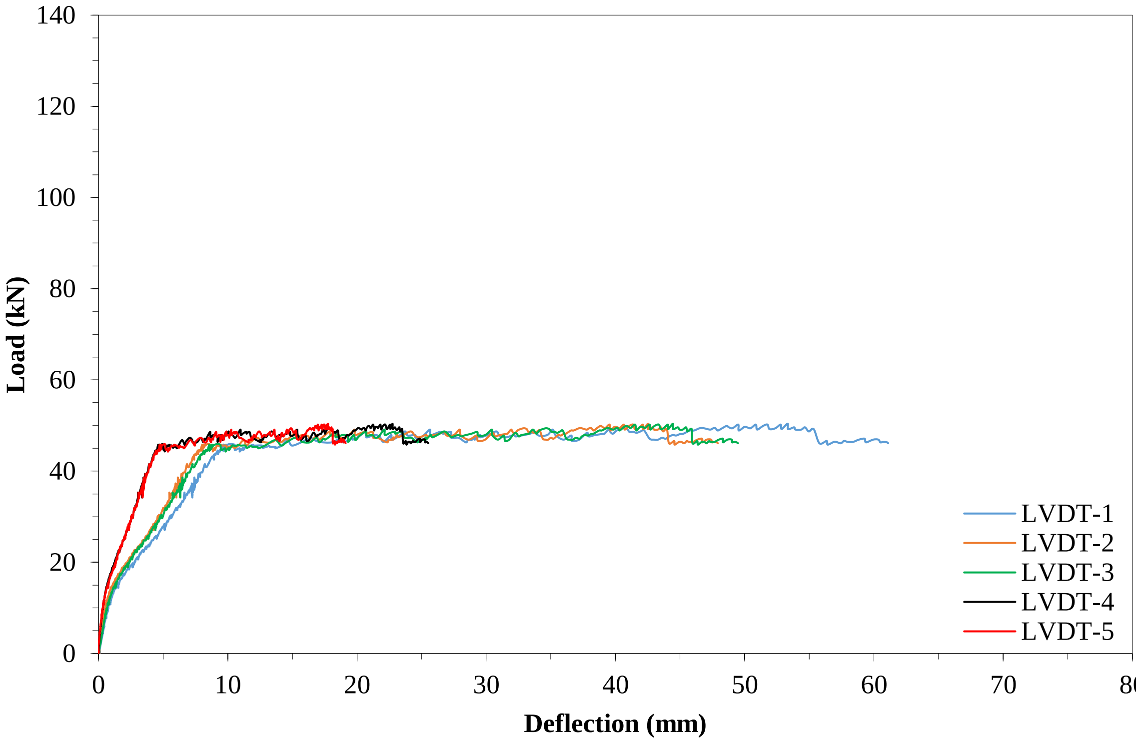

Figure 5.3: Load versus deflection curve of control beam (B1-2)

First flexure crack was observed at the bottom of the beam surface at a load of 24.8kN and at a mid-span deflection of 4.12mm. The cracks spread rapidly towards the upper part of the beam. It was 9 major cracks and some minors. Most of these cracks were in the region of the maximum bending moment. The ultimate load was 50.4kN at a mid-span deflection of 53.22mm. The experimental ultimate load carrying capacity was found to be 17% higher than the calculated load carrying capacity which was 43.2kN. The ductility index was 5.41 for the control beam (B1-2). The beam failed by flexure and showed a pure bending behavior. The crack pattern and failure mode of the control beam (B1-2) are shown in Figure 5.4.

Figure 5.4: Failure mode and crack pattern of control beam (B1-2)

5.2.2 Application of Pre-load

Load versus mid-span deflection curves for preloading and unloading up to 75% of the ultimate load are shown in Figure 5.5 and preloading and unloading up to 65% of the ultimate load are shown in Figure 5.6. All the beams were shown similar behavior for preloading and have some permanent deflection after unloading. The beams showed a similar type of cracks due to preload. The maximum applied load for preloading, maximum deflection with the corresponding load, permanent deflection after unloading, the number of cracks form due to preloading are shown in Table 5.2. Maximum deflections for application of 75% load were between (6.9 - 7.58mm) which were (3.88 - 4.60mm) for application of 65% load. Permanent deflections were (1.36 - 2.36mm) for 75% loading and (0.98 - 1.34mm) for 65% loading. The load-deflection relationship and crack pattern of all beams due to preloading are shown in Appendix A.

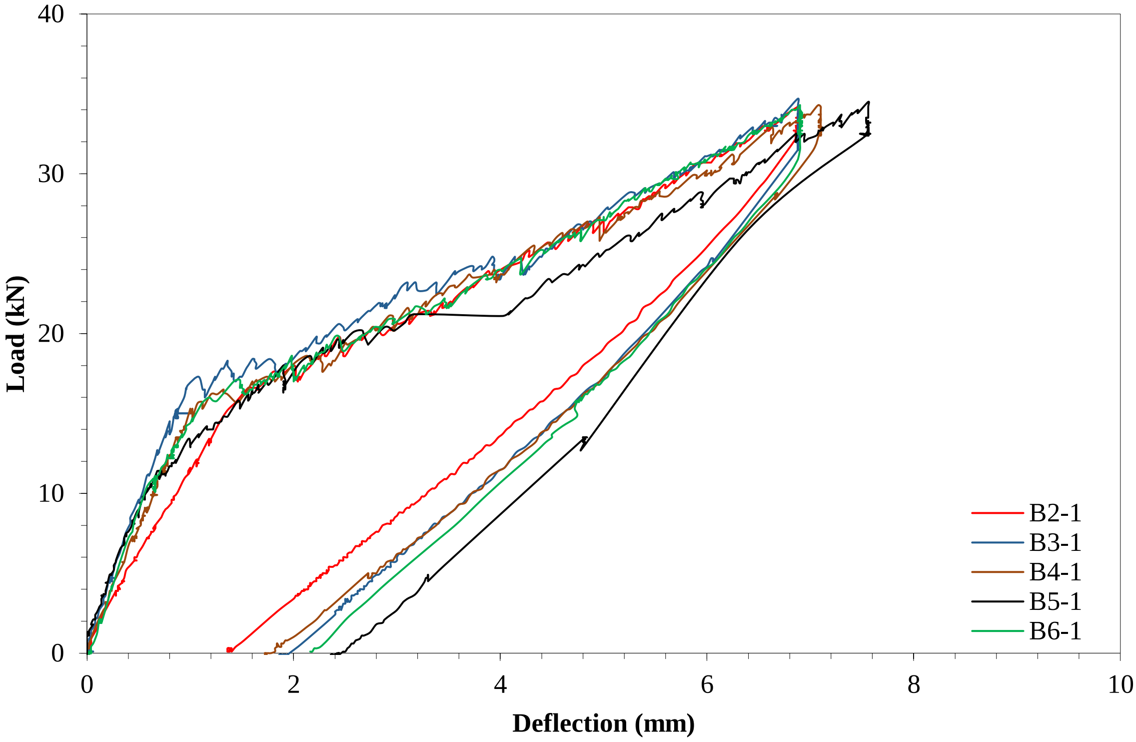

Figure 5.5: Load versus mid-span deflection curve for application of 75% load

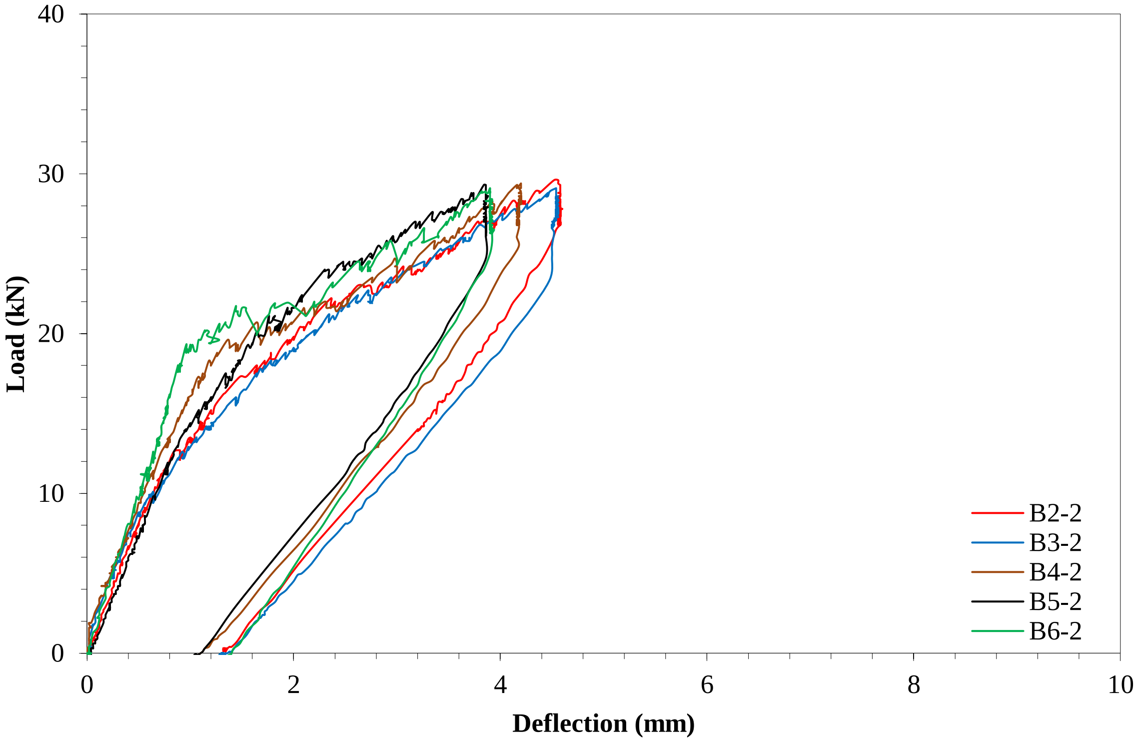

Figure 5.6: Load verses mid-span deflection curve for application of 65% load

Table 5.2: Summary of preloading

Specimen ID	Applied Load (kN)	Maximum Deflection (mm)	Permanent Deflection (mm)	No of cracks
B2-1	34.2	6.90	1.36	14
B2-2	29.6	4.60	1.34	10
B3-1	29.1	4.56	1.28	10
B3-2	34.7	6.90	1.82	14
B4-1	34.3	7.10	1.72	15
B4-2	29.4	4.20	0.98	11
B5-1	34.5	7.58	2.36	10
B5-2	29.3	3.88	1.32	7
B6-1	34.3	6.92	2.16	16
B6-2	29.1	3.94	1.04	8

5.2.3 Beams Strengthened with External Steel Plate

Figure 5.7 shows the load-deflection relationship of beam strengthened with external steel plate (B2-1) bonded with epoxy adhesive (Epoxy-1). The beam showed elastic behavior before the separation of steel plate due to the failure of the bond between concrete and steel plate. Although the steel plate is a ductile material the beam showed a brittle behavior due to the separation of steel plate. The steel plate separated before the yield stress of the steel plate, thus the beam showed brittle behavior. The steel plate separated at a load of 59.6kN while the mid-span deflection was 4.12mm. After the separation of steel plate, the load reduced to the level of ultimate load of controlled beams. The calculated load carrying capacity of that beam was 105.0kN while the experimental ultimate load carrying capacity was only 57% of the calculated load carrying capacity. As there was no yield point so the ductility index was 1. The beam failed by flexure after the separation of steel plate from the bottom surface. Figure 5.8 shows the failure mode and crack pattern of beam strengthened with external steel plate (B2-1). Seven new cracks (red marking) formed in different place from preloaded cracks (black marking) of this beam. The length and width of new cracks were smaller than preloaded cracks. The alignment of some new cracks was different from others.

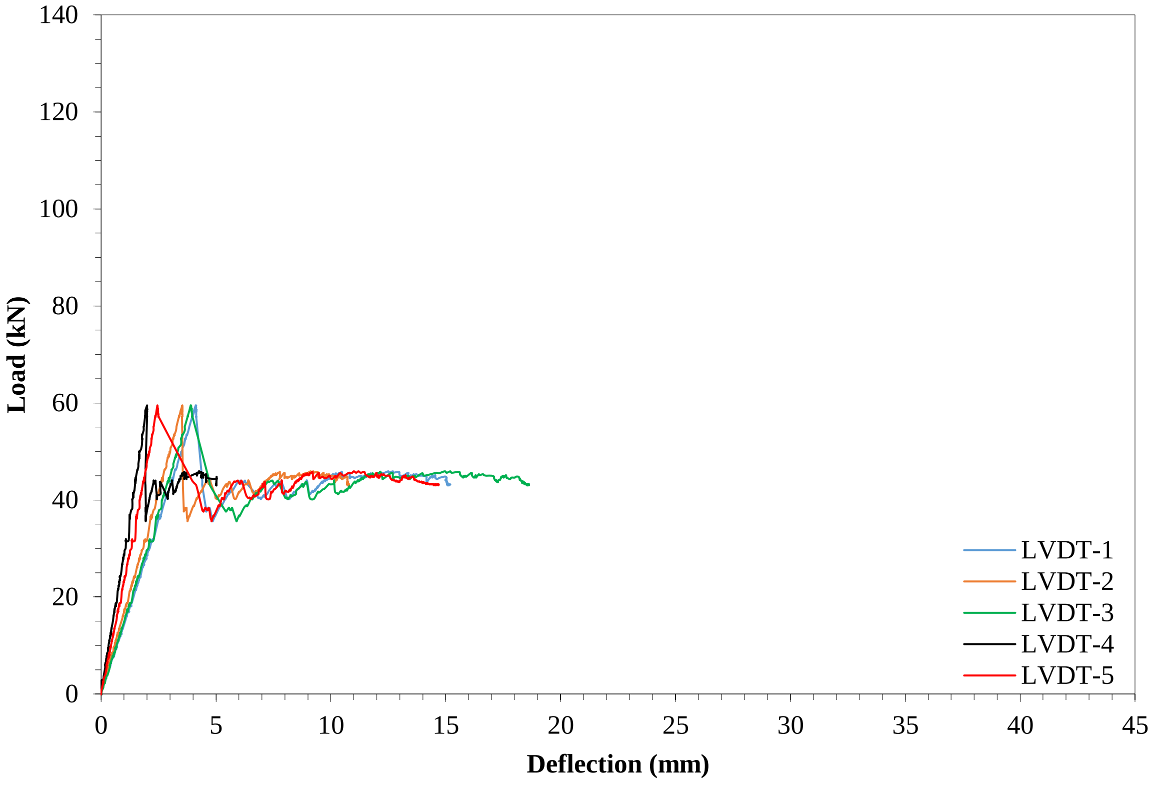

Figure 5.7: Load versus deflection of beam strengthened with steel plate (B2-1)

Figure 5.8: Failure mode of beam strengthened with steel plate (B2-1)

Figure 5.9 shows the load-deflection relationship of beam strengthened with external steel plate (B2-2) bonded with epoxy adhesive (Epoxy-2). The beam showed the same electric behavior as the beam B2-1 up to the separation of steel plate due to the failure of the bond between concrete and steel plate. The steel plate separated before the yielding of steel plate at a load of 92.4kN while the mid-span deflection was 6.33mm. After the separation of steel plate, the load reduced to the level of ultimate load of controlled beams. The beam failed by flexure after the separation of steel plate. The ultimate capacity was 88% of the calculated load carrying capacity of that beam. The ductility index was 1 due to no yield point. The cracks pattern and failure mode of the beam strengthened with external steel plate (B2-2) are shown in Figure 5.10. Six new cracks (red marking) formed in different position of the preloaded cracks (black marking). The length of new cracks and width were smaller than preloaded cracks. The numbers of new crack were less than preloaded cracks. Some cracks were aligned in different directions of preloaded cracks.

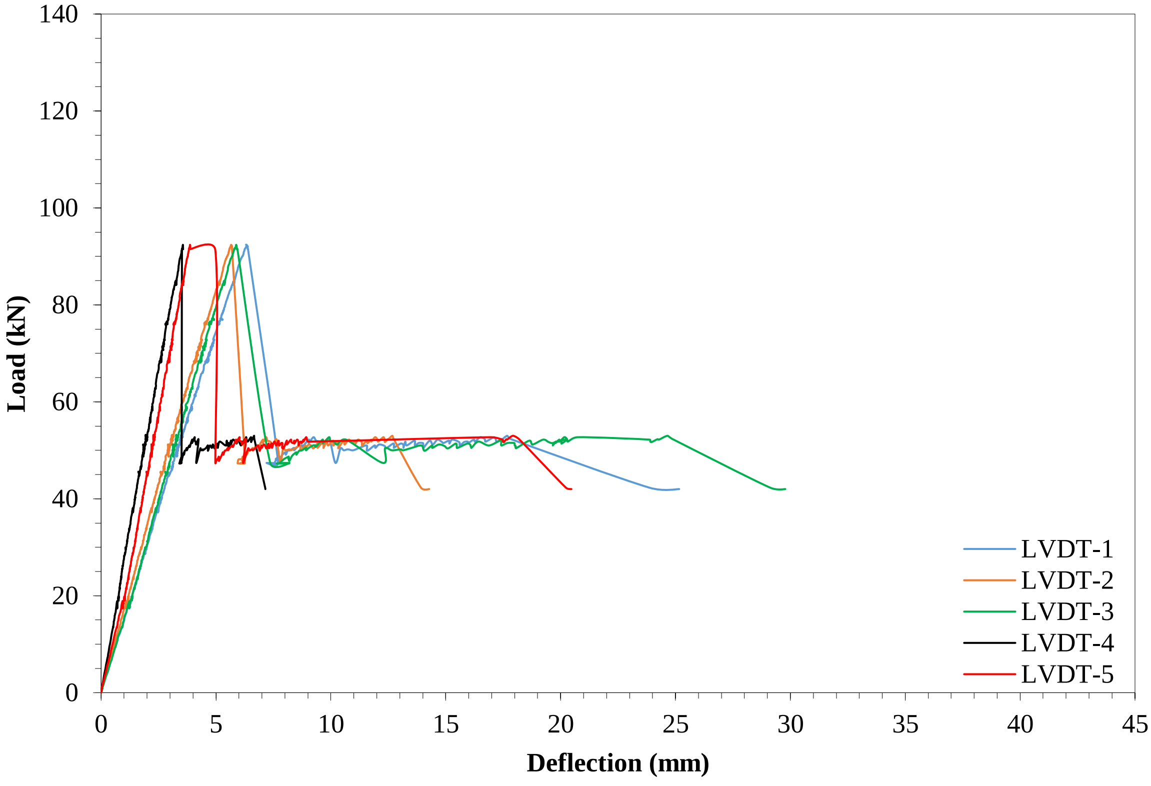

Figure 5.9: Load versus deflection of beam strengthened with steel plate (B2-2)

Figure 5.10: Failure mode of beam strengthened with steel plate (B2-2)

5.2.4 Beams Strengthened with External Steel Plate Anchorage by Steel Bolts

Load versus deflection curves of beam strengthened with external steel plate bonded with epoxy adhesive (Epoxy-1) and anchorage by bolts are shown in Figure 5.11. The beam showed elastic behavior up to a load of 90.8kN while the mid-span deflection was 6.66mm. The load reduced to 65.4kN while the separation of steel plate started at 90.8kN. The beam showed ductile behavior with further increase of load. Finally, the beam failed in flexure by the pull of steel bolts at a load of 83.8kN. The ultimate load bearing capacity was 90.8kN which was 86% of the calculated load bearing capacity of this beam was 105.0kN. Although the behavior of two beams (B2-1 & B3-1) are similar the load carrying capacity increased 59.6kN (for B2-1) to 90.8kN (for B3-1) by using of steel bolts anchorage. The ductility index was 1 for beam B3-1. The failure mode and crack pattern of beam strengthened with external steel plate bonded with epoxy adhesive and anchorage by steel bolts are shown in Figure 5.12. Total 11 new cracks (red) formed in different places of preloaded cracks (black). Some of the new cracks were in the shear region and the length of those cracks was more than preloaded cracks. The width of new cracks was smaller than preloaded cracks.

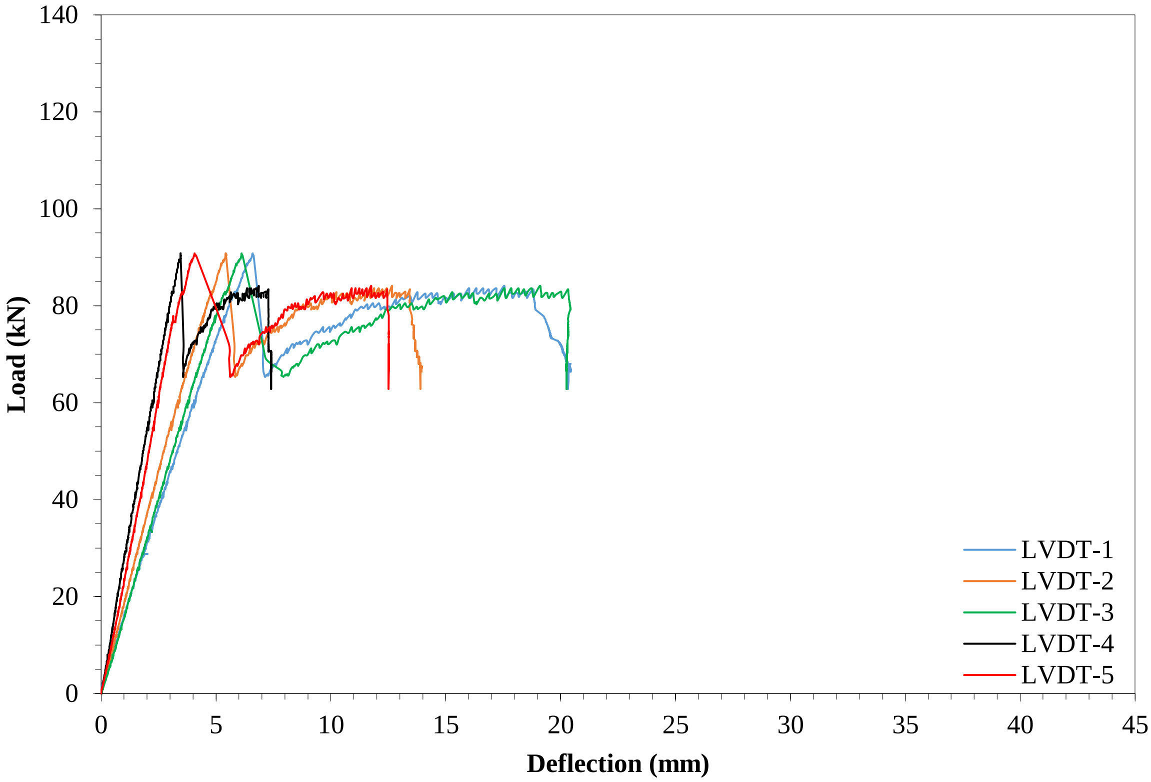

Figure 5.11: Load versus deflection of beam strengthened with steel plate and bolts (B3-1)

Figure 5.12: Failure mode of beam strengthened with steel plate and bolts (B3-1)

Beam (B3-2) strengthened with external steel plate bonded with epoxy adhesive (Epoxy-2) and anchorage by bolts showed elastic behavior up to a load of 83.9kN with a deflection of 6.28mm at the mid-span. The separation of the steel plate started at that stage. The beam showed ductile behavior with further increment of load. The ultimate load-bearing capacity of this beam was 100.4kN while the bolts failed in shear. The mid-span deflection was 29.46mm at the failure load which was higher than that observed in the beam with Epoxy-1 (Beam B3-1). The beam failed in flexure after the failure of bolts in shear. The experimental ultimate load carrying capacity was 96% of the calculated load carrying capacity. Figure 5.13 shows the load-deflection relationship of beam strengthened with steel plate and anchored by bolts. The ductility index for this beam was 4.12 which was greater than the other beams strengthened with steel plate without bolts. Some reverse deflection was observed on LVDT-1 and LVDT-3. This may be due to shaking for sudden deflection at the start of separation of steel plate. The Failure mode and crack pattern of B3-2 are shown in Figure 5.14. Total 4 new diagonal cracks formed in different places of preloading cracks. The length and width of cracks were smaller than preloaded cracks.

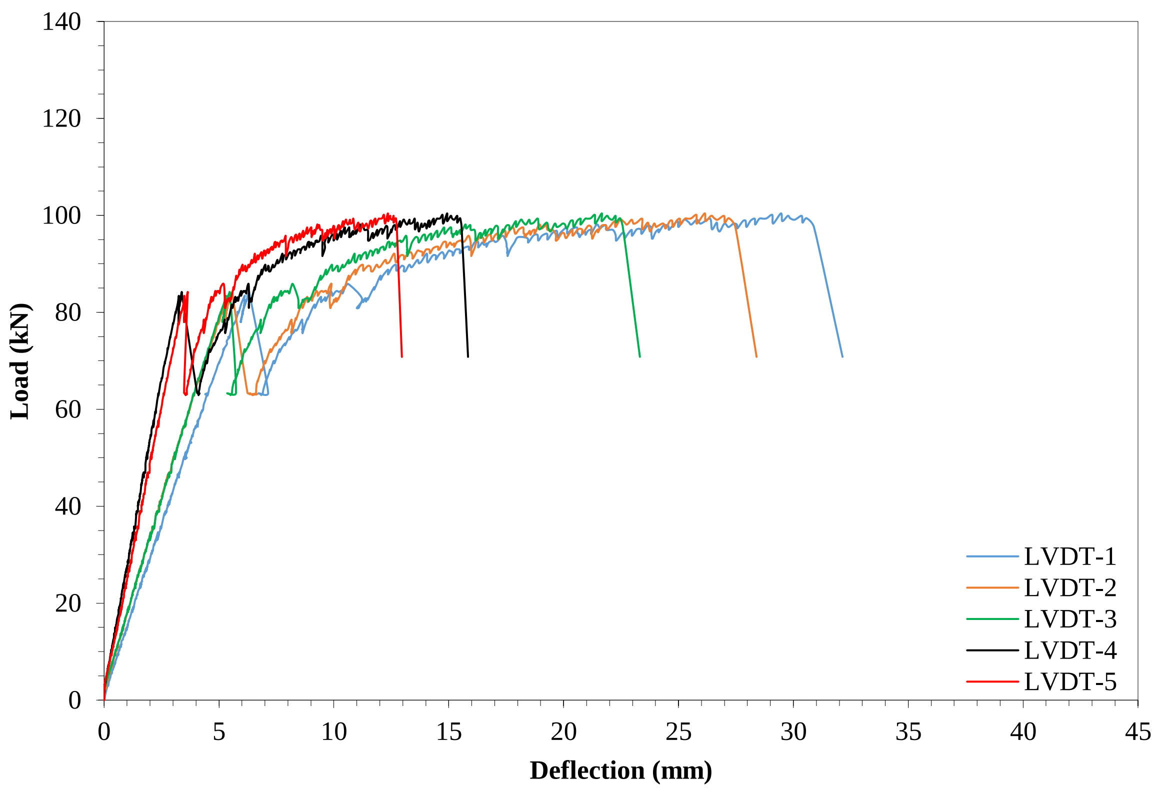

Figure 5.13: Load versus deflection of beam strengthened with steel plate and bolts (B3-2)

Figure 5.14: Failure mode of beam strengthened with steel plate and bolts (B3-2)

5.2.5 Beams Strengthened with Near Surface Mounted (NSM) Bars

The flexural test results of beam strengthened with NSM bars (B4-1) bonded with epoxy adhesive (Epoxy-1) are shown in Figure 5.15. The beam showed elastic behavior up to a load of 63.3kN while the mid-span deflection was 6.52mm. The relationship was again linear up to a load of 94.2kN and a deflection of 10.78mm. The ultimate load was 101.9kN at mid-span deflection of 12.64mm. The calculated ultimate load for beam strengthened with NSM bars was 107.3kN. The experimental ultimate load carrying capacity was 95% of the calculated load carrying capacity. Although steel is a ductile material, the beam showed a brittle behavior due to the separation of steel bars from the grooves of the beam by bond failure. The ductility index was 1 for B4-1. The beam was failed in flexure after the separation of steel bars from the grooves. The failure mode and crack pattern of beam strengthened with NSM bars (B4-1) are shown in Figure 5.16. Total 7 new cracks (red) formed in this beam some of these were diagonal cracks. The width of the new cracks was smaller than the preloaded cracks (black).

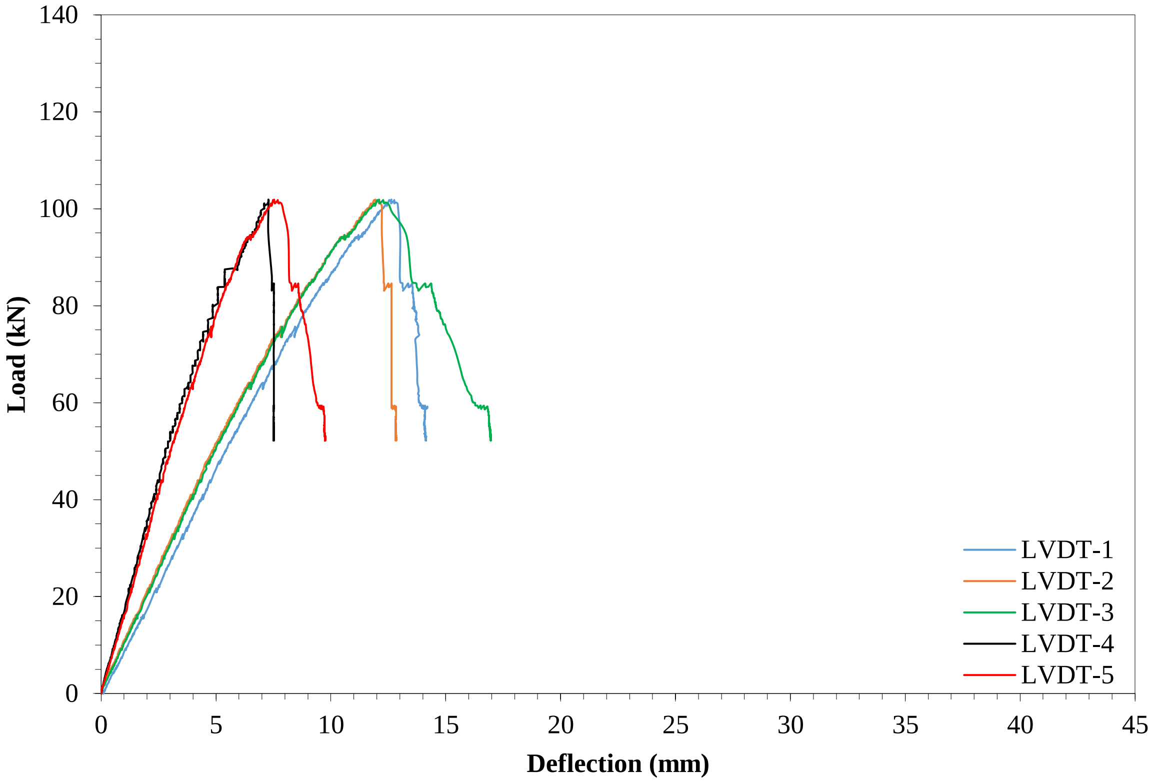

Figure 5.15: Load versus deflection of beam strengthened with NSM bars (B4-1)

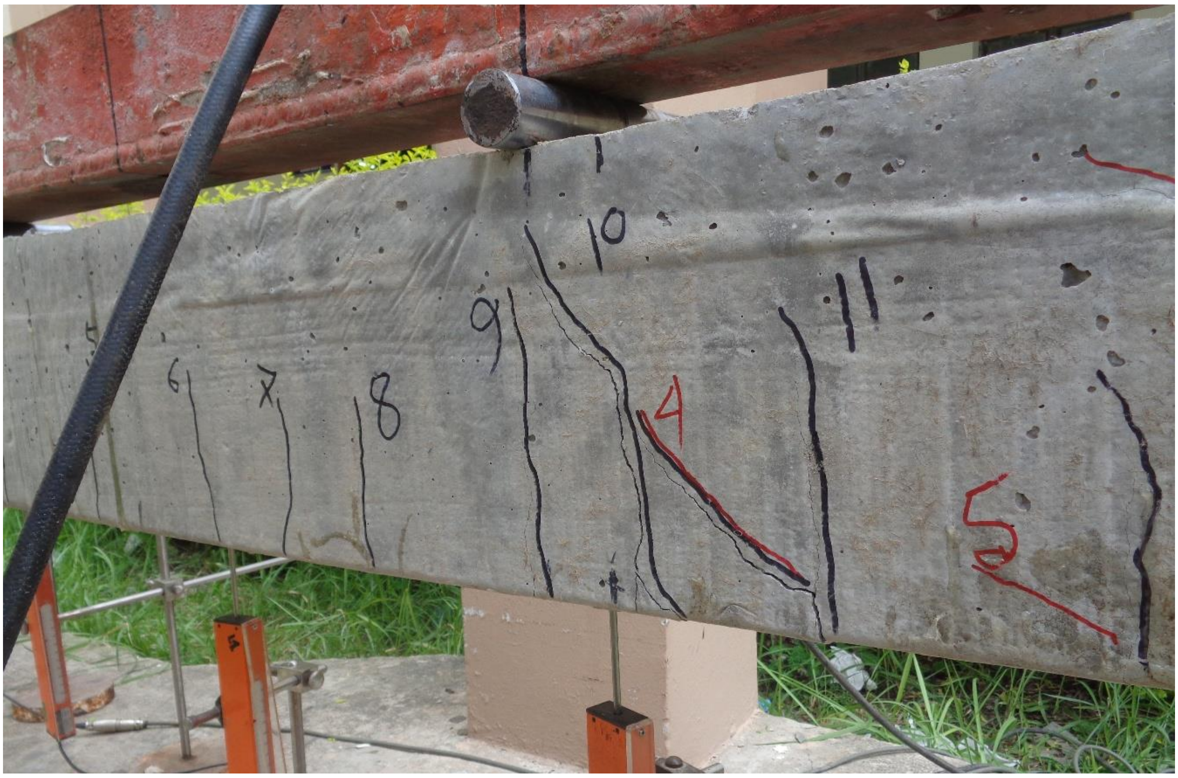

Figure 5.16: Failure mode of beam strengthened with NSM bars (B4-1)

The 2nd beam strengthened with NSM steel bars (B4-2) bonded with epoxy adhesive (Epoxy-2) showed similar elastic behavior as B4-1. In this case, the elastic load was 53.3kN at mid-span deflection of 5.28mm and the relationship was again linear up to a load of 112.2kN while the mid-span deflection was 12.51mm. The load-deflection curves of beam B4-2 are shown in Figure 5.17. The beam showed ductile behavior in flexure test. The beam failed in flexure at the ultimate load of 115.0kN at mid-span deflection of 25.7mm. The experimental ultimate load carrying capacity was 107% of the calculated load carrying capacity and the ductility index was 1.99. The beam failed in flexure and showed pure bending behavior through the yielding of main steel reinforcement and attached NSM steel bars. The failure mode and crack pattern of beam strengthen by NSM steel bars (B4-2) are shown in Figure 5.18. Total 11 new cracks (red) formed in this beam. Some of them were in the shear region and some of them are in the flexural region. The length of new cracks was varied, but the width of new cracks was smaller than the preloaded cracks at ultimate load.

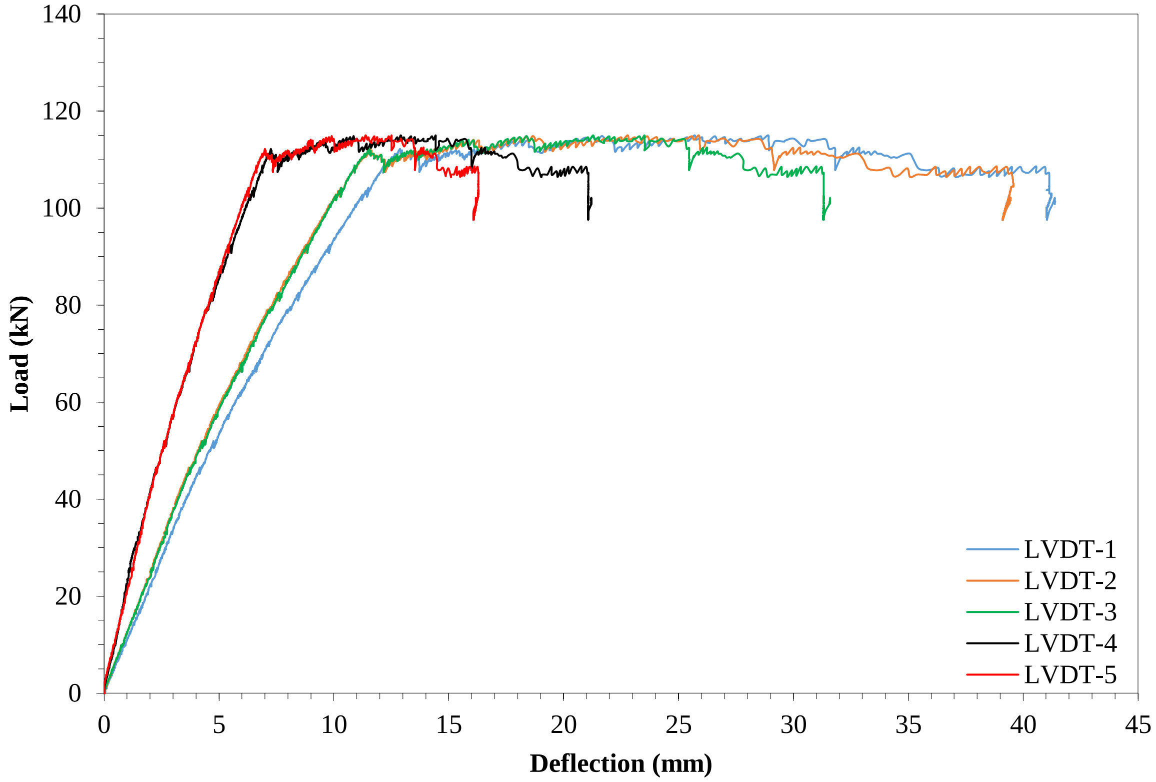

Figure 5.17: Load versus deflection of beam strengthened with NSM bars (B4-2)

Figure 5.18: Failure mode of beam strengthened with NSM bars (B4-2)

5.2.6 Beams Strengthened with External Steel Angles

Figure 5.19 shows the load-deflection relationship of beam strengthened with external steel angles (B5-1) attached with bottom stirrups by welding. The elastic limit of this beam was 18.8kN at mid-span deflection of 1.11mm. The relationship was again linear up to a load of 79.2kN while the mid-span deflection was 6.94mm. The ultimate load was 124.4kN at mid-span deflection of 39.20mm. The experimental ultimate load carrying capacity was 105% of the calculated load carrying capacity for that beam which was 118.2kN. The ductility index was 2.76. The beam was failed by crushing of compressive concrete after the yielding of steel bars and external steel angles. Total 10 new cracks (red) formed some of them were in the shear region and some in the flexural region. The length and width of the new flexural cracks are smaller than the preloaded cracks but the length of shear cracks was larger. The failure mode and crack pattern of beam strengthened with external steel angles (B5-1) are shown in Figure 5.20.

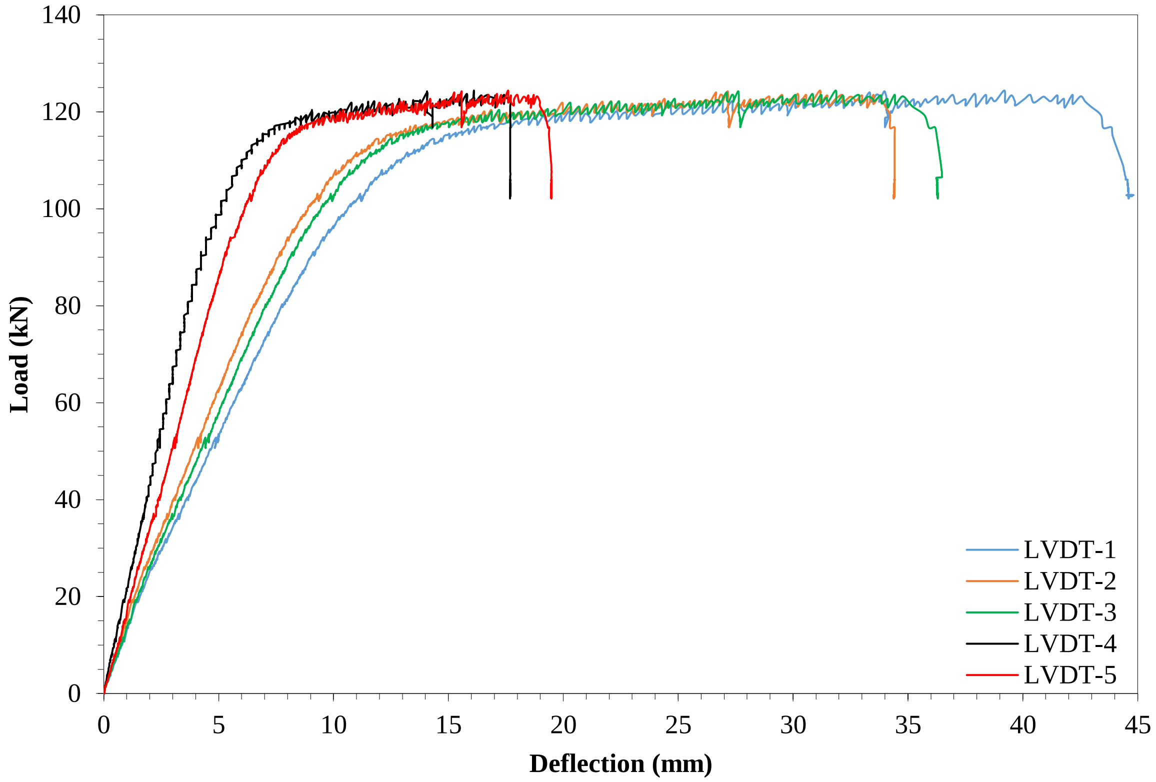

Figure 5.19: Load versus deflection of beam strengthened with steel angles (B5-1)

Figure 5.20: Failure mode of beam strengthened with steel angles (B5-1)

Figure 5.21 shows the flexural test results of beam strengthened with external steel angles (B5-2) attached with bottom stirrups by welding. The beam showed similar behavior as B5-1 in flexure up to a load of 110.4kN and mid-span deflection of 11.26mm. The ultimate load was 116.8kN at mid-span deflection of 14.76mm. The experimental ultimate load carrying capacity was 99% of the calculated load carrying capacity. B5-1 showed better ductile behavior than B5-2. The ductility index for B5-2 was 1.27 which was 2.76 for B5-1. The reduction of ductility may be due to more welding failure in case of B5-2. Total 7 new cracks (red) formed in the shear region. The bottom part which was cast after the attachment of steel angles tended to separate from original concrete which may be due to the failure of welding. The failure mode tended to switch from flexural to shear failure. Figure 5.22 shows the failure mode and crack pattern of beam strengthened with external steel angles (B5-2).

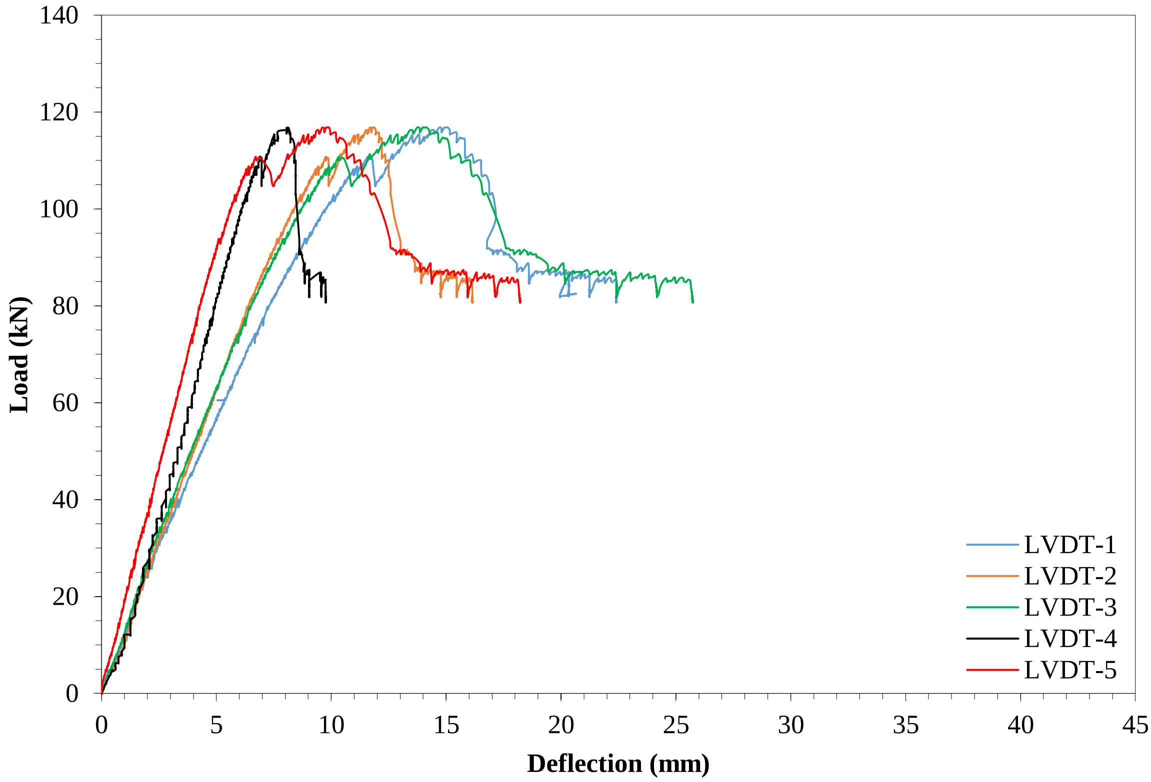

Figure 5.21: Load versus deflection of beams strengthened with steel angles (B5-2)

Figure 5.22: Failure mode of beam strengthened with steel angles (B5-2)

5.2.7 Beams Strengthened with External Steel Bars

The flexural test results of beam strengthened with external steel bars (B6-1) attached with bottom stirrups by welding are shown in Figure 5.23. The ultimate experimental load was 127.5kN at mid-span deflection of 17.22mm. The calculated load carrying capacity was 133.6kN. The beam showed elastic behavior up to a load of 91kN at a mid-span deflection of 9.12mm. The experimental ultimate load carrying capacity was 95% of the calculated load carrying capacity. The ductility index was 1.10. Although the steel bar is a ductile material the beam shows a brittle behavior in flexure test and failed in shear. Total 9 new cracks (red) formed in the shear region. The length and width of new cracks were larger than preloaded cracks (black). The bottom part which was cast after the strengthening of the beam by attaching external steel bars by welding tended to separate from the original concrete of the beam. Figure 5.24 shows the failure mode and crack pattern of the beam (B6-1).

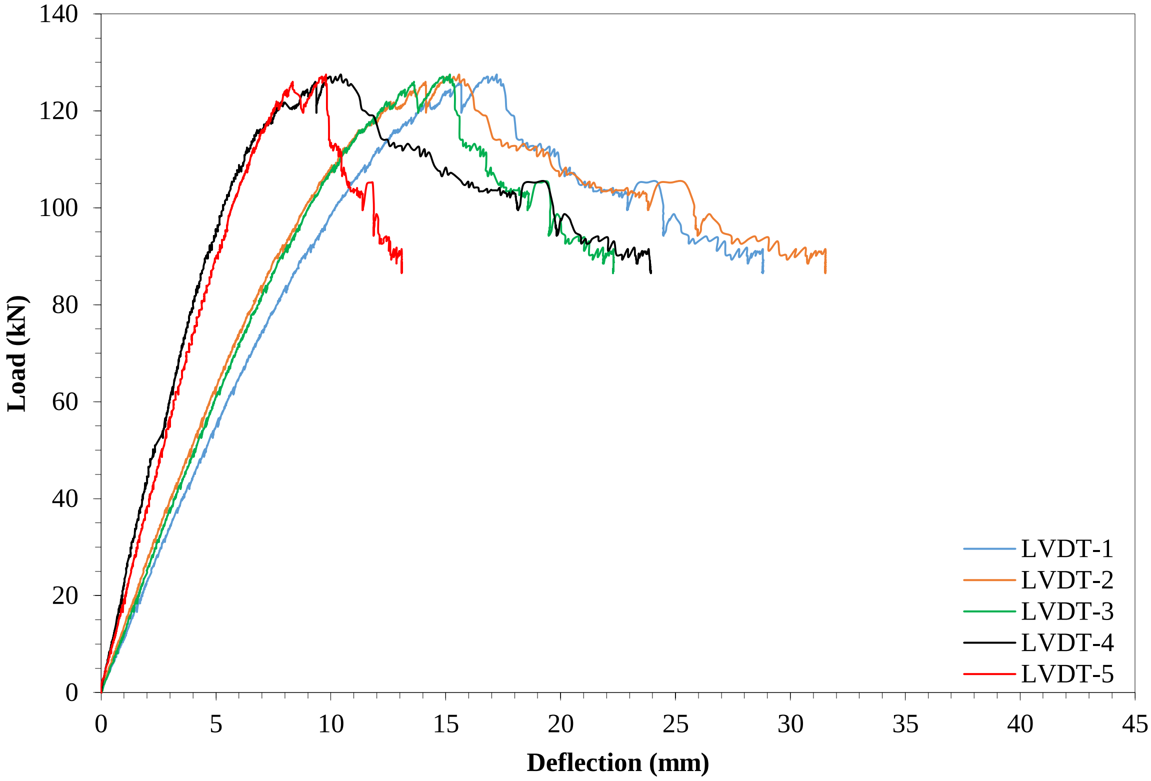

Figure 5.23: Load versus deflection of beam strengthened with steel bars (B6-1)

Figure 5.24: Failure mode of beam strengthened with steel bars (B6-1)

The load versus deflection curves of beam strengthened with external steel bars (B6-2) attached with bottom stirrups by welding are shown in Figure 5.25. B6-2 showed a similar behavior like B6-1. The ultimate experimental load was 134.0kN at mid-span deflection of 12.44mm. The calculated load carrying capacity was 133.6kN. The experimental ultimate load carrying capacity was almost same as the calculated load carrying capacity. The beam showed elastic behavior up to a load of 105kN at a mid-span mid-span deflection of 7.66mm. The ductility index was 1.00. Although the steel bar is a ductile material the beam shows a brittle behavior in flexure test and failed in shear. The crack pattern and failure mode of the beam strengthened with external steel bars (B6-2) are shown in Figure 5.26. Total 9 new cracks (red) formed. Most of the new cracks were shear cracks. The bottom part which was cast after attaching steel bars with bottom stirrups by welding tended to separate from the original concrete of the beam.

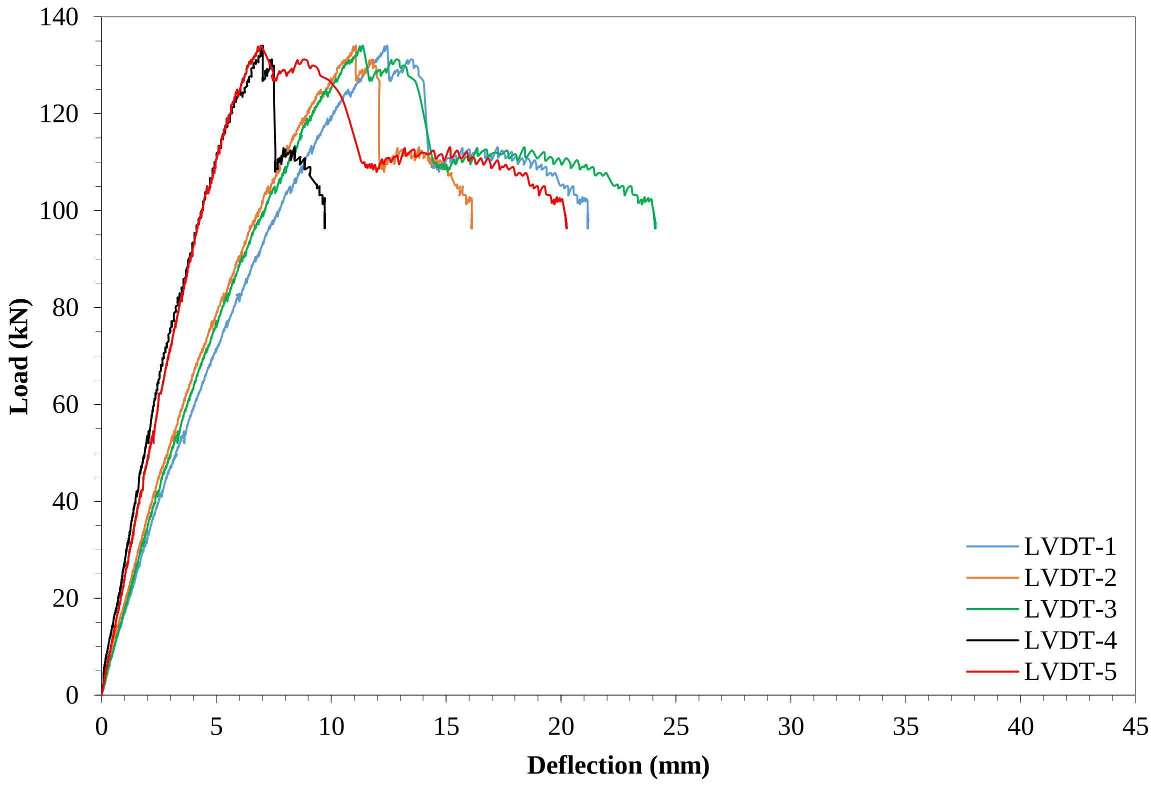

Figure 5.25: Load versus deflection of beam strengthened with steel bars (B6-2)

Figure 5.26: Failure mode of beam strengthened with steel bars (B6-2)

5.3 Comparison of Results

The load-deflection relationship of control beams (B1-1 & B1-2) are shown in figure 5.27. The yield and ultimate load and ductility index of two beams were very close. The ultimate load for B1-1 was 47.9kN which was 50.4kN for B1-2 this value was 11% and 17% higher than the calculated ultimate load. The average load bearing capacity of control beams was 49.1kN. Two beams showed similar behavior in flexure. The ductility index was 6.59 and 5.41 for the control beams B1-1 and B1-2 respectively.

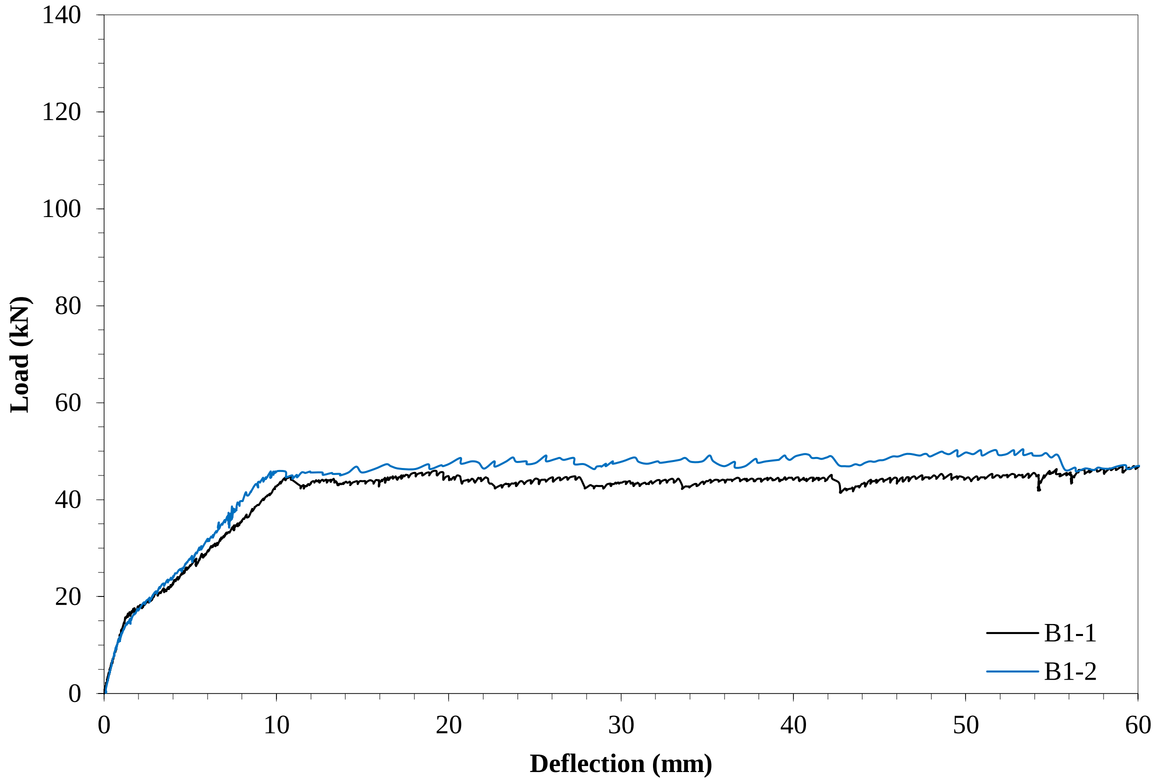

Figure 5.27: Load versus mid-span deflection of control beams

The load versus mid-span deflection of beams strengthened with external steel plate (B2-1, B2-2, B3-1 & B3-2) bonded with epoxy adhesives are shown in figure 5.28. Epoxy-1 was used as bonding agent for B2-1 & B3-1 and Epoxy-2 was used as bonding agent for B2-2 & B3-2. Steel bolts were used for anchorage of steel plate for B3-1 & B3-2. In case of steel plate, the ultimate capacity depended on the bonding agent. The ultimate capacity increased from 59.5kN to 92.4kN depending on types on adhesive for B2-1 to B2-2. The capacity increased 90.8kN to 100.4kN depending bonding agent in case of steel bolts anchorage. By anchoring with steel bolts, the ultimate capacity increased 59.5kN to 90.8kN and 92.4kN to 102.4kN depending on adhesive types. All four beams were failed by the separation of steel plate from the bottom surface of beams. The ultimate load for B2-1, B2-2, B3-1 and B3-2 was 59.5kN, 92.4kN, 90.8kN and 100.4kN respectively which was 21%, 88%, 85% and 104% higher than the ultimate capacity of control beams. Those values were 57%, 88%, 86% and 96% of the expected values after strengthening. Although the ductility was reduced after strengthening the capacity increased at a significant rate. The ductility index was 1 for B2-1, B2-2 and B3-1 and 4.12 for B3-2 which was less than the ductility index of control beams which was 6.59 and 5.41 for B1-1 and B1-2 respectively.

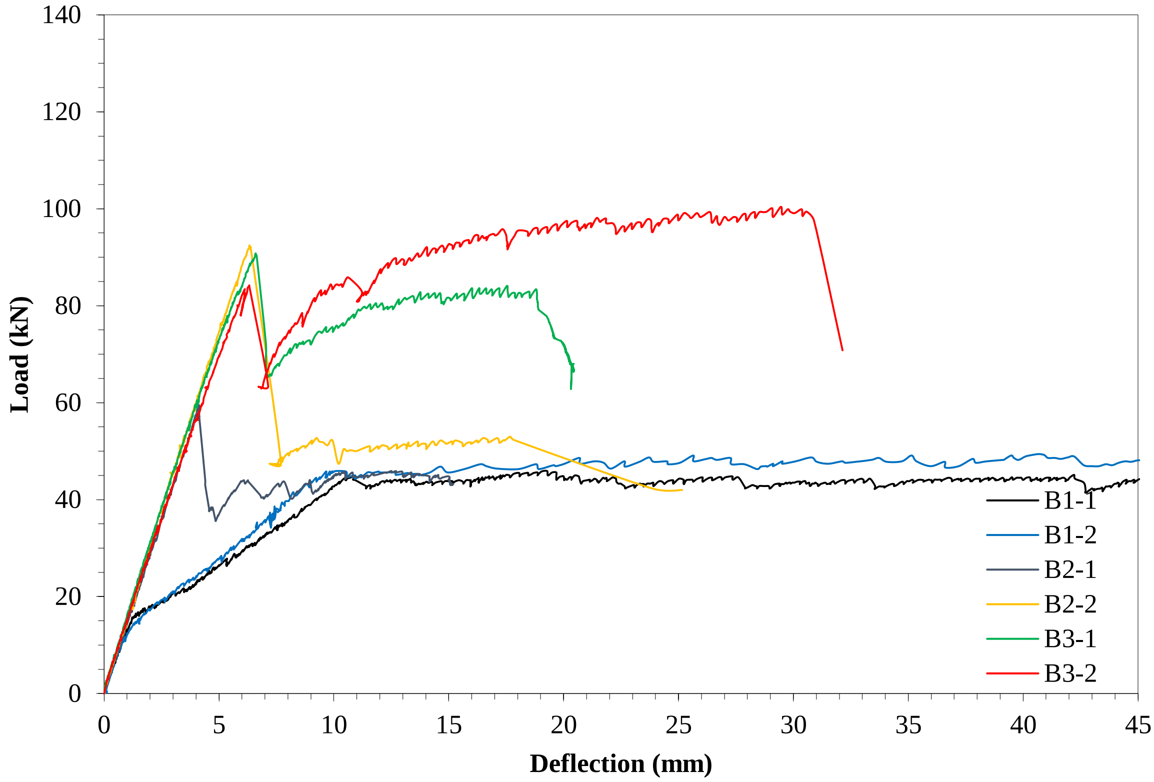

Figure 5.28: Load versus mid-span deflection of beams strengthened with steel plate

The load versus mid-span deflection of beams strengthened with near surface mounted steel bars (B4-1 & B4-2) bonded with epoxy adhesives are shown in figure 5.29. Epoxy-1 was used as bonding agent for B4-1 and Epoxy-2 was used as bonding agent for B4-2. The ultimate load for B4-1 and B4-2 was 101.9kN and 115.0kN which was 108% and 134% higher than the ultimate capacity of control beams. The ultimate capacity for B4-1 and B4-2 was 95% and 107% of the calculated value for the strengthened beams. The beam B4-1 was failed by the separation of steel bars from the groove of the surface of the beam but B4-2 failed by yielding of main steel bars and NSM steel bars. The ductility index was 1 and 1.99 for B4-1 and B4-2 respectively.

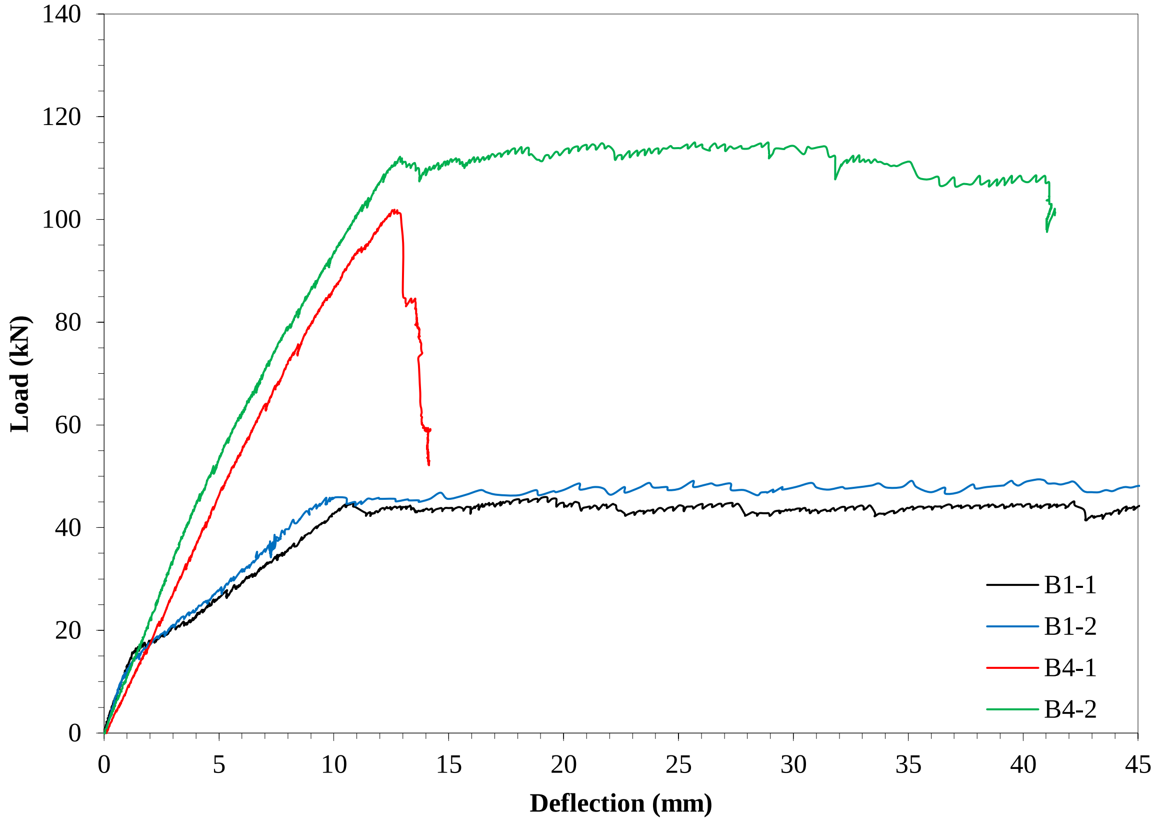

Figure 5.29: Load versus mid-span deflection of beams strengthened with NSM bars

The load versus mid-span deflection of beams strengthened with steel angles attached with bottom stirrups by welding (B5-1 & B5-2) are shown in Figure 5.30. The ultimate load for B5-1 and B5-2 was 124.4kN and 116.8kN which was 153% and 138% higher than the ultimate capacity of control beams. The ultimate capacity for B5-1 and B5-2 was 105% and 99% of the calculated value for the strengthened beams. The beams were failed by the yielding of main steel bars and external steel angles. The ductility index was 2.76 and 1.0 for B5-1 and B5-2 respectively which was 6.59 and 5.41 for the control beams.

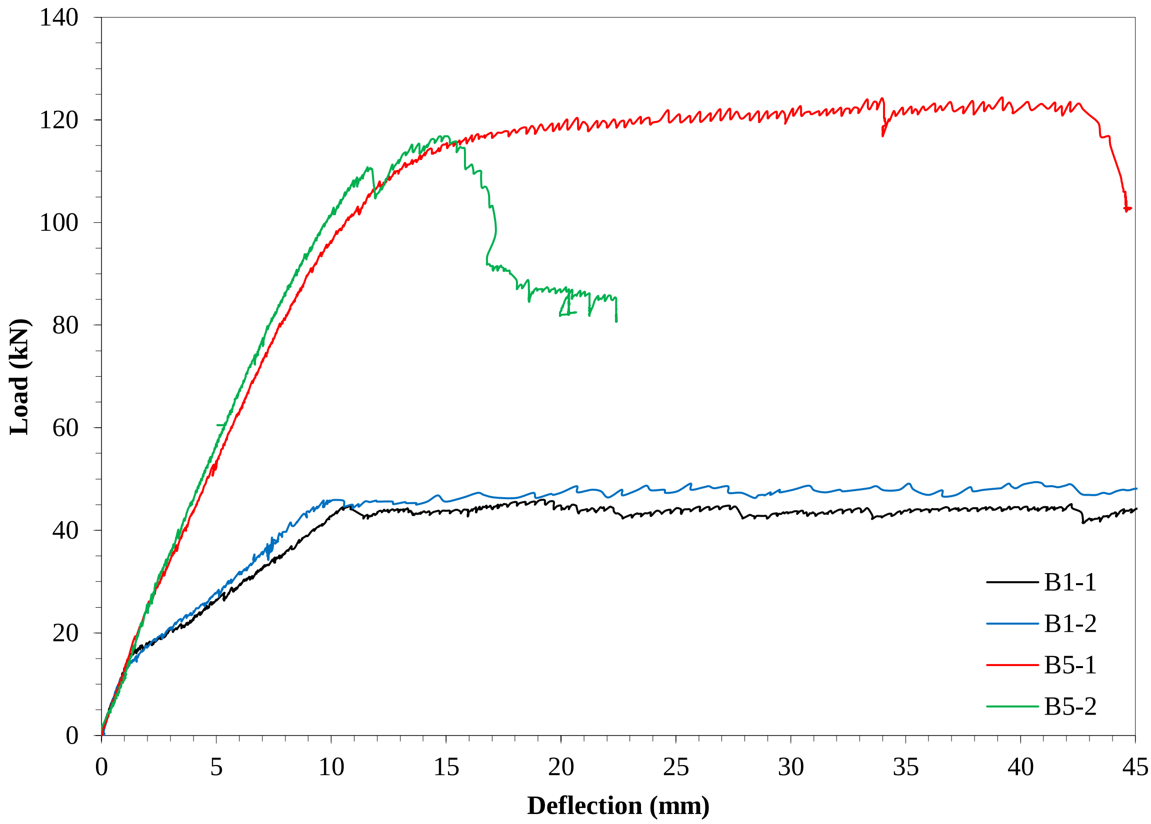

Figure 5.30: Load versus mid-span deflection of beams strengthened with steel angles

The load versus mid-span deflection of beams strengthened with external steel bars attached with bottom stirrups by welding (B6-1 & B6-2) are shown in Figure 5.31. The ultimate load for B6-1 and B6-2 was 127.5kN and 134.0kN which was 160% and 173% higher than the ultimate capacity of control beams. The ultimate capacity for B6-1 and B6-2 was 95% and 100% of the calculated value for the strengthened beams. The beams failed in shear. Manik, Halder, Paul & Rahman (2013) investigate the effect of welding on tensile properties of mild steel and found a reduction of 12% of ultimate strength. This may be the possible reason to change the behavior of beams in flexure. Another possible reason to show brittle behavior and failed in shear may be due to the failure of welding joint between the stirrups and external steel bars. It may be also due to the reduction of shear reinforcement area at the time of welding. The ductility reduced after strengthening. The ductility index was 1.10 and 1.00 for B6-1 and B6-2 respectively which was 6.59 and 5.41 for the control specimens.

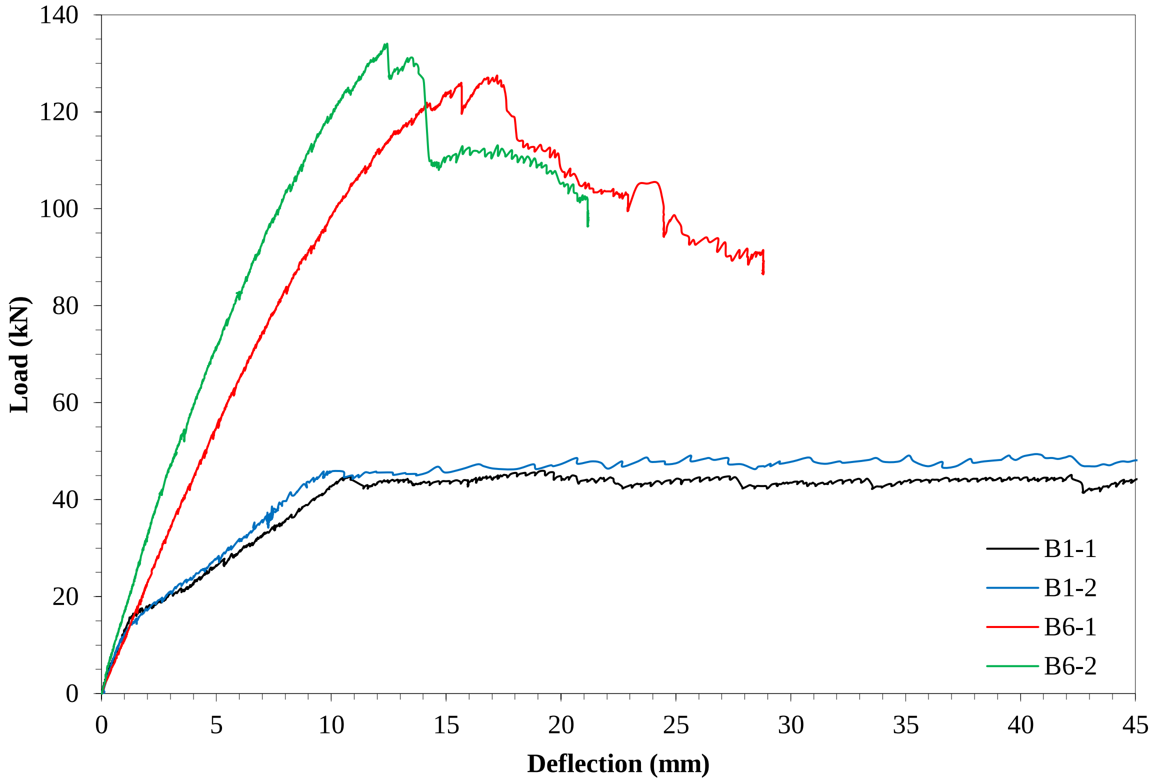

Figure 5.31: Load vs mid-span deflection of beams strengthened with steel bars

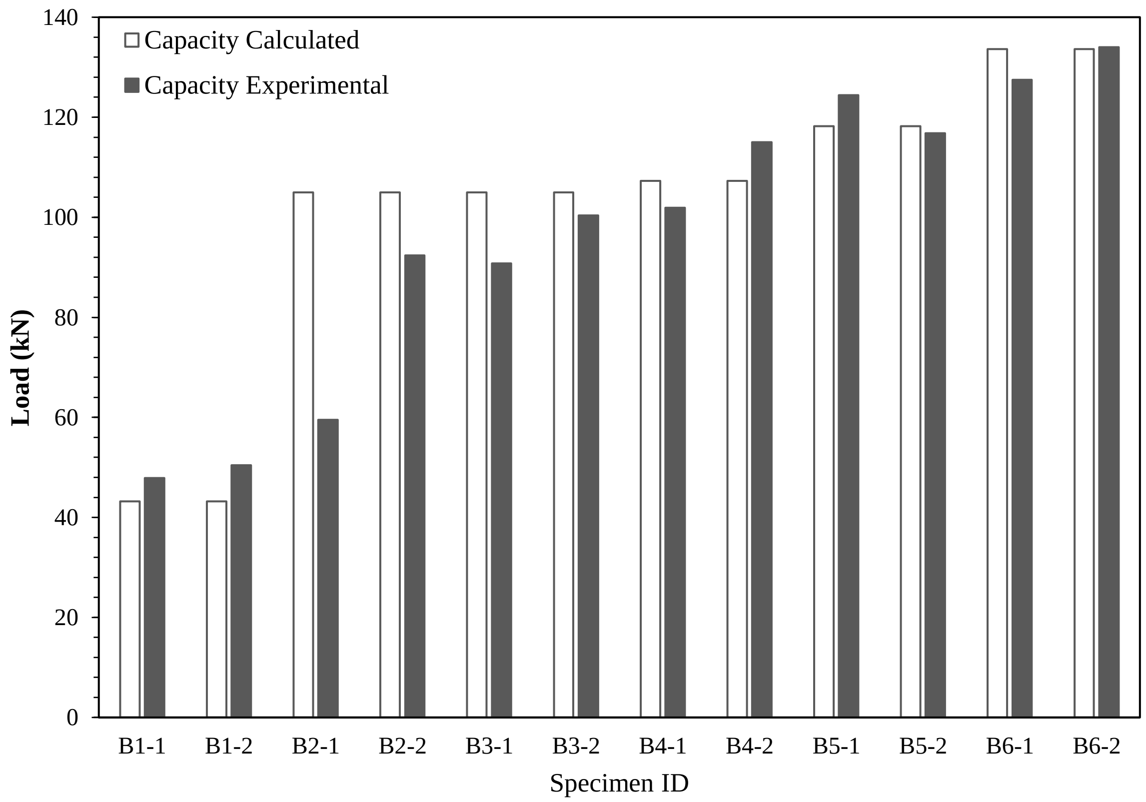

Figure 5.32: Relationship between calculated and experimental capacity

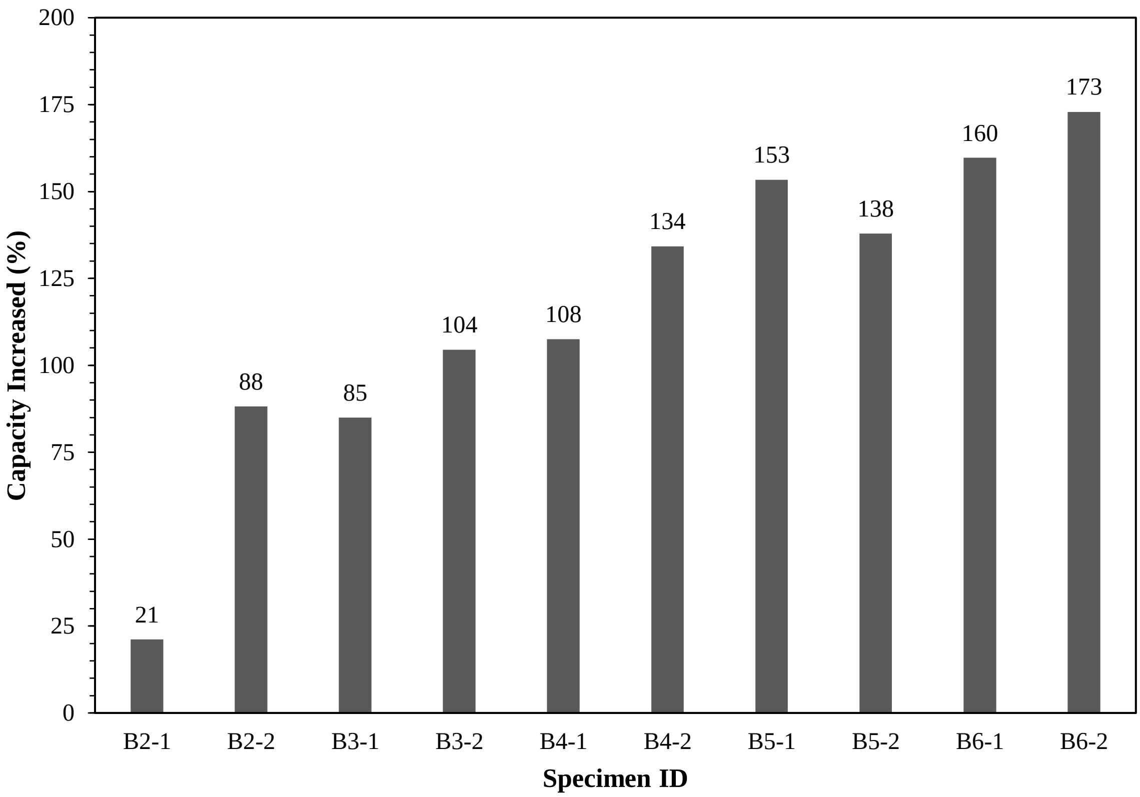

Figure 5.33: Capacity increased of strengthened beams

The relationship between the calculated and experimental load bearing capacity is shown in Figure 5.32. The experimental capacity of all beams (except B2-1, B2-2 & B3-1) were very close to expected calculated results. Figure 5.33 shows the capacity increased of the strengthened beams. By strengthening the capacity was increased up to 173%. Figure 5.34 shows the experimental capacity of all beams with respect to expected results in percentage. The experimental results were 111% & 117% for control beams. The capacity of strengthened beams (except B2-1, B2-2 & B3-1) were 95% to 107%. Figure 5.35 shows the comparison of load versus mid-span deflection of all six groups. The stiffness of all strengthened beams was slightly higher than the control beams. The stiffness of all strengthened beams was almost similar except the 4th and 5th groups (NSM bar & steel angle). The stiffness of 4th and 5th groups was slightly lower than the other four groups. Figure 5.36 and Figure 5.37 show the normalize results depending on the external steel ratio and the external steel ratio & grade of steel. The NSM technique was more effective than other methods in respect of the external steel ratio and grade of steel.

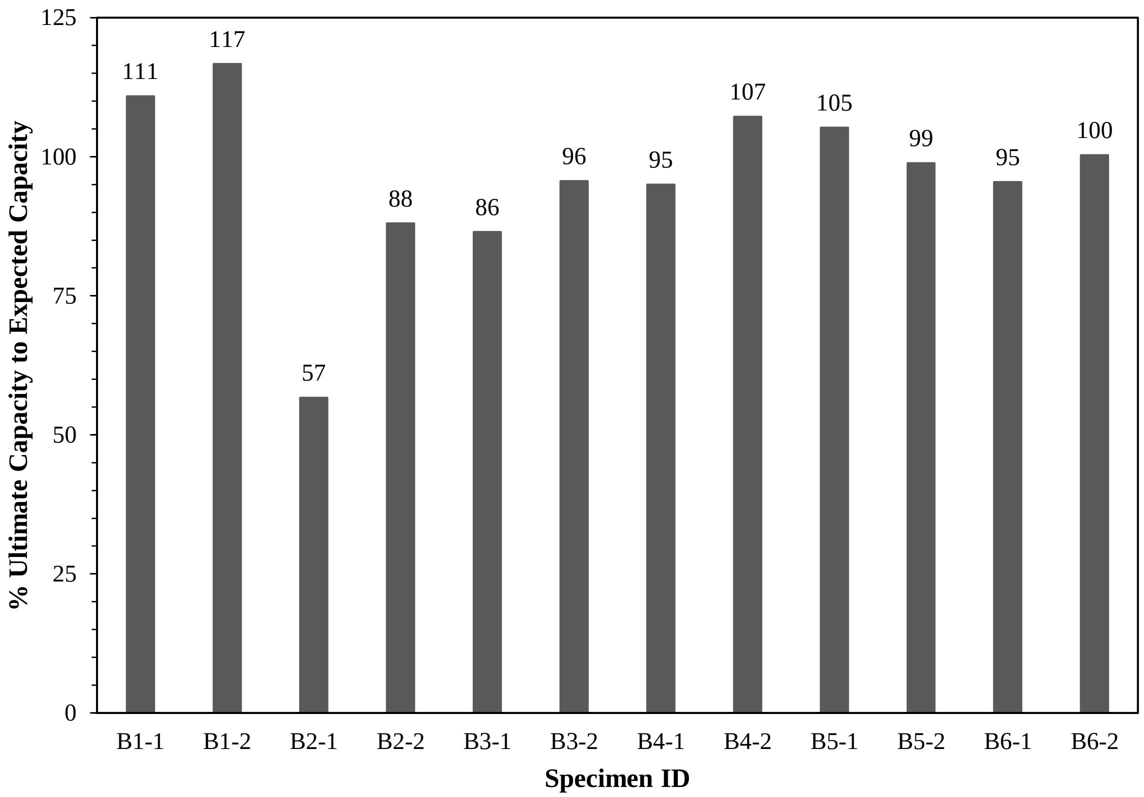

Figure 5.34: Experimental capacity of beams with calculated capacity

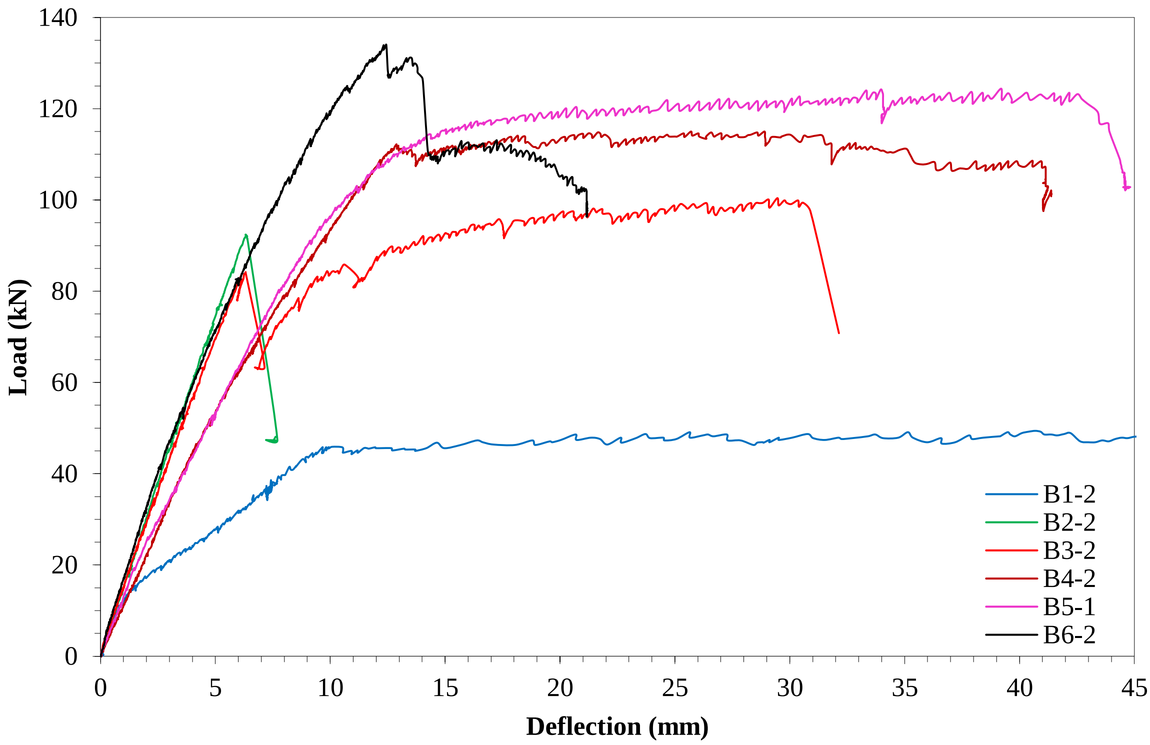

Figure 5.35: Comparison of load versus mid-span deflection of all six groups

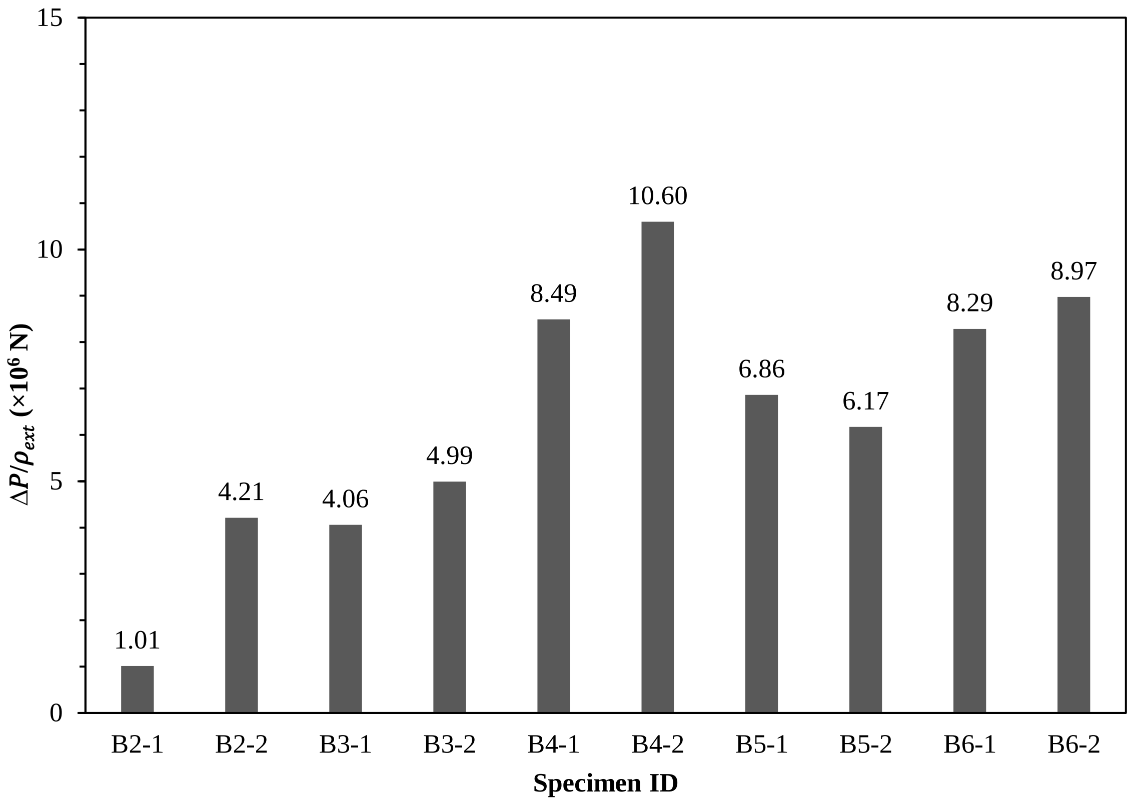

Figure 5.36: Normalize results depending on external steel ratio

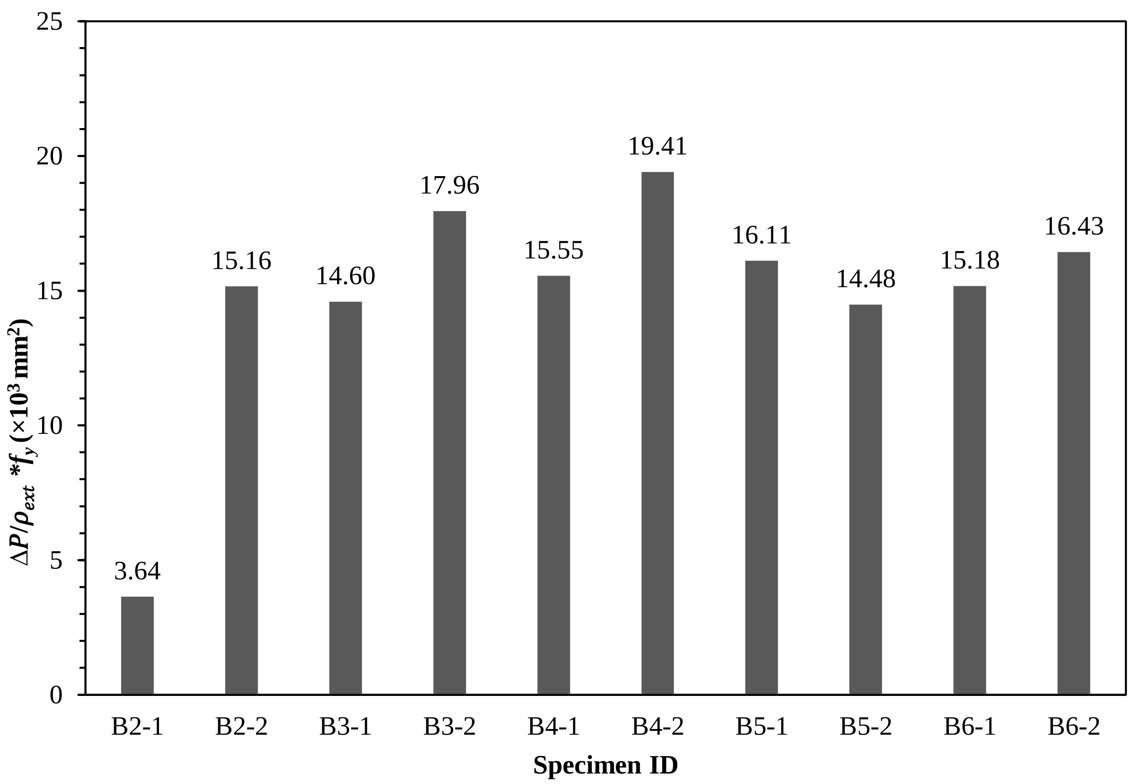

Figure 5.37: Normalize results depending on external steel ratio and steel grade

CHAPTER VI

CONCLUSION & RECOMMENDATION

6.1 General

This thesis work aimed to study and compare the effect of different selective strengthening techniques applied on simply supported beams. To achieve this goal, all the necessary tasks were completed. On the basis of the results obtained from the tests stated in the previous chapter, this chapter described concluding remarks and recommendations.

6.2 Conclusions

Based on the results of the study, the following conclusions were drawn.

- In case of strengthening with steel plate, all beams were failed by separation of steel plate. By strengthening with Epoxy-2 and anchorage by bolts and without anchorage was shown similar behavior. The ultimate load was increased by 104% and 88% for using anchorage and without anchorage respectively which was 96% and 88% of the calculated load carrying capacity. In the case of strengthening with Epoxy-1, the ultimate load was increased by 85% for using anchorage and only 21% without using anchorage. The ultimate load was 90% of the calculated ultimate capacity for using anchorage but only 57% without using anchorage.

- The action of NSM bars was mainly depending on the bonding between adhesive to steel bars and adhesive to concrete. The beam strengthened with NSM bars bonded with Epoxy-2 provided ultimate load of 115kN which was 134% higher than control beams and 107% of the calculated ultimate load. The ultimate load was 101.9kN for the beam strengthened with NSM bars bonded with Epoxy-1 which was 108% higher than the control beams. Although the ultimate load was 95% of the calculated ultimate load but failed by separation of NSM bars.

- The contribution of external steel angles was almost similar to external steel bars. It provided ultimate load of 124.4kN and 116.8kN and that was 153% and 138% higher than original beams. In the case of external steel angles, the ultimate load was 105% and 99% of the calculated ultimate load.

- The flexural strength of simply supported RC beams was greatly enhanced by the addition of external steel bars as secondary reinforcement. It provided ultimate load of 127.5kN and 134kN to a beam that already got cracks under the action of preloading. And this increased capacity of original beams by 160% and 173%. The ultimate load was 95% and 100% of the calculated ultimate load.

6.3 Recommendations

The following suggestions are recommended on the basis of the results:

- Need necessary care for the application of external steel bars and external steel angles for strengthening existing beams. Care required at the time of removal of the concrete cover and at the time of welding of external steel with stirrups.
- NSM bars strengthening technique can be a good solution for the purpose of strengthening with available materials. Further investigation is required about fire resistant of epoxy adhesive. Adequate bonding agent required for using NSM bars technique.
- As the ultimate load was 95% to 107% (except steel plate) of the calculated ultimate capacity a strength reduction factor (such as 0.95 or 0.90) can be used for the design of strengthened beams for the safety issue.
- The efficiency of strengthening by using steel plate bonded with epoxy adhesive depends on bonding properties of epoxy with steel plate. For this reason, sufficient bond strength of adhesive required for this technique.

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Appendix A

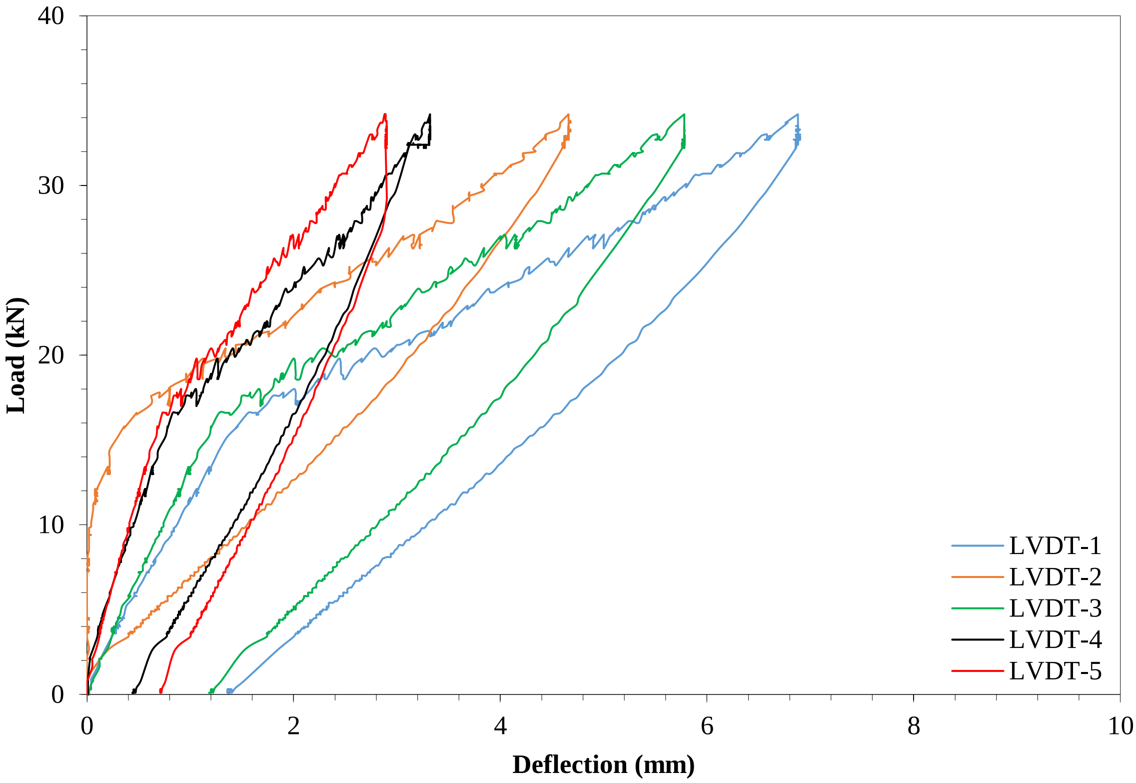

Figure A-1: Load versus deflection of beam for preloading (B2-1)

Figure A-2: Crack pattern for preloading (B2-1)

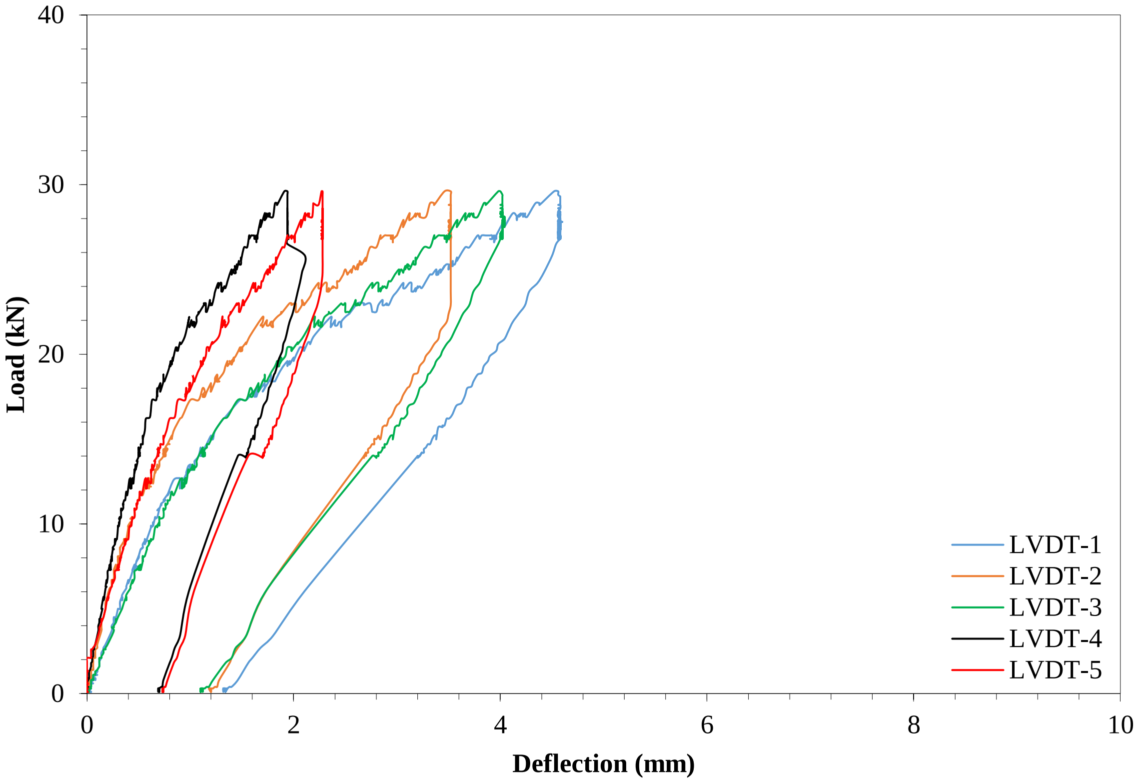

Figure A-3: Load versus deflection of beam for preloading (B2-2)

Figure A-4: Crack pattern for preloading (B2-2)

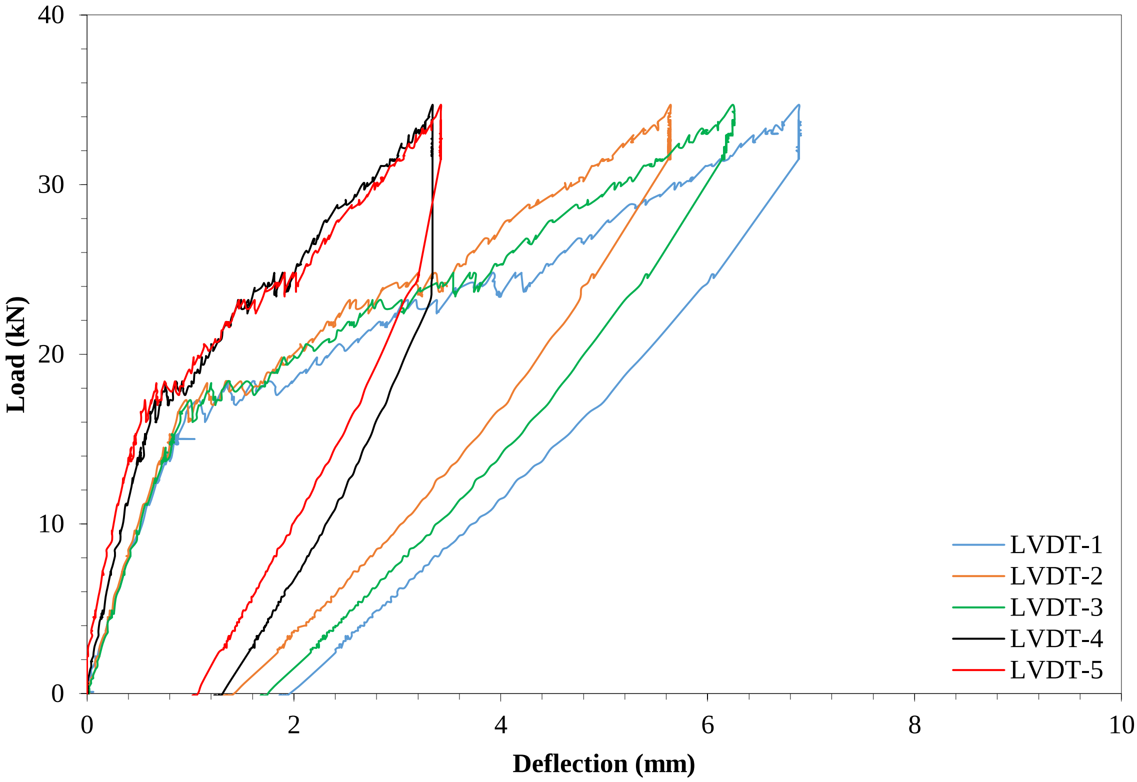

Figure A-5: Load versus deflection of beam for preloading (B3-1)

Figure A-6: Crack pattern for preloading (B3-1)

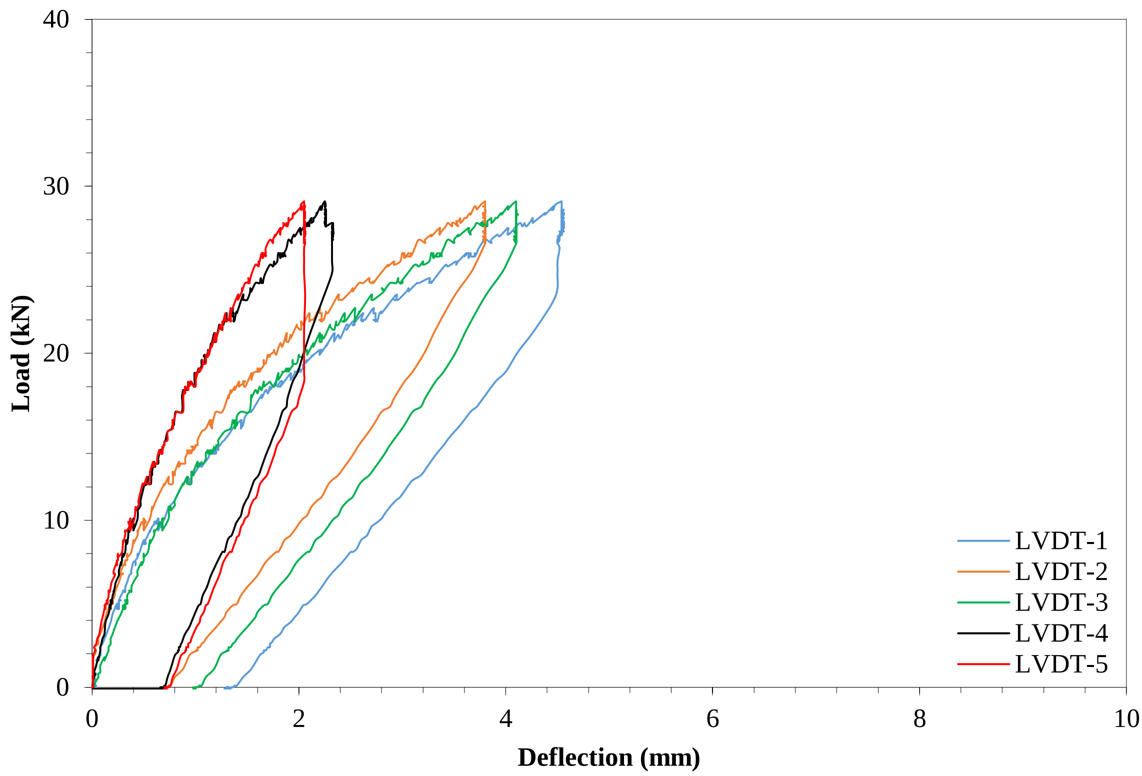

Figure A-7: Load versus deflection of beam for preloading (B3-2)

Figure A-8: Crack pattern for preloading (B3-2)

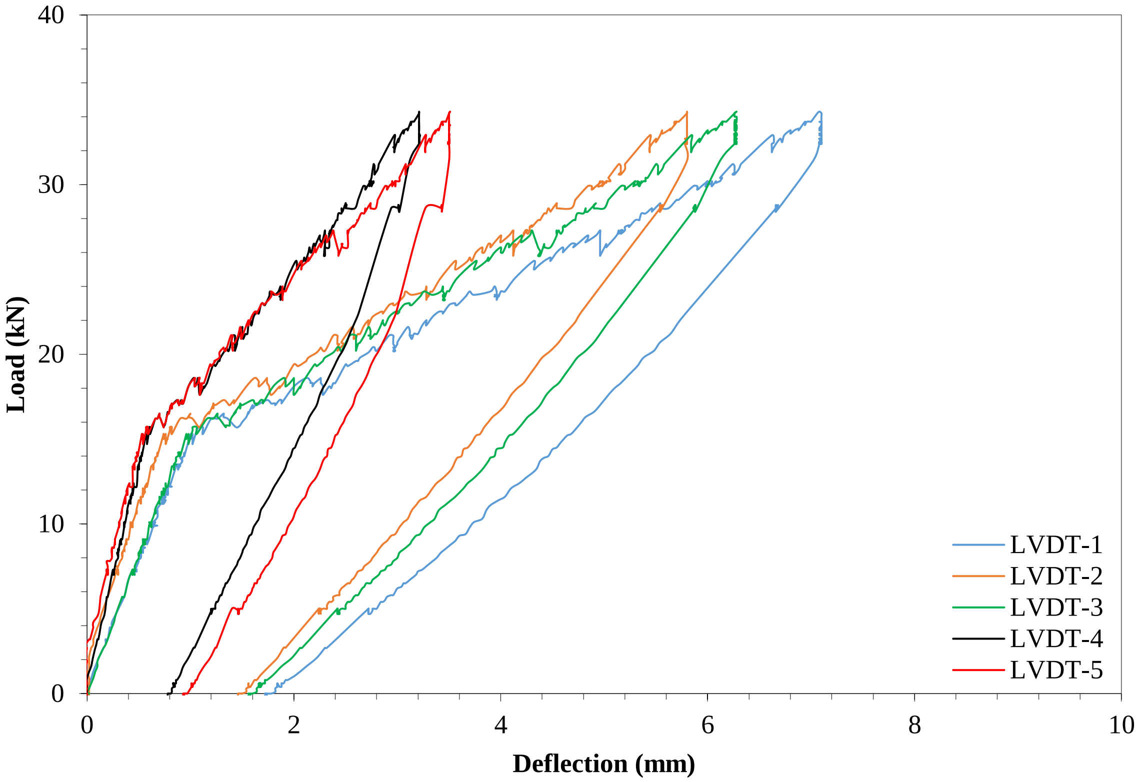

Figure A-9: Load versus deflection of beam for preloading (B4-1)

Figure A-10: Crack pattern for preloading (B4-1)

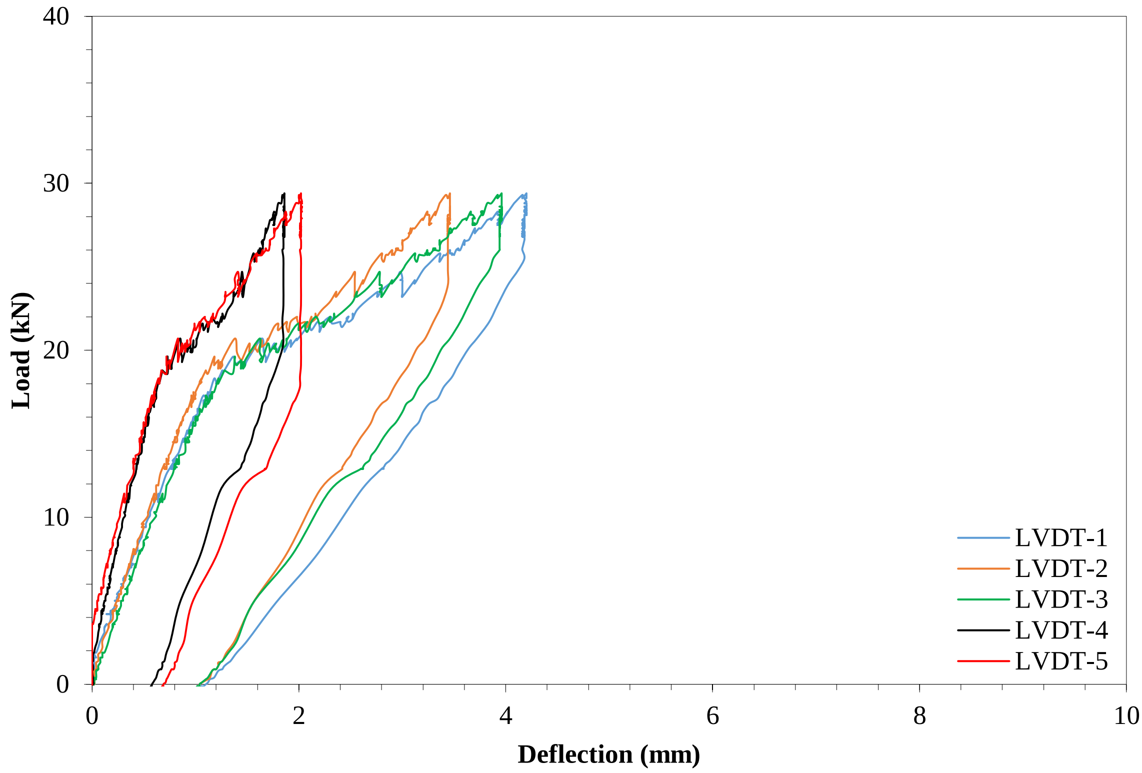

Figure A-11: Load versus deflection of beam for preloading (B4-2)

Figure A-12: Crack pattern for preloading (B4-2)

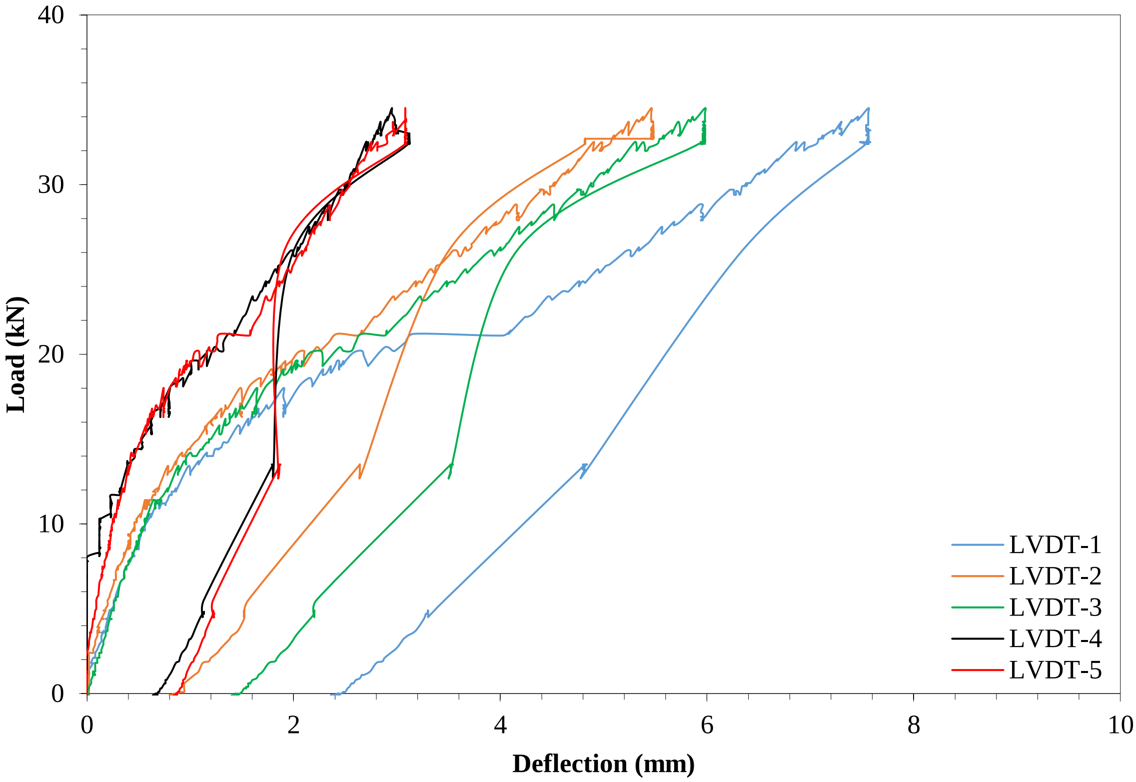

Figure A-13: Load versus deflection of beam for preloading (B5-1)

Figure A-14: Crack pattern for preloading (B5-1)

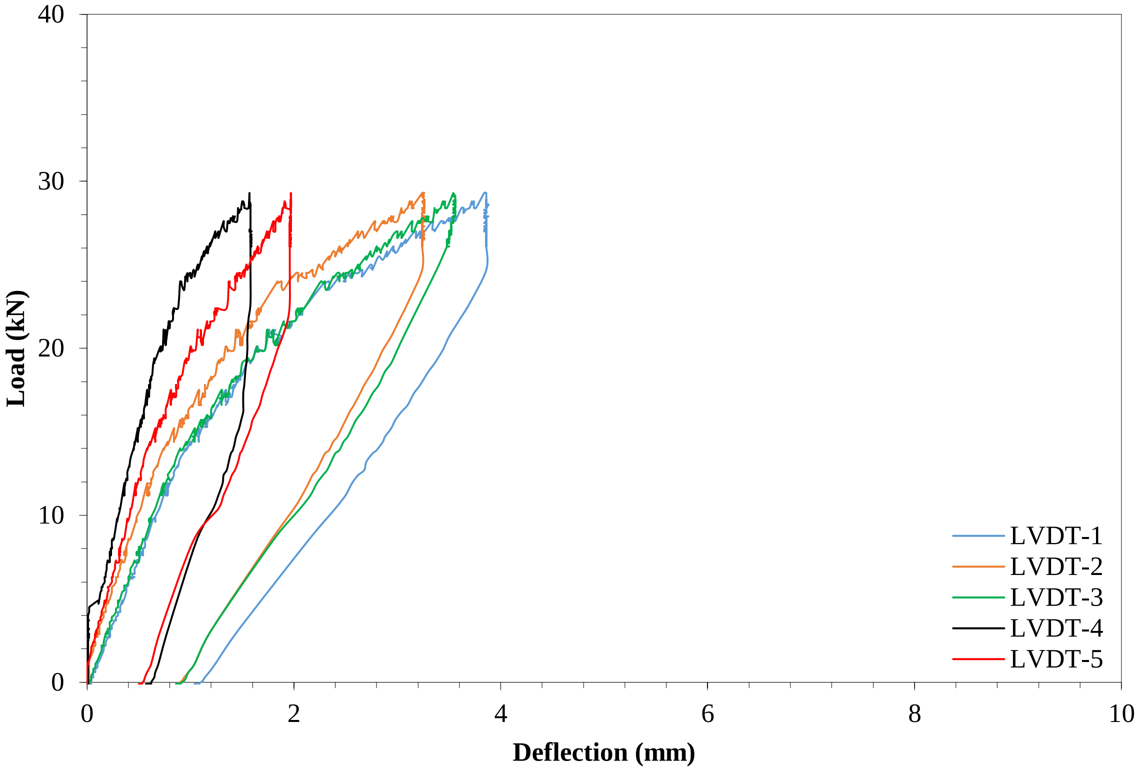

Figure A-15: Load versus deflection of beam for preloading (B5-2)

Figure A-16: Crack pattern for preloading (B5-2)

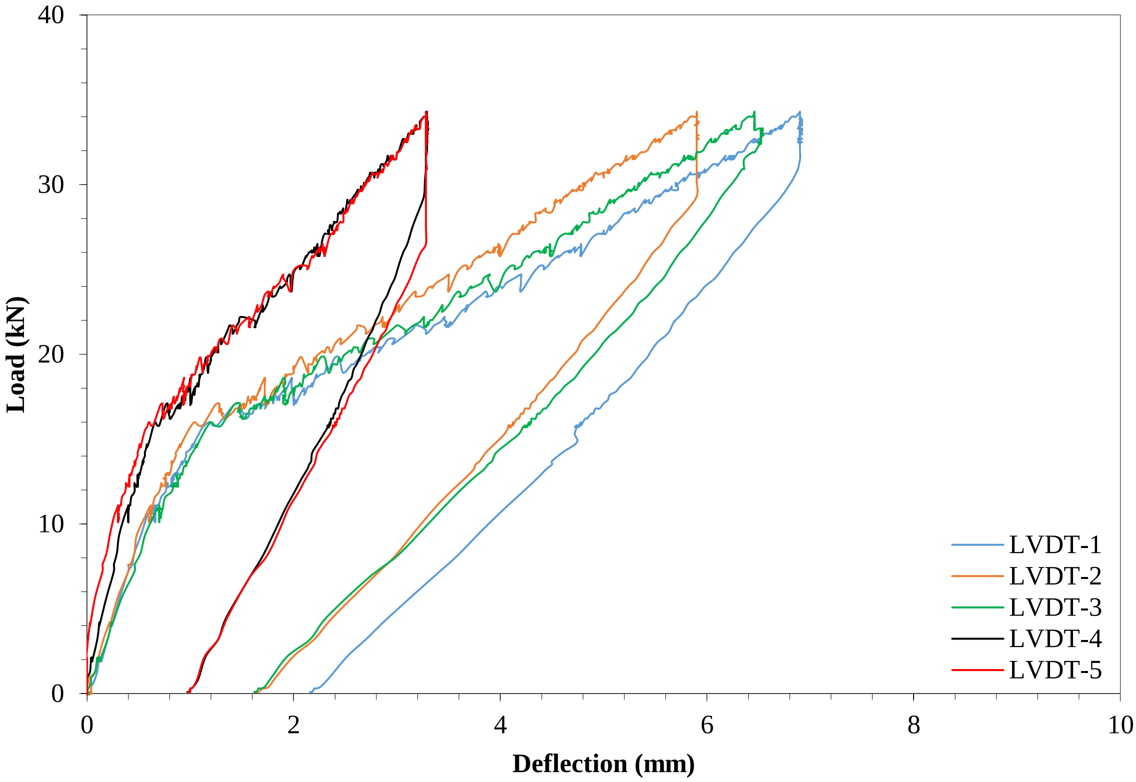

Figure A-17: Load versus deflection of beam for preloading (B6-1)

Figure A-18: Crack pattern for preloading (B6-1)

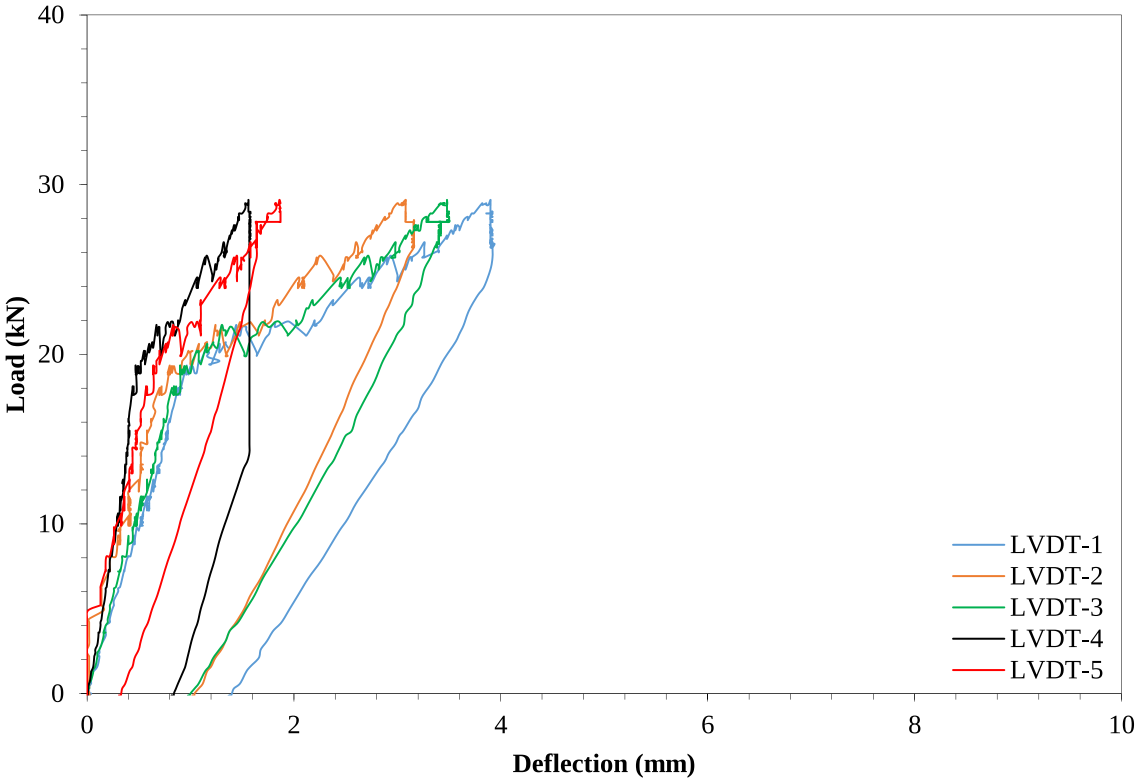

Figure A-19: Load versus deflection of beam for preloading (B6-2)

Figure A-20: Crack pattern for preloading (B6-2)