MAJOR PROJECT-II REPORT

ON

RETROFITTING OF CORROSION-EFFECTED STEEL BRIDGE MEMBERS USING STEEL - CFRP BONDING SYSTEMS

Submitted in partial fulfillment of the requirements for the award of the Master of Technology in Civil Engineering with specialization in
STRUCTURAL ENGINEERING

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CANDIDATE'S DECLARATION

This is to certify that the major project-I report entitled **Retrofitting Of Corrosion-Effected Steel Bridge Members Using Steel - CFRP bonding systems** being submitted by me is a bonafide record of my own work carried by me under the guidance of **Dr. Awadhesh Kumar, Associate Professor** in the partial fulfillment of the requirement for the award of the degree of Master of Technology in Civil Engineering with specialization in **STRUCTURAL ENGINEERING**, **DELHI TECHNOLOGICAL UNIVERSITY** (Formerly Delhi College of Engineering) DELHI- 110042.

The matter embodied in this project has not been submitted for the award of any other degree.

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CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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With all humility I thank him for his valuable guidance at each and every stage of this Report for its successful completion. I consider myself fortunate for having the opportunity to learn and work under his supervision and guidance over the entire period of association.

Rupali Vij
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Symbols and notations

FRP	Fibre Reinforced Polymers
CFRP	Carbon Fibre Reinforced Polymers
GFRP	Glass Fibre reinforced Polymers
Tg	Glass Transition Temperature
HDT	Heat Deflection Temperature
τ_{max}	Maximum Local Bond Shear Stress
δ_1 , δ_2	Corresponding slips
δ_f	Ultimate slip
Gf	interfacial fracture energy
BR	Impact test optional; if required at room temperature; semi-killed/killed
a_o	Notch size
H	specimen height
UV	Ultra Violet
UC	Universal Column Section.

ABSTRACT:

Most steel bridges and structures often need strengthening or retrofitting after a set period of time either due to damage/ loss of cross-section caused by prolonged exposure to severe corrosive environments or due to a need of upgradation because of increased live loads in due period of time or due to changes in the kind use of structure or to improve fatigue performance by reducing stress level for a given loading condition.

The current techniques of retrofit of steel structures available have several short comings including requirement of heavy equipments for installation and skilled labour. Thus, retrofitting/strengthening of structures using advanced composites have become a very popular method in the past couple of decades. Recent research has emphasized on strengthening and rehabilitation of steel structures and bridges using Carbon Fibre Reinforced Polymer (CFRP) materials. The technique of retrofitting with Carbon Fibre Reinforced Polymer (CFRP) has drawn growing attention in research field and in practice and also its environmental durability is of high importance. This report majorly encompasses the use of adhesively bonded Carbon Fibre Reinforced Polymers (CFRP) sheets in retrofitting steel bridge members affected by corrosion.

While the use of fibre reinforced polymer sheets for the repair and strengthening of reinforced concrete structures is well recognised, research work on the relevance of FRP composites to steel structures has been limited. The use of FRP material for the repair and rehabilitation of steel members has abundant benefits over the traditional methods of retrofit by bolting or welding of steel plates. Carbon FRP'S (CFRP'S) have been preferred over other FRP material for strengthening of steel structures since CFRP'S tend to posses higher stiffness.

The emergence of high modulus CFRP plates, with an elastic modulus higher than/comparable to that of steel, enables researchers to achieve considerable load transfer in steel beams prior to the yielding of steel. In such CFRP strengthened structures, the behaviour of CFRP-to-parent material bonded joints play a very important role. Existing work on CFRP-to-steel bonded joints is very limited. In a CFRP-to-steel bonded joint, the weak link has been found to be the epoxy adhesive, while in a CFRP-to-concrete bonded joint, the concrete is the weak link. Thus it can be said that the bond behaviour of CFRP materials to steel structures is quite unparalleled from that of concrete structures. Preliminary test results confirmed indispensable amount of very high bond stresses for most strengthening applications due to the tune of strengthening required for steel structures and bridges. Bond stresses might be much more critical for steel structures/members than for concrete structures since more strengthening material is required for steel structures to achieve a similar ascend in strength due to the high intrinsic strength of steel. There is, however, a concern in relation to possible galvanic corrosion when carbon and steel are bonded together.

In this report, surface preparation methods of steel and CFRP for effective bonding and means of preventing galvanic corrosion have been discussed. The results obtained from an experimental program verifying the efficiency of method of retrofitting the steel bridge members using CFRP-epoxy strengthening systems with or without end-anchorages have been presented. Also the effectiveness of the retrofitted members in resisting further corrosion in extreme exposures has been tested and the results comparison documented.

Chapter 1: INTRODUCTION

1.1 GENERAL

Bridges play a significant role in railway infrastructure throughout the world. With the introduction of trains with progressively heavier axle load and high horse power locomotives, bridges are being subjected to much greater loads than the original design loads considered. Indian Railways now have the herculean task of investigating and retrofitting aged bridges across the country. It is a great challenge considering Indian Railways has more than 36,000 bridges that are aged over 100 years old. [1]

Hence, the main challenge for bridge engineers of date is to assess the strength and capability of the existing bridges and retrofit them to enable them to suffice for the enhanced loading. The Railways has spent nearly Rs. 6,000 Crore on the repair of bridges in the past 10 years.[1] That amount would have to be multiplied many a times to render the kind of infrastructure that would enable the Railways to perform efficiently. In most cases it has been concluded that the cost of retrofit will be considerably lower than the cost of replacement. In addition, retrofitting comparably takes less construction time and greatly reduces service disruption time.

Existing old rail/road bridges are facing following kinds of problems:

- Damage/loss of cross-section caused by prolonged exposure to severe corrosive environments
- Lack of proper maintenance
- Damage due to accidents
- Aging and fatigue conditions
- Upgradation requirements for enhanced loading standards for axle load
- Need of stabilization for vibrating structures
- Design or construction defects like insufficient structural depth
- Reconsideration of meter gauge bridges for broad gauge (BG) use.

In this project the main focus will be on study of technique of retrofitting of corrosion affected steel members.

1.2 OBJECTIVE

1. To study the technique of retrofitting of corrosion-effected steel bridge members using Carbon Fibre Reinforced Polymer (CFRP) sheets, their applications, advantages and disadvantages over conventional retrofitting techniques.

2. To experimentally verify the effectiveness of the method of retrofitting of steel bridge members using CFRP strengthening systems with or without end anchorages.

3. To experimentally verify the effectiveness of the retrofitted members in resisting further corrosion in extreme exposures.

1.3 METHODOLOGY

Literature review was carried out for existing work in the field of retrofitting/strengthening and repair of steel members using CFRP and various aspects such as the structural behavior of the strengthened beams, bond and force transfer mechanism between steel and CFRP, modes of failures of CFRP-steel joints and the durability of retrofitted systems particularly providing solutions to eliminate galvanic corrosion have been summarized.

Secondly, an experimental program for the verification of the technique of strengthening corrosion-affected steel members is proposed. Following the chosen technique of retrofit nine beams were artificially corroded using a suitable method of corrosion and the corroded beams were retrofitted using CFRP-bonding systems with and without end-anchorages and the test beams thus prepared were tested under three-pointing best test arrangement.

Further, to experimentally verify the efficiency and durability of the method of retrofitting steel members using CFRP and epoxy resins under extreme exposures. The test beams were exposed to saline water and alternated drying-wetting cycles and tested similar to prior beams.

1.4 VARIOUS OTHER METHODS OF RETROFIT

1. **Retrofitting the steel bridges for fatigue considerations**:

   The causes of fatigue of steel bridges may be categorized as:
   a. Built-in welding defects incorporated at the time of fabrication.
   b. Assumed incorrect structural detail of low fatigue strength.
   c. Unexpected Deformations and Stresses in design occurred at element joints.
   d. The structure behaved in an unexpected manner, such as due to vibration.

   Repair Methods include:
   - Crack removal
   - Re-welding
   - Surface treatments such as TIG dressing
   - Post welding surface treatment in addition to re-welding
   - Provision of Bolted splice
   - F Shape improving methods
   - G-Stop hole
   - Connection detail modification
   - Strengthening by steel plate splicing

2. **Concrete covering [4]**: This method is same as shotcreting. Also Concrete covering (refer Fig. 1) helps to avoid the member buckling.

3. **Steel stitching [3]**: Steel stitching (refer Fig. 2) is a common technique for retrofitting of steel bridge girders. Some advantages of this method are listed here:
   a. Feasible at any location
   b. Minimal traffic interruption
   c. It is also economical and cost-effective technique

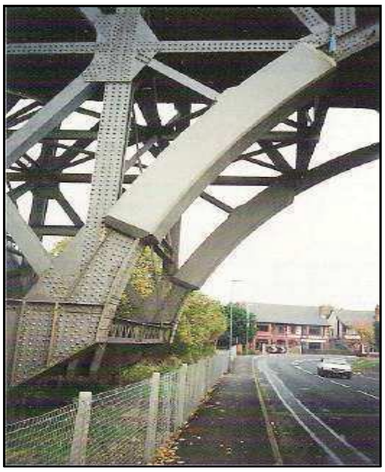

Figure 1: Concrete Covering Retrofit [4]

Figure 2: Steel Stitching [4]

Figure 3: Shear Links for Seismic Retrofit [3]

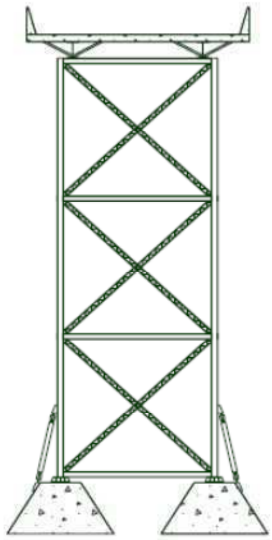

Figure 4: Unbonded Brace [3]

4. **Seismic retrofitting techniques [3]**

   Repair methods include:
   - Provision of shear links: Built-up shear links (refer Fig. 3) fabricated using plates of varied grades of steel.
   - Provision of unbounded brace (refer fig. 4) for controlled rocking approach to seismic resistance in steel truss bridge piers.
   - Seismic isolation methods.
   - Provision of lead rubber bearings.

5. **Steel Plate Bonding**

   In this method the deficiency in the structure is retrofitted by addition of steel plates of desired thickness as a splice to the existing member plate, flange or web as required in order to strengthen the member in shear, flexure or compression. The plates are either welded or bolted to the existing member. The addition of plate strengthens the member locally and also helps by enhancing the moment of inertia of the section as a whole. This method is the most common method of retrofitting but has some serious drawbacks as discussed in the next section.

1.5 Advantages of using CFRP Bonding System over Conventional Methods

Most existing methods of retrofit for steel bridges either make use of welding or bolting of steel plates or sections to the existing members. However, constructability and durability disadvantages are related with these methods. Welding additional steel plates have a major disadvantage of poor fatigue performance and also poor corrosion performance due to material difference in weld and parent material leading to increase in future maintenance costs. Wherein bolted plate retrofits are preferred because of rendering good fatigue performance to the member but it has disadvantages such as reduced cross section area due to drilling which in turn leads to requirement of additional strengthening material. Also the corrosion performances in bolted retrofits are also affected due to accumulation of debris and moisture around edges of bolted connections and plates thus added. Constructability disadvantages are requirement of heavy lifting machinery for steel plates, addition of considerable more dead weight to the structure, future maintenance costs, time consuming and cost ineffective.

Thus, the need for adopting cost-effective and durable materials retrofit techniques is quiet evident. One of the feasible alternatives is to use high performance, non-metallic materials such as fiber reinforced polymers (FRP). The intrinsic high strength and stiffness of steel makes it a more challenging material to strengthen, compared to concrete. When steel is retrofitted using a material having a lower Young's modulus, load transfer/sharing of the strengthening material will only be substantial after the steel has completely yielded. Therefore, materials like glass fiber reinforced polymer (GFRP) that have relatively low inherent low tensile modulus, are rendered less desirable for retrofitting steel structures. On the other hand, the superior physical and mechanical properties of CFRP materials make them a quite promising solution for repair and strengthening of steel structures. By using CFRP sheets global cost savings may be brought about through saving the labour costs, the minimised needs of handling and transporting equipments to place the reinforcement in position and the addition of insignificant dead weight to the steel structures. Regardless of the high CFRP costs, the overall costs of the strengthening project can be greatly optimised.[2]

The use of CFRP and GFRP bonding systems for retrofit of concrete structures has evidently been quiet successful. Its effectiveness has been verified for a variety of retrofitting mechanisms and is becoming more widely accepted practically. These are used in the form of plates or sheets bonded to the concrete surface for flexural and shear retrofitting or as sheets for wrapping columns to improve their ductility and axial strength. The use of CFRP on steel structure has also attracted great attention and is being studied but yet many aspects need consideration. The work carried out by various researchers in the field of retrofit using CFRP-bonding systems has been discussed in chapter 3.

Chapter 2: INTRODUCTION TO MATERIALS

2.1 Carbon Fibre Reinforced Polymer, CFRP

Fibre reinforced polymers, FRP's, are composite materials made of two components, fibres and resins. Continuous fibres are set in the polymeric resin matrix such that the resultant material has distinct non-reactive materials bonded together and possesses the combined properties of both the materials. Reinforcement (Fiber) is harder and stronger, and the resin matrix works as a shielding layer that holds the reinforcement collectively and helps transfer forces between them. Moreover, the mechanical and the material properties of the composite material are superior to the constituent materials separately. Different kind of FRP's available are Glass Fibre Reinforced Polymer (GFRP), Carbon Fibre Reinforced Polymer (CFRP), Aramid Fibre reinforced polymer (AFRP) and Basalt Fibre Reinforced Polymer (BFRP). The commonly used are resins are epoxy, polyester, phenolic, polyurethane and vinyl ester resins. FRP composites hence formed exhibit linear elastic stress–strain behaviour prior to brittle failure by rupture. This linear–elastic–brittle stress–strain behaviour has important inferences for the structural use of FRP composites in civil engineering applications.

In civil engineering applications, however, carbon fiber is found to be the dominating reinforcement, even though glass fibers have also been used in some applications. Carbon fibers are being preferred over other materials because of its good material properties such as strong and stable bond at the atomic level, higher rigidity, strength, resistance to many chemical aggressive environments, low density and availability. However, the material is brittle.

Advantages of using CFRP to strengthen steel structures are:

- High strength to weight ratio
- High stiffness (Comparable to steel) to weight ratio
- Corrosion resistance
- Can be bonded with ease using adhesives.
- Easy yet speedy transportation, handling and installation with reduced costs
- Least disruption to service during repair
- Retrofit contributes less additional dead weight
- Less stress concentration compared to mechanical fastenings
- Fatigue performance much better compared to welded cover-plates without introducing new residual stresses [33]
- Good creep properties
- Ability of CFRP's to follow curved and irregular surfaces
- Material properties can be tailored for particular applications and also can be varied in different directions.
- Useful in enhancing seismic resistance of steel tubes/shell sections by strengthening against local buckling
- No heat affected zones as in case of welding. Hence, CFRP bonding systems offers an ideal strength compensation method.

However, environmental durability and galvanic corrosion concerns exist.

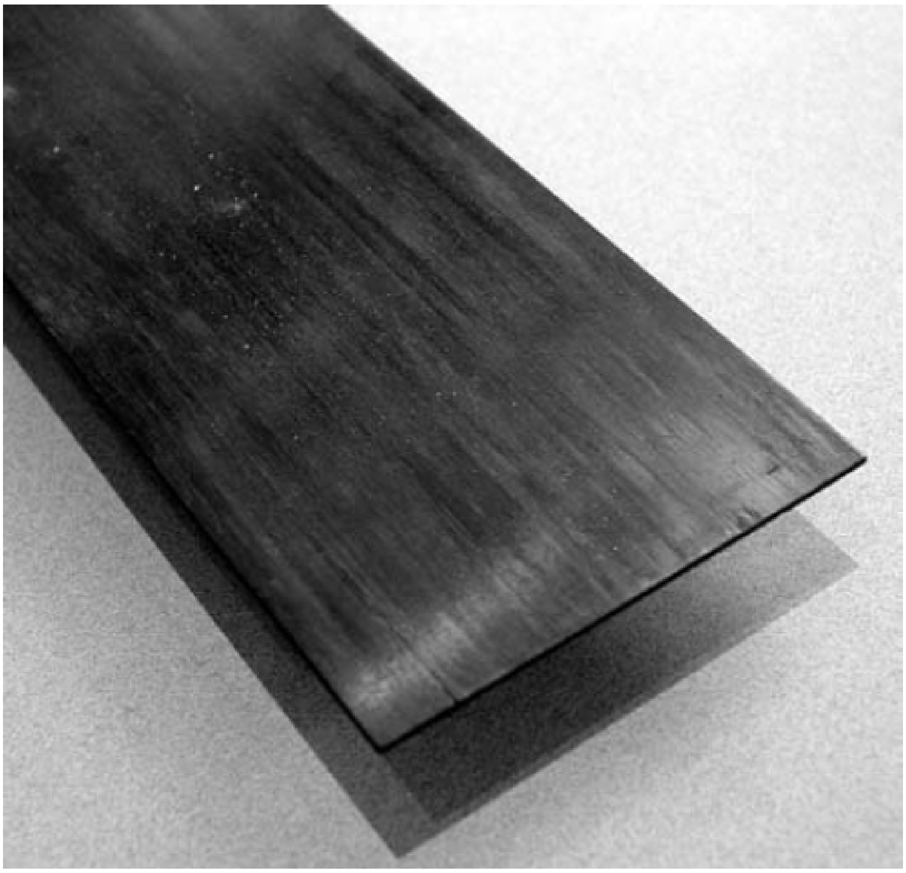

Figure 5: CFRP laminate [3]

In this project work, Sika CarboDur S: Pultruded carbon fiber plates (refer figure 5) for structural strengthening have been used. There are three types of CFRP plate's available namely high strength (Sika CarboDur M), intermediate modulus (Sika CarboDur S) and high modulus (Sika CarboDur H) plates respectively. Table 1 shows the properties of these plates and Table 2 shows the detailed properties of the particular plate being used in this project i.e. Sika CarboDur S plates which are pultruded carbon fibre reinforced polymer (CFRP) laminates, designed for strengthening concrete, timber, masonry, steel and fiber reinforced polymer structures. [Reference product data sheet 34, 35]

Table 1: Properties of different types of CFRP Plates available.

PLATE TYPE	ELASTIC MODULUS (GPa)	TENSILE STRENGTH (MPa)	ULTIMATE STRAIN (%)
Sika CarboDur M	**210**	**2900**	**1.35**
Sika CarboDur S	**165**	**2800**	**1.7**
Sika CarboDur H	**300**	**1350**	**0.45**

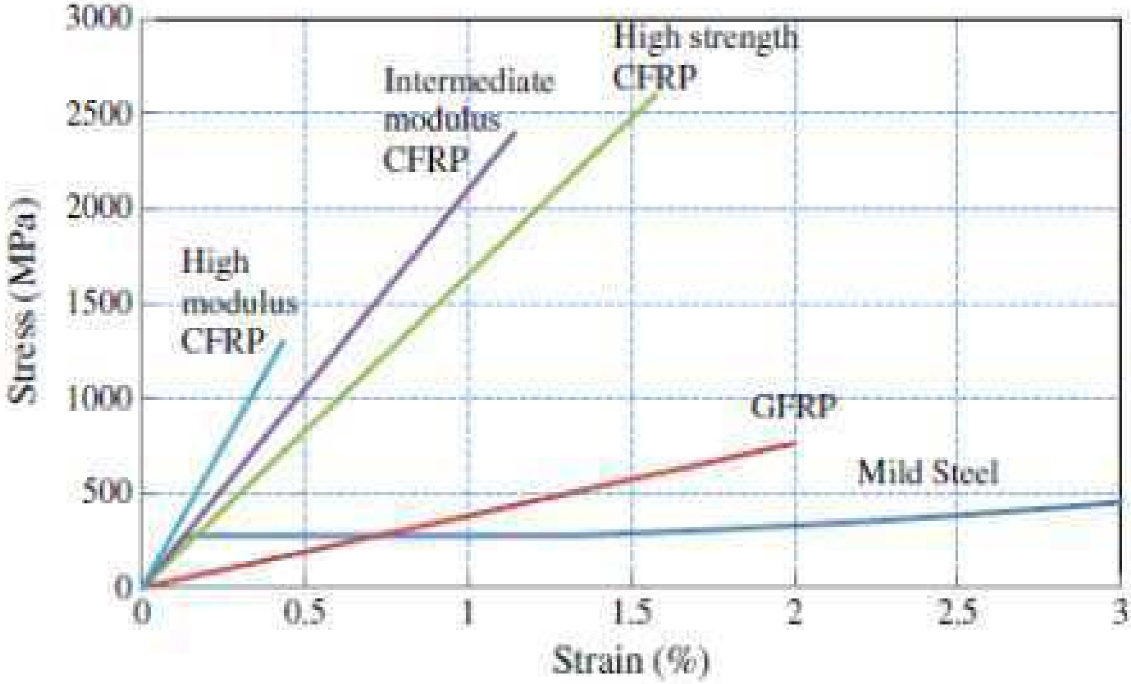

Figure 6: Typical stress strain graph of FRPs and Mild Steel [32]

Table 2: Properties of adhesive Epoxy [34,35]

Name	Width (mm)	Thickness (mm)	Cross sectional area (mm^2)	Elastic modulus (GPa)
Sika CarboDur S 514	50	1.4	70	165

2.2 Adhesive: Epoxy Resin

Epoxy resins are a class of thermoset materials broadly used in structural applications because they offer a distinctive blend of properties that are inaccessible with other thermoset resins. Epoxies have characteristic high strength, low shrinkage, and excellent adhesion to various structural materials, effective electrical insulation, chemical and solvent resistance, low cost and low toxicity. They are chemically compatible with most structural materials and get wet readily, thus suitable for composite applications.

Epoxy resins have wide applications as adhesives, encapsulates, coatings and binders. An epoxy resin system consists of the base resin, curatives and the modifiers. The different combinations of these render varied physical and mechanical properties. When choosing a epoxy resin consideration is usually given to tensile strength, modulus and strain, compression strength and modulus, impact resistance, heat deflection temperature or glass transition temperature, flammability and durability in services depending on the type of requirement which can be further modified or tailored for specific applications.

The CFRP laminates are externally bonded as reinforcement to the steel structures using a separate structural adhesive such as epoxy resins. The adhesives are often applied to the steel surface/CFRP as a paste. These epoxies are generally two-component epoxy systems mixed in a particular fixed proportion.

The strength of the adhesive depends on some material factors such as:

- Variation in the thickness of the adhesive
- The extents of voids, disbands and other defects.

In this project, Sikadur-30LP (IN) [reference product data sheet 36], Adhesive for bonding reinforcement supplied by SIKA has been used to bond CFRP to steel surface. This epoxy is a thixotropic, structural two part adhesive, based on a combination of epoxy resins.

The adhesive was chosen based on an experimental project conducted as a part of Major Project-I in which four different types of epoxy samples available commercially were studied. The details of the samples studied are:

Sample A: Locally available Epoxy grade 8300/hardner grade 691.
Sample B : Sika Sikadur 30 LP (IN)
Sample C : ARALDITE A/B
Sample D : Fibbond 253

The test parameters considered were tensile strength, flexural strength, slant shear strength and their coefficient of thermal expansion. It was thus concluded that Sample B was the most suitable epoxy available for this strengthening application.

Table 3: The physical/ mechanical properties of adhesive epoxy resin at 25°C curing temperature for 7 days. [Reference product data sheet 36]

Epoxy	Tensile Strength (MPa)	Compressive Strength (MPa)	Shear Strength (MPa)	Flexural Strength (MPa)	Tensile Bond Strength To Steel (MPa)	Elastic modulus (MPa)
Sikadur-30LP (IN)	**15-18**	**> 90**	**10**	**>25**	**18**	**10,000**

The two part epoxy system comes as Part A white in colour, Part B black in color and the final mix of epoxy adhesive prepared using Parts A and B mixed together is light grey in colour. The Ratio of Part A: Part B = 3: 1 by weight or volume as recommended in the technical data sheet.

2.3 Mild Steel

In this research, steel I-section and plates from Grade BR- E250 (Fe410W) as per IS:2062-2011 and IS:800-2007 has been used. Table 4 shows the properties of the material. Details of the beam and the components have been discussed in chapter 4.

Table 4: The physical/ mechanical properties of steel as per IS:2062-2011 and IS:800-2007. [49]

Grade Designation	Quality	Tensile Strength, Min (MPa)	Yield Stress, Min (MPa)	Ultimate stress, (MPa)	Percentage Elongation, Min at Gauge Length, Lo=5.65	E-Modulus, (MPa), Mean Value
E250 (FE410W)	**BR**	**410**	**250**	**410**	**23**	200000

All components of the beam i.e. steel I-beam, plates sections used in stiffener plates, end anchor plates, pack plates, loading plates have been manufactured using the same grade of steel, the properties of which are as given in Table 4.

Chapter 3: LITERATURE REVIEW

3.1 GENERAL

In chapter 1, we discussed the objective of this project, the existing methods of retrofit and the need for the study. CFRP composite materials are experiencing a continuous increase of use in retrofit and strengthening of structures around the world. A lot of work has been carried out in the research for a better understanding of the behavior of the retrofitted structures in varied conditions. Most work carried out is concentrated on the study of retrofitted/strengthened concrete structure although retrofit of steel structures using CFRP bonding systems has also started attracting a lot of attention and already many researchers have studied many parameters of the same. In this chapter a brief review of all the work carried out all around the world has been done.

The use of CFRP bonding to metallic structures was first implemented in mechanical, marine and aerospace engineering. CFRP has been successfully used to repair damaged aluminum and steel aircraft structures and also in marine applications in large ships and submarines [5, 6, 7, 8, 9]. Since then many researchers have studied the application of CFRP to steel structures and various aspects of strengthening, durability and environmental factors related to it.

3.2 Failure modes of retrofitted beams

J.G. Teng et al. [32] suggested that when a beam is retrofitted by adding a CFRP strip/plate to its tension flange i.e. assuming a beam in positive bending the kind of failure modes possible are [refer fig 7,9 and section 3.3.4] :

- In-plane bending failure
- Lateral buckling
- Rupture of the laminate at mid span when the maximum axial stress in the laminate reaches its ultimate strength
- Plate end debonding due to high localized interfacial stresses and peeling stresses in the vicinity of plate ends [32] or due to maximum shear in the bond-line at the end of the plate [26]
- Intermediate debonding due to local cracking or yielding at a distance from the end of the plates somewhere in the middle.
- Mid splitting of CFRP below point load
- Interlaminar shear failure at the end of laminate [26]
- Local buckling of compression flange
- Local buckling of the web.

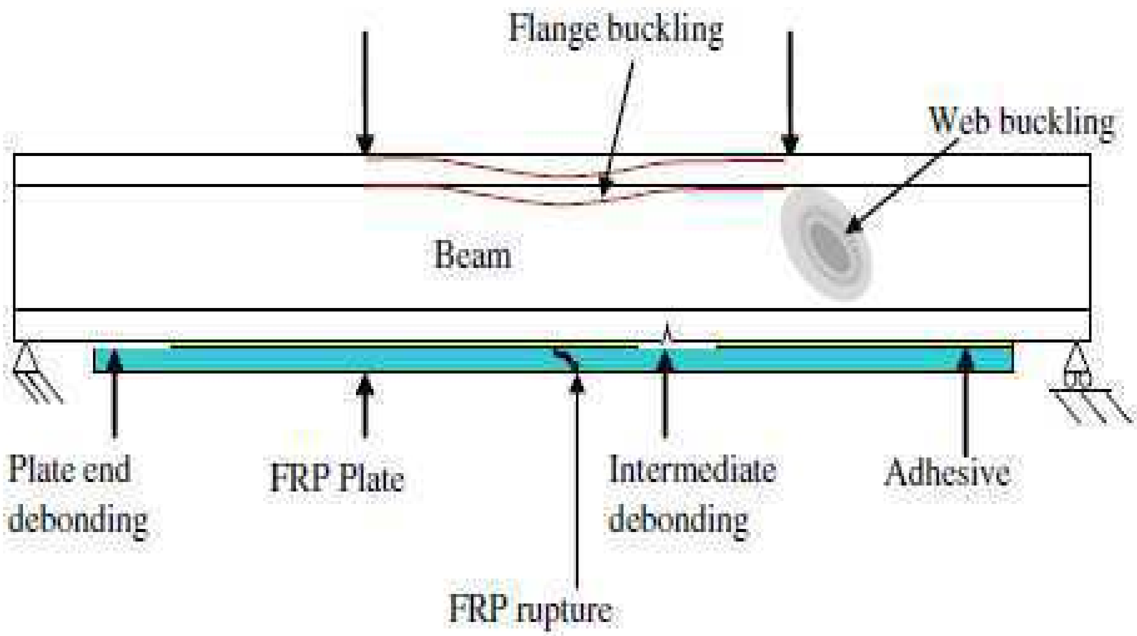

Figure 7: Different Failure Modes of Strengthened Steel Beams [32]

3.3 <u>Debonding failures</u>

Debonding failures are the most challenging issue in the flexural strengthening of steel beams as the adhesive is the weak link; most debonding failures depend upon the adhesive properties. A lot of work need be done in modeling bond-slip for FRP-to-steel interfaces with special attention to bonded FRP plates subjected to compressive stresses which leads to buckling failures [32].

3.3.1 **Characterization and Modeling**

Buyukozturk et al. [13] defined Debonding failure as the significant decrease in member capacity of a retrofitted system due to instigation or transmission of debonding. Debonding in FRP strengthened structural systems occurs at regions of high stress concentrations (i.e. due to material discontinuity or presence of cracks) depending upon elastic and strength characteristics of the retrofit systems, the constituent materials and their interface fracture properties. They reviewed the progress of understanding debonding problems in Reinforced Concrete (RC) and steel members retrofitted using FRP.

Upon studying the work of various researchers on modeling of debonding problems using:

1) Strength approach [23, 24, 13] that involves characterization based on elastic properties of materials. In this approach, prediction of debonding failure is made by calculating interfacial or bond stress distribution in FRP strengthened systems and further comparing them with the ultimate strength of the materials based on which the probable mechanism and load level of debonding failure is suggested. Many researchers have studied different methods of predicting debonding failure based on this approach with varied assumptions of shear, normal stresses and elastic behavior of materials. The solutions thus obtained provide close results to experimental results except for a very small zone at the ends of adhesive layer.

2) Fracture approach [23,13] that takes in account the fracture properties along with the elastic properties in strength approach. A lot of work has been done in aircraft industry since 1970's using linear elastic fracture mechanics (LEFM) on behavior of FRP-concrete bond but work on FRP-steel repaired members is very limited and needs consideration since the system behavior is based on fatigue crack propagation.
Semi-empirical and empirical models [23] based on certain parameters that influence their debonding behavior such as shear capacity of strengthened member, bond yield condition of FRP plate etc.

Thus, Buyukozturk et al. [13] proposed that a lot of work need be done in theoretical modeling at both material and structural level with consideration to diverse aspects of debonding problems in retrofitted structural members.

Analytical models show that a slip occurs in steel-CFRP joints between steel and CFRP which leads to the complicated behavior of the system, the slip and shear stresses are found to possess a linear relationship (studied in detail in next section) [24].

3.3.2 Bond-slip relationship

A precise bond-slip model for FRP-to-steel interfaces is of elemental importance to the properly understand, characterize and model the true behavior of FRP-strengthened steel members. As the bond-slip model illustrates the relationship between the local interfacial shear stress and the relative slip between the two adherends which can be obtained experimentally though tests carried on bonded joints [32]. Single-lap pull test has been found to be the most suitable method for studying full range behavior of FRP-steel bonded joints.

Fernando [37] conducted a series of single-lap pull tests on FRP-steel bonded joints using varied types of adhesives and concluded that a two-branch slip model (refer fig. 8 (a)) is only suitable to predict behavior of bonded joints employing a brittle linearly behaving adhesive. However it is not suitable for ductile non linear adhesives having a much higher strain capacity as compared to that of linear adhesive. Thus the shape of bond-slip curve for such joints was proposed to be trapezoidal (refer fig. 8 (b)) based on the test results obtained.

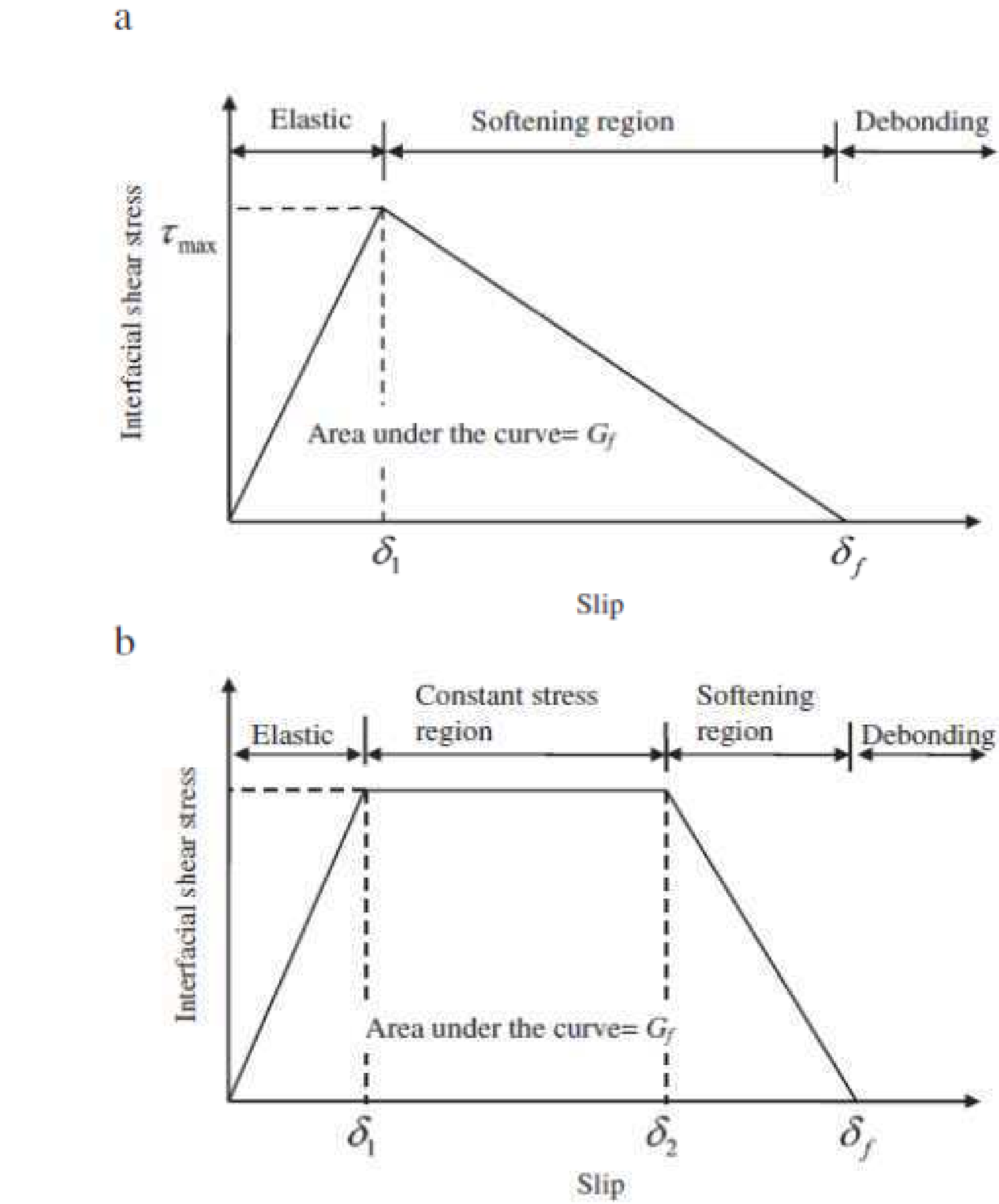

Figure 8 : Generalized Bond-Slip curves a) for linear adhesives b) for non linear adhesives [32,37]

Amer et al [48] studied the crack-dependant response of steel members strengthened with CFRP composite systems, subjected to axial tension. Nine specimens were constructed to study the effect of CFRP strengthening on steel members having various crack properties by taking into consideration the level of damage i.e. a_o/h ratios (a_o = notch size and h = specimen height) and CFRP reinforcement ratio. CFRP strengthening improved the load-carrying capacity and axial stiffness of the elements while the strengthening effect was more pronounced for increased a_o/h ratios and the crack propagation is dependent upon the debonding characteristics of the CFRP and the size of a_o/h ratios. Also, CFRP strengthening leads to significant increase in energy release rate.

3.3.3 Cyclic loading effects on debonding

The study [13] shows the performance of strengthened structures under cyclic loading may fall below that under monotonic loading in RCC retrofits due to brittle debonding failures, based on strengthening parameters and anchorage conditions. However, in steel members retrofitted using FRP composites not much effect has been seen under low amplitude fatigue loading but results under high amplitude cyclic loadings are not available and need careful consideration.

3.3.4 Debonding failure modes

Debonding in FRP strengthened materials, theoretically, can take place at:
[refer fig. 9] [13]

1) Steel/CFRP interface
2) Between Adhesive layer (also referred to as, cohesive failure)
3) Adhesive/CFRP interface
4) CFRP delamination

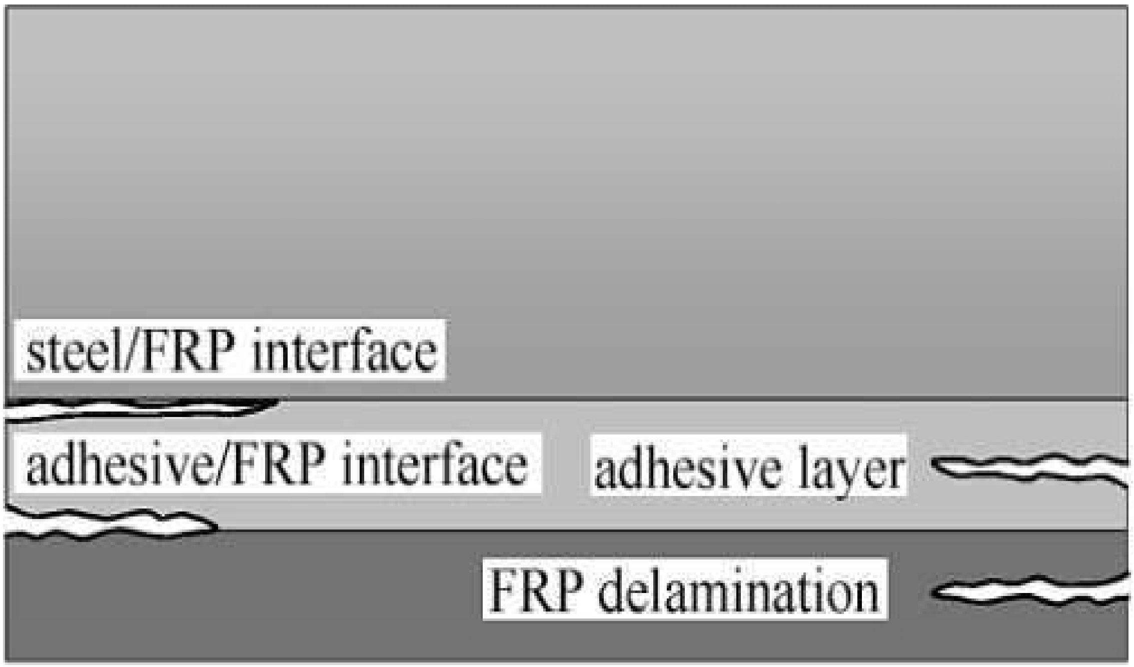

Figure 9 : Debonding failure planes [13]

Further, Emrani et al [41] suggested two types of failures modes occurring at the end of CFRP plates in a strengthening system, namely, interlaminar shear failure (delamination) at the end of laminates and debonding failure due to maximum shear in the bond line at the laminate ends.

3.3.5 Solutions to debonding failure

Xia and Teng [25] showed the effect of thickness of adhesive layer on failure mode of the bond. They demonstrated that as thickness of adhesive layer is increased from 2mm, debonding happens by plate delamination due to brittle failure mode instead of debonding between adhesive layers as in the case of thin adhesive layer. Also, adhesives with high ductility have been found to distribute the load more effectively within the adhesive layer in case of amplified loading.

Employing reverse taper or beveling the ends (refer fig. 10); CFRP plates decrease the peeling stress notably [12]. Further, adding steel plate anchorages or clamps at the end of CFRP plates improves the behavior of the bond [26]. Also good surface preparation (discussed in detail in chapter 4) and the elimination of any kind of contaminants from the surface of both steel and CFRP play an important role in improving the bond behavior of the strengthened system[12].

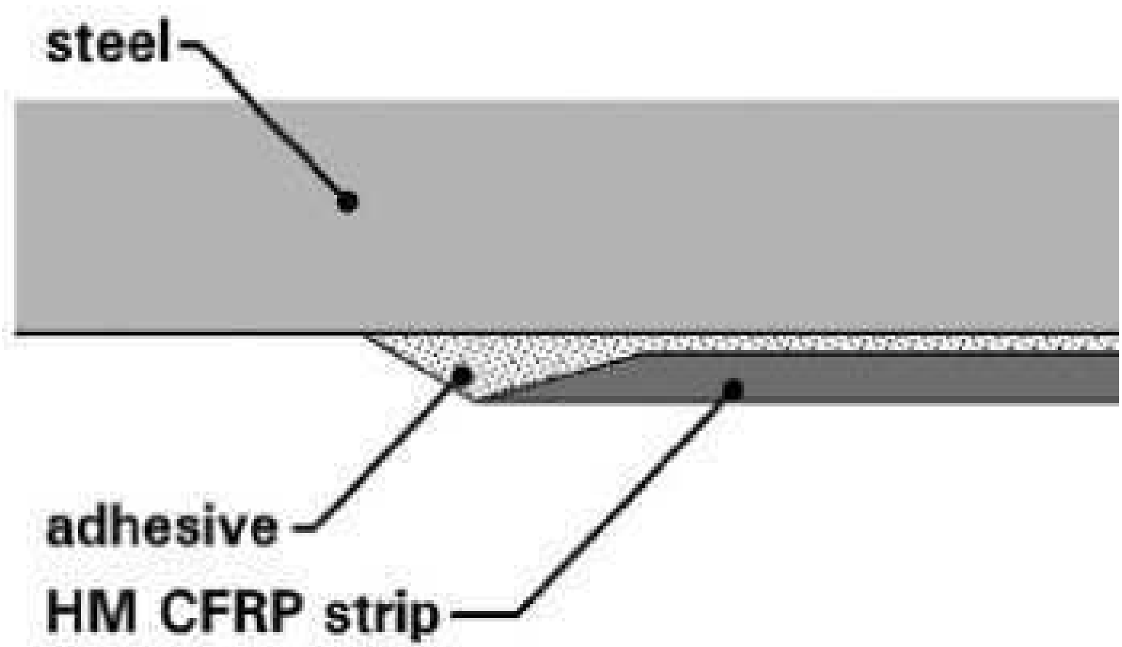

Figure 10 : Configuration of Reverse Tapered End [38]

Sen et al. [42] also designed steel clamps as end anchorages for CFRP laminates to resist the predicted peeling stresses but the clamps were kept of such a size so that no drilling was required in both CFRP and steel member. The clamp also facilitated the load transfer capacity of the epoxy adhesive.

Lui et al [28] recommended wrapping GFRP sheets around the bottom (tension) flange and a part of the web perpendicularly to the longitudinal CFRP sheets bonded for strengthening the member. These sheets were proposed to be attached along the whole length of the girder to avoid delamination of the CFRP sheets.

3.4 **Behavior of steel-CFRP joints**

Hart-Smith [18] developed a theoretical model to predict the strength of steel-CFRP double strap joints at room temperature by adopting bi-linear shear stress model to represent non-linear properties of the adhesive. In his model he adopted maximum shear strength as failure criterion to predict joint strength. He proved theoretically that a strength plateau exists for these joints i.e. an increase beyond a certain bond length (namely, the effective bond length) only relieves the already low stresses and has no effect on critical stress and strains in the adhesive. The ultimate load is thus governed by the critical stress concentration at the end of joints.

Nguyen et al. [15] used the Hart-Smith model [18] to characterize mechanical behavior of steel-CFRP joints subjected to elevated temperatures along with a kinetic model to describe the temperature-dependent mechanical properties of the adhesive based on its glass transition temperature (Tg). The results so obtained from the model were compared to those of an experimental data obtained by testing steel-CFRP double strap joints in tension at elevated temperatures ranging from 20 to 60°C. They obtained effective bond lengths at different temperatures by testing specimens with different bond lengths for two types of fiber layouts i.e. single layer and three layers. With the above experiment they made some important conclusions such as: 1) effective bond length increases with temperature to as high as twice for temperatures close to Tg , 2) experimentally verified the finding of Hart-smith model [18] that strength plateau for bond lengths exists, 3) joint stiffness decreases with temperature 20% reduction at Tg, 50 % at 10°C above Tg and 80% at 20°C above Tg, 4) failure is more rapid at temperatures higher than Tg, 5) decrease of joint ultimate load is related to shear strength degradation of adhesive as well as change of effective bond length. The decrease follows the same trend as the decrease in joint stiffness and thus can be described as a function of temperature and bond length.

Bocciarelli et al. [55] conducted numerical as well as experimental study on debonding strength of axially loaded double shear lap specimens between CFRP and steel. The failure mode in all cases was found to be steel-adhesive interface. Thicker adhesives produced higher failure loads but, significantly lower than yield load of the steel plate and no interaction between steel plasticity and interface debonding could be witnessed. Fracture and stress based models were used to estimate failure load and a good agreement between numerical and experimental results was achieved.

Zubaidy et al. [56] studied the effect of impact load on bond strength, effective bond length and failure modes of double strap joints between steel and CFRP. The bond strength showed significant increase under dynamic load particularly when bond length was kept less than the effective bond length. Although, the effective bond length was least influenced. The failure mode also changed from CFRP-adhesive interface failure to CFRP delamination failure under impact loading due to shear strength enhancement of epoxy adhesive in the joint.

3.5 **Retrofit Of Steel Bridge Members : Flexural strengthening**

Gillespi et al [27] retrofitted deteriorated girders removed from an old bridge and tested them in the laboratory. Four corroded bridge girders with a span of 9.75 m were removed, they were found to be uniformly corroded along their length with most corrosion concentrated on the tension flange of the girders; as is the case with most bridges. Examination revealed a loss of approximately 40% in the tension flange thus corresponding to a 29 percent reduction in stiffness. Only the tension flange of the girder was retrofitted using a single layer of CFRP as the webs were found to be intact and less affected by corrosion. The retrofitted girders were tested under cyclic loading within the elastic range upto a load approximately 50% of yielding load of the initial non-retrofitted girder. It was found that the retrofit lead to an increase of elastic stiffness by 10-37%. The inelastic strain in the tension flange were reduced by 75% as compared to initial non-retrofitted girder and ultimate capacity of the girders was found to be increased by 17 and 25 percent depending upon the severity of corrosion.

Lui X. et al.[28] conducted three-point bending tests on CFRP retrofitted girders with a span of 2.4 m with notches cut in the tension flange to simulate heavy loss of section due to corrosion. The beams were retrofitted in a way that one was covered with a CFRP laminate through-out the length where as the other was covered only one-quarter the length of beam. Lateral torsional buckling was restricted by providing four pair of lateral supports and the effect of bond length was studied. The mode of failure was found to be debonding of the CFRP at the midspan due to high stress concentration and high shear near the notch. The only difference being that in the first beam the debonding occurred slowly at mid span and propagated towards the end and in the other beam the debonding was sudden. Also the results suggested a 60 and 45 percent increase in plastic load capacities respectively in the two beams.

Tavakkolizadeh and Saadarmanesh [29] tested 1.3 m span girders with two types of notches- shallow and deep, under four point bending configuration with different lengths of CFRP bonded to each type of beams. The results showed that regardless of the length of the CFRP, the stiffness and the ultimate load carrying capacity of the beam were close to intact values of the un-notched un-strengthened beam. The beams tested were with CFRP lengths of 100, 200 and 300 mm for shallow notch and 200, 400 and 600 mm for deep notch. The main point of difference that could be seen in the results of deep and shallow cut notched beams was considerable loss of ductility.

Shulley et al [30] studied the method of retrofit using FRP strip bonded to the damaged web of the steel beams under three-point bending. In his study he used the method of cutting a hole to stimulate damage located in the shear span of the beam, 100 mm in diameter. Six different type of FRP sheets were used for retrofit. All retrofitted beams failed in same manner. With increasing load the FRP started to buckle over the region where hole was positioned because of principle compressive stress field being developed in the web followed by complete debonding of the sheet. As a result the retrofitted beam could not even attain as much strength as the initial undamaged beam.

Edberg at al. [31] applied four different retrofit schemes as shown in the figure below to the tension flange of a steel I-beam of span 1.37m. Aim of the study was to demonstrate advantages of using FRP in different arrangements in strengthening of steel girders. They performed four point bending tests on the specimens so prepared. The increase in stiffness was found to be 20, 30, 11 and 23 percent for arrangement a) , b), c) and d) as shown in fig. 11 respectively, while increase in strength was of the order of 42, 71, 41 and 37 percent for the same order of arrangements. Thus it was concluded that sandwich CFRP-plated retrofit (fig. 11(b)) was found to be the most efficient technique of retrofit while GFRP wrapped (fig. 11(c)) was found to be the least efficient.

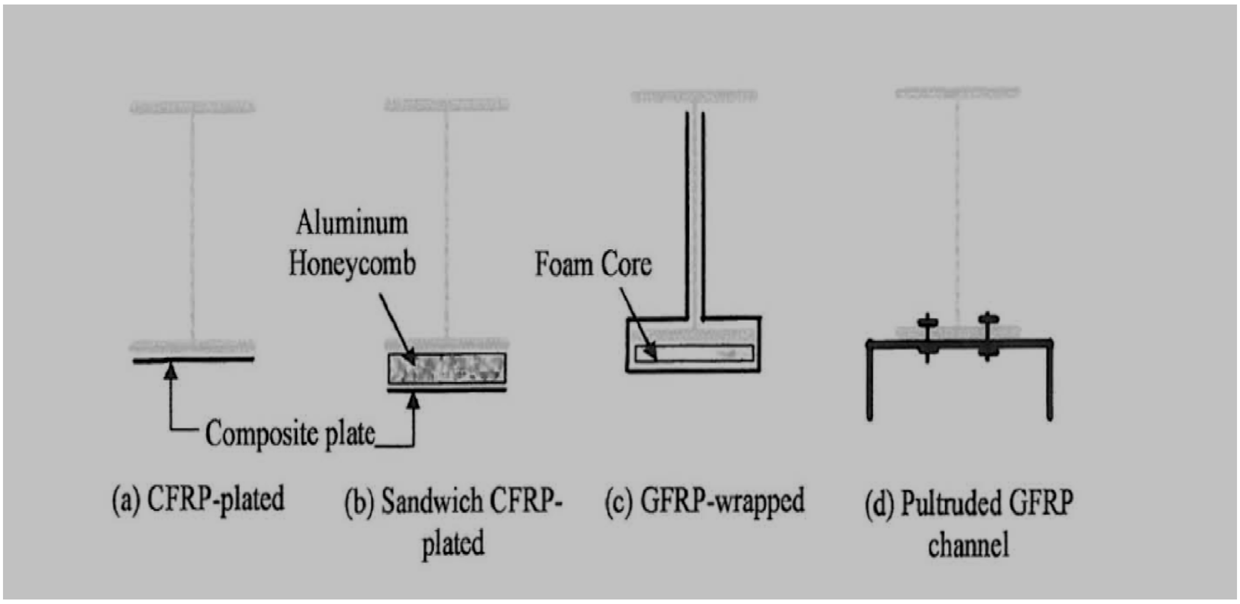

Figure 11 : Different Retrofit Scheme Arrangement [31]

Hmidan et al. developed an experimental program to observe the residual behavior of notch-damaged CFRP strengthened steel girders subjected to long-term loading configurations in colder regions. The beams were tested for two levels of sustained load i.e. 40% and 60% of the ultimate capacity of a short-term strengthened beam at temperature as low as -30°C. A total of thirteen strengthened beams in addition to one unstrengthened beam to act as control beam were monotonically tested in three-point bending after being exposed to the persistent load for 7000 h. Experimental results showed that the long-term loads significantly influence the load-carrying capacity of the beams due to the existence of creep damage near the crack-tip field above the notch. Cold temperature was also found to affect the capacity and flexural stiffness of the girders. The failure of the strengthened beams was consistently governed by CFRP-debonding, irrespective of the degree of temperature exposure. Local CFRP rupture was observed near the location of the notch in the beams subjected to 60% of ultimate load due to the combination of stress concentrations and creep damage of the fibers. Interfacial stresses all along the CFRP–steel interface were restricted by the sustained loads. Cold temperature resulted in decreased interfacial slip of the CFRP, while the thermal effect appeared to be insignificant on the magnitude of bond stress. Stress redistribution alongside the CFRP–steel interface is noticed caused by the enduring load.

3.6 <u>Fatigue behavior of CFRP retrofitted steel members</u>

One of the most important facets of strengthening of steel structures is to increase the fatigue life of the structure. The conventional method of using steel plates bonded to existing structure to extend fatigue life has several disadvantages as welding details are sensitive to fatigue loads.

Thus FRP strengthening is found to be very effective as compared to the traditional method for fatigue life improvement of steel structures [32, 31].

Gillespi et al. [39] continued their earlier research [27] by testing two strengthened girders under fatigue loading for 10 million cycles at the estimated stress range in the field. No debonding of any kind was found during periodical monitoring and inspection, carried out throughout the 10 million cycles.

Miller et al. [12] reviewed rehabilitation of steel girders using CFRP strips bonded to the tension flange of steel bridge girders. By analytical studies they found that nearly 98% of the total transferred force was within the first 100 mm of the end of CFRP bonded plate when a 457 mm plate was attached to the tension flange of a steel girder. The results of the analytical study were presented in the form of a graph as shown in figure 12.

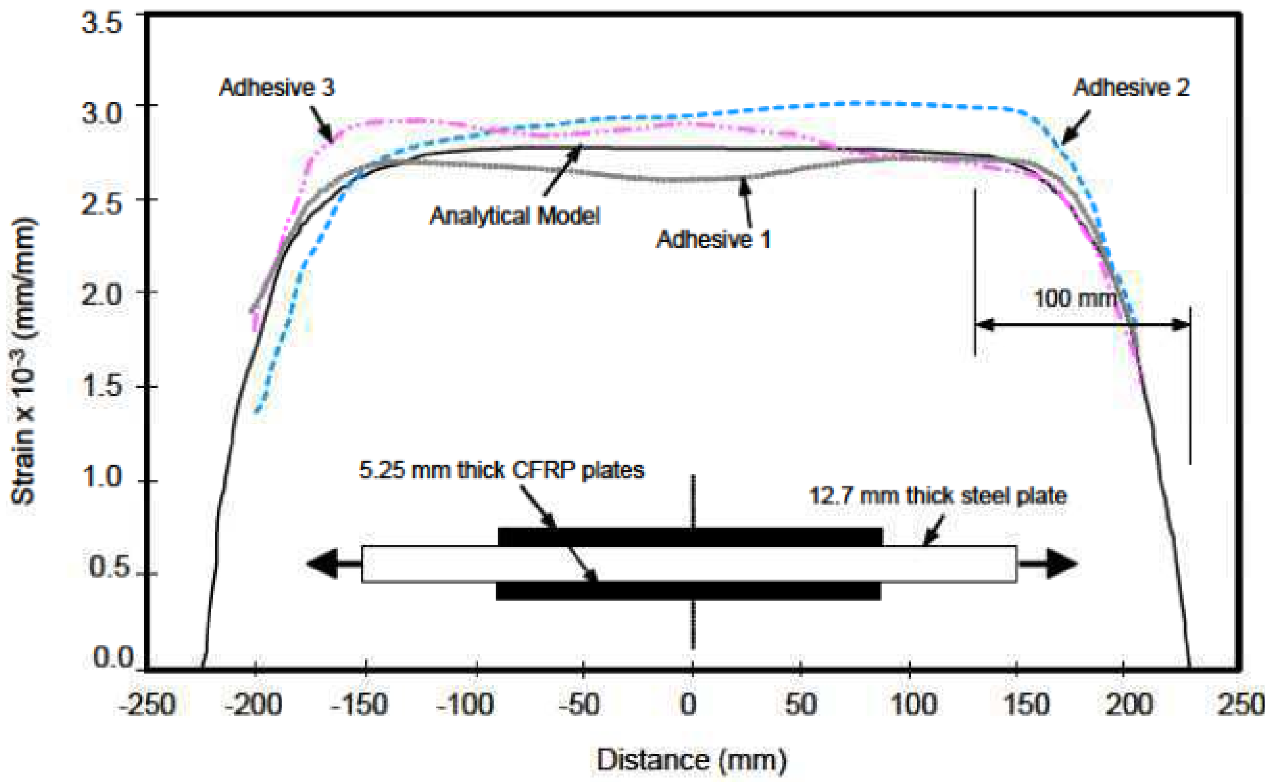

Figure 12: Measured and predicted strain distributions along the bonded length of a CFRP-Steel double lap joint.[12]

Buyukozturk et al. [13] also did an experimental study [21] involving steel specimen repaired using FRP sheets of various widths and lengths, being tested under fatigue loading which confirmed the effectiveness of this method by increasing the fatigue life of the retrofitted steel specimens and also drew attention to various research related issues in design and durability of the repair technique.

Basetti et al. [19,20] conducted various laboratory tests on FRP repaired notched steel coupons and later carried out a field study by implementing the method to a ninety-nine year old riveted steel bridge. The small- scale testing of FRP repaired central notched coupons was carried out under a stress range of 80 MPa and a stress ratio of (R=0.40) and it was observed that crack growth rate was radically decreased and the fatigue life was improved by approximately 20 times depending upon the pre-stressing level. The full scale tests were carried out by removing cross girders from the said old riveted bridge. The test results showed great effectiveness in preventing fatigue crack growth and initiation at rivet holes and also at other locations. No crack growth was recorded upto 20 million cycles in the retrofitted girders, however, the non-retrofitted girder cracked after 3.5 million cycles. Both studies showed that fatigue damaged steel-members repaired using FRP-Steel retrofit systems provide high structural strength and life of the flawed structure is extended at an economical cost.

Tavakkolizadeh and Saadatmanesh [40] prepared 21 specimens of steel beams of clear span 1.22 m, which were tested under four-point bending with a spacing of 200 mm between the consecutive points of loading. Further, the tension flange of each beam was notched at mid-span by 12.7 mm and CFRP sheets of 300 mm length were bonded to the lower face of the tension flange to cover the full width of the flange. These beams were tested for fatigue strength and it was found that for all stress ranges considered in the study, the technique of retrofit improved the fatigue life by a factor of 2.6 to 3.4 times that of non-retrofitted girder. Also, for the same crack length retrofitted beams were found to be stiffer than the unretrofitted beams.

J.G. Teng et al.[32] pointed out that debonding , both plate-end debonding and intermediate debonding are a vital matter of concern in the CFRP strengthened steel beams subjected to fatigue. While, plate-end debonding is not a concern as it can be prevented by various measures, such as

a) providing reverse taper at ends [38, fig. 10] which means providing more adhesive at ends,

b) by using softer adhesive at the plate ends and

c) by providing mechanical anchorages or clamps at the plate ends.

On the other hand intermediate debonding may have significant effect on crack growth rate in steel and needs appropriate consideration which is lacking in current modeling techniques.

Deng and Lee [59] studied crack initiation and crack growth rate by testing a series of small-scale beams bonded with CFRP laminates under fatigue loading. The results pointed out that stiffness of retrofitted beam deteriorated with crack growth. Also, a S-N curve was developed from the test results for further use in prediction of behavior and design of retrofitting of different sizes with same adhesive.

3.7 **Bonding characteristics**

CFRP strengthening of steel members involves bonding CFRP plates or strips to the steel surface using epoxy adhesive materials, which ensures transferring the stress along the length of the composite bond uniformly and thus results in lower stress concentration [24]. Formation of an uninterrupted bond between CFRP and substrate ensures full composite action by transferring shear stresses across the thickness of the adhesive layer. CFRP-to-steel bonding mechanisms are as follows:

- Physical bonding
- Chemical bonding
- Diffusion or inter diffusion theory
- Mechanical interlock theory

Further, the bond strength increases when the thickness of CFRP plate and adhesive decreases [24].

3.7.1 Effect of CFRP properties on bond

CFRP stiffness plays an important role in determining the fracture mode and the ultimate load capacity of the strengthened joint. Reduction of stiffness causes increase in bond strength. High modulus CFRP plates have been found to show a less ductile behavior as opposed to a higher load capacity.

Kennedy [43] analyzed the effect of bonded CFRP sheets on the transfer of load through steel plates with the help of an internal crack through the thickness of plate. The study showed that the load flowed through the length of the CFRP laminates, on the bonded face, across the crack and transfer quickly back into the steel.

3.7.2 Effect of Surface Preparation on adhesive bond

Surface preparation is the most important step governing the quality of an adhesive bond. Many problems associated with the adhesive bond failures can be avoided to a great extent by ensuring that the surface preparation has been done appropriately. Proper surface preparation produces a rough surface free from any kind of contamination and thus leads to the enhancement of chemical bonds between the steel and the adhesive. The basic aim of surface preparation is to render a surface free of contaminants, forming a fresh active surface, and if needed, to chemically amend the surface. Almost all treatment methods leave some degree of change in surface roughness but blast cleaning has been found to be the most efficient method for preparation of steel surfaces.[24, 38]

Moreover, the combination of sand blasting or blast cleaning with chemical or electrochemical treatments has also been said to cause bond to be more durable. On similar lines, Dawood and Rizkalla [52] have indicated that using a silane coupling agent for treatment of steel surfaces improved the bond durability significantly. Silane coupling agents are alkoxy silanes containing functional groups such as methyl groups, phenyl groups and other organic groups, used for surface modification of inorganic materials to impart water repellency and thus improved durability.

3.8 Durability of CFRP Composite joints

Durability of CFRP has been studied by many researchers. The results have shown that it behaves very efficiently and with great durability against environmental effects. However, the durability of steel and CFRP bonds still need more attention. It has been pointed out by Gholami et al. [24] that the performance of CFRP-steel strengthening system needs more concern for sever environmental exposures. The main factors influencing the system are temperature and moisture, in addition, factors like wet dry, thermal and freeze thaw cycles, chemical attack and exposure to Ultra Violet (UV) radiation also have a deleterious effect.

The key factor affecting the durability of steel members is the environmental surroundings. The CFRP composite system itself is non-corrosive, however, when the carbon fibers come into direct contact with steel a process of galvanic corrosion may initiate. Three basic necessities for galvanic corrosion to initiate between carbon and steel are: an electrical connection between the materials, presence of an electrolyte such as saline water and a constant cathodic reaction on the carbon. In absence of any of these, the galvanic cell is disrupted. Thus, a good adhesive, with high degree of resistance to chlorides, moisture ingress and freeze-thaw cycles, is a must in order to ensure good durability of CFRP composite systems.

Shulley et al. [30] studied the durability of bond for different types of CFRP and GFRP. Tests were performed on three types of CFRP and two types of GFRP by placing them in five different environments i.e. hot water, freeze/thaw, freezer, room temperature water and saline water, for two weeks. The specimens were then subjected to the wedge test i.e. inserting a wedge into the bondline and then replacing them back into their respective environments. The results showed that GFRP based system have a more durable bond with steel as compared to CFRP based systems. And the most durable bond was subjected to sub-zero, i.e. freeze temperatures.

Karbhari and Shulley [7] suggested a hybrid system with GFRP layer in between steel and CFRP in order to ensure both durability and performance. Allan et al [8] suggested a method of introducing a moisture barrier consisting of an additional foil sheet and a chopped strand glass laminate to envelop the CFRP laminate. Besides the electrical isolation of the CFRP laminate from steel by the resin matrix, thus two of the three necessary conditions (explained earlier) were restrained.

Tavakkolizadeh, and Saadatrnanesh [44] also investigated galvanic corrosion in steel and carbon fibers when coupled together. The results showed that the corrosion rate increases by 24 percent in deicing salt solution as opposed to an increase of 57 percent in seawater. Also a direct relationship was established between corrosion rate and epoxy coating thickness, where, applying a thin film of epoxy coating of the order of 0.1 mm decreased the corrosion rate in seawater by seven times and applying a thicker epoxy coating of the order of 0.25 mm the corrosion rate was further decreased to a twenty-one times.

West [45] also studied introducing a GFRP layer as a corrosion barrier similar to Karbhari and Shulley [7] and pointed out that although this technique is efficient, but can lead to blistering in the composite if proper care is not taken in laying the glass fibers. Brown [46] also studied the same and suggested that the blistering is caused due to glass fibers creating conditions favorable to osmotic pressure being developed which clearly is not healthy for maintaining a durable bond.

Hollaway [53] pointed out that however, the Carbon fibers are not susceptible to elevated temperatures, the mechanical properties of CFRP sheets reduce as the temperature over passes the glass transition temperature (Tg) of the matrix resin. Similarly, the adhesive layers are also sensitive to temperature and the mechanical properties of steel/CFRP bonds at elevated temperatures are governed greatly by the behavior of the adhesive [15]. Also, the effective bond length is dependent upon temperature and the load carrying capacity reduces greatly when temperature approaches glass transition temperature (Tg) of the adhesive [15].

In another study Nguyen et al [47] examined specimens at different load levels (i.e. 80%, 50% and 20% of their ultimate load at room temperature) and constant temperatures below and above the Tg i.e. from 35°C to 50°C. Results showed that at a constant tensile load (< ultimate load) and a constant temperature in elevated range the adhesively bonded CFRP joint exhibits a time-dependant behavior, including increased elongation and decreased load carrying capacity with time. The cyclic thermal loading was found to have no significant effect on strength reduction of joints as opposed to constant temperature exposure even at the same order of maximum temperature of 50°C which employs that joint degradation is not only a function of temperature but also time dependent and affected by thermal loading. As a result of joint strength degradation with time, the time to failure of the adhesively-bonded steel/CFRP joints vary, depending upon magnitude of load applied and constant exposure temperature.

The coefficient of thermal expansion of CFRP is much lower than that of steel [10]. Many researchers have focused their work on the low coefficient of CFRP as they are often subjected to stresses in temperature changes. High shear stresses have been found to develop across the adhesive joint due to the same and influence system performance.

Dutta and hui [11] studied the low temperature and freeze-thaw durability of FRP and its composites and also discussed their mechanical properties at temperatures as low as -60° C. It has been found that freezing condition degrade CFRP material due to matrix hardening which in turn causes micro-sized cracks parallel to the fibers and also at fibre-matrix interface. It is suggested that expected service temperature should be about 15°C lower than the glass transition temperature (Tg).

Miller et al. [12] and Buyukozturk et al. [13] investigated durability of CFRP and steel bond under varying environmental conditions and fatigue. And concluded that the bond durability of FRP bonded steel systems have reported degradation caused by temperature and moisture cycle in addition to galvanic corrosion problems. But the current stage of knowledge on bond durability is very limited. Theoretical modeling studies focused in bond durability are yet to be performed.

Bassetti et al [20] also studied the influence of varying the type of adhesive (epoxy) and the curing temperature on the fatigue behavior of strengthened members. The test results showed no significant effect on the fatigue life or behavior.

The behavior of CFRP in salt water has also been studied by a few researchers and a difference in degradation from that of related to moisture alone has been found. The tensile strength of CFRP/steel joints decreased by 17% after 12 months immersion in salt water at 20° C [16].

The ultra violet radiations in the sunlight cause damage to the polymers due to splitting of the bond of the molecules present in them. The damage is initiated due to absorption of UV rays causing bond dissociation and is continued due to subsequent reaction with Oxygen. The degradation is of the order of a few microns but leads to potential stress concentration zone and makes the damaged area prone to further environmental attack due to various other factors. It has been revealed by experimental studies that the tensile strength and elastic modulus decrease by 15 to 20% on exposure to Ultraviolet rays present in sunlight due to brittleness caused to the resin and fiber matrix interface. However, Nguyen et al [54] suggested that UV radiation has no role in reduction of stiffness and he suggested that the elevated temperature alone causes the brittleness and the reduction in tensile strength by about 35%. Thus, the effect of UV radiations on joint strength needs a more careful investigation.

3.9 **CONCLUDING REMARKS**

In this chapter, the work of various researchers in the field of retrofit of steel members using CFRP bonding systems, all over the world, has been summarized. Work on various aspects of this strengthening technique such as the modes of failure, specially the major concern i.e. the premature debonding was studied and the various solutions proposed by numerous researchers to address this problem were reviewed. Further, the previous work carried out by several other researchers in the retrofit of steel beams and girders for flexural strengthening as well as the behavior in fatigue load was also analyzed and the results have been discussed. Also, the effect of CFRP and the surface preparation of Steel on bond were considered and bonding characteristics such as transfer of forces and the bonding mechanism was discussed in brief.

It can be concluded that:

1. From the work that has been done in understanding the bond behavior, failure modes and bond characteristics of the steel-CFRP bonding system. The various modes of failure can be summarized as debonding failure (at interface of adhesive and adherands), bond-slip, CFRP rupture, yielding of steel, delamination of CFRP, adhesive layer failure along the bond line. It has been concluded that debonding failure is of major concern as it leads to ineffective bond behavior and utilization of complete strength of CFRP and steel is not ensured.

2. As debonding failure is a major concern, thus, methods of avoiding it have been proposed such as provision of end-anchor plates and beveling of CFRP end and keeping the length of CFRP bond longer so as to keep end in regions of lower bending. Out of these the provision of end-anchors was found to be the most suitable and thus selected for this study.

3. The behavior of steel CFRP joints has been studied by various researchers and it has been found that increasing the thickness of adhesive layer reduces the efficiency of the bonded retrofit technique due to shear deformation.

4. In case of requirement of a higher order of upgradation than that rendered by using a single CFRP laminate successive layers of laminates can be provided to increase the stiffness of the section. However, it has been shown that thicker the reinforcing material layers, the higher the chance of bond failure and thus the efficiency for utilizing the full strength of the CFRP decreases. Thus, a balanced design should be considered to effectively utilize the strength of CFRP and also to ensure cost-effectiveness of the method.

5. The use of CFRP is not only effective and limited to the restoration of the lost capacity of a degraded steel section, as a repair technique, but is also quiet significant in effective strengthening of existing steel structures for upgradation to resist higher loads.

6. Four-point bending tests show small influence of CFRP bond length on the delamination failure. However, three point bending tests have shown importance of bond length on this particular mode of failure, due to presence of shear stresses along the entire span of the beam and hence are a preferred method of study of steel-CFRP bond behaviors.

7. Epoxy bonded CFRP sheets are quiet promising in extending fatigue life of steel structures because of significant reduction in crack propagation.

8. CFRP bonds are susceptible to environmental conditions and behave differently at elevated and lower temperatures. Also, galvanic corrosion is a major concern, the methods to avoid galvanic corrosion have been proposed. The choice of adhesive is very important to ensure a durable bond

Techniques for eliminating galvanic corrosion:

- the use of a nonconductive layer of fabric between the carbon and the steel,

- an isolating epoxy film on the steel surface or a moisture barrier applied to the bonded area.

- steel and carbon fibers coated with a thin film of epoxy can be coupled together. Since most structural adhesives are insulators, and provided that a continuous film of adhesive can be maintained over the bonded region, galvanic corrosion can be minimized.

- In order to prevent the expected galvanic corrosion completely, a corrosion barrier (glass fabric layer) can be placed between the CFRP plate and the steel during the bonding process. Several studies have shown this procedure to be effective in preventing galvanic corrosion. However, care must be taken such that the placement of glass fibers does not lead to blistering of the composite. Also, glass fibers placed into a carbon fiber weave are found to promote blistering of the composite by creating conditions favorable for osmotic pressure to be developed. Clearly, water held within the bond line by osmotic pressure is not favorable for maintaining a durable bond.

- Coupling CFRP with aluminum

- Avoiding mechanical bolting for bonding steel and CFRP.

Chapter 4: EXPERIMENTAL PROGRAM

4.1 GENERAL

In the previous chapter, the existing work in the field of retrofit of steel members using CFRP bonding systems has been reviewed. As has been pointed out earlier (refer section 1.1), most existing bridges in our country need retrofit and corrosion is one of the major issues. Thus, the main focus of this study lies in retrofit of corrosion affected steel bridge members. Based on the study of existing work, it has been concluded that the bottom flange of the member i.e. the tension flange is most susceptible to corrosion due to accumulation of debris. Thus, in this study, only bottom flange i.e. the tension flange has been artificially corroded and then retrofitted. Also, it is quite evident that premature debonding leads to reduction of effectiveness of the retrofit and the most effective way to avoid this is the provision on end-anchorage to the CFRP laminates. In this project, an experimental program has been proposed to verify the effectiveness of the method of retrofit of corrosion affected beams using CFRP and to compare the results of those of retrofit with CFRP alone and CFRP with Steel plate end-anchorages. Furthermore, the strengthened members have been exposed to extreme conditions of exposure to saline water and wetting-drying cycles, to study the durability of the technique. The detailed experimental program has been discussed in this chapter and the results and conclusions of the experiment have been elaborated in the subsequent chapters.

4.2 TEST BEAM DETAILS

In this research, steel I-beams, CFRP sheets and epoxy adhesives have been chosen as discussed in detail in Chapter 2. Further, the sectional properties and the component details of the test beams will be discussed in this section.

Table 5 and 6 give the details of I-beams and its components their dimensional section properties and also the scheme of arrangement of retrofit. Figure 13 and 14 show section view of the test beam along with component details and dimensions of beams with End-anchorage and without End-anchorage respectively. Figure 15 and 16 give the elevation and plan of the strengthened beam showing the retrofit arrangement and location of End anchorage and CFRP laminates.

Steel End-Anchorage plates

As pointed out by various researchers (discussed in Chapter 3, Section 3.3.6) providing end-anchorages and clamps are an effective way to avoid premature debonding. Thus, steel plates have been provided to act as an anchor at both ends of the CFRP laminate as shown in fig. 13 and 15. The dimensional details of the plates have been provided in table 5.The plates have been attached to the bottom flange with the help of bolts. The width of the plates has been kept such that the bolts are provided outside of the CFRP laminates so that there is no drilling through laminates thus no reduction of sectional area and also the carbon fibers are thus not exposed directly to the steel due to punching/ drilling. Also, pack plates (Fig. 13) have been provided between end-anchorage plate and the bottom flange plate outside of the width of CFRP laminate of the same thickness as that of CFRP and adhesive combined so as to ensure that there is no gap (as shown in Fig 13).

Bolts

Bolts were provided to hold the end anchorage steel plate to the bottom flange plates also help increase the bonding between the adhesive, CFRP and the steel beams. Four bolts have been provided in each Anchor plate (for details refer table 5). The location and the arrangement scheme of proposed bolts have been shown in Fig 13 and 15.

Stiffener Plates

Stiffener plates (details given in Table 5) were provided to render stiffness to the compression flange and web against local buckling failure (refer Fig. 7) under direct point load. The stiffeners have been placed in the mid-span of the beam under the point of application of direct load. The schematic arrangement of stiffener plates has been shown in Figure 13, 14, 15 and 16.

Loading Plate

A plate of thickness comparable to that of flange thickness of the beam (details given in table 5) has been provided in the region of application of point load in order to ensure distribution of load to a broader area and to avoid local failure of flange plate. The plate was provided at the mid span of the beam under point of application of direct load.

Table 5: Steel I-beam and its components dimensional details

<table>
<tr><th colspan="2">Description</th><th>UC 152X152X37</th></tr>
<tr><td colspan="2">Flange Width, (mm)</td><td>152</td></tr>
<tr><td colspan="2">Overall Height, (mm)</td><td>152</td></tr>
<tr><td colspan="2">Flange Thickness, (mm)</td><td>11.5</td></tr>
<tr><td colspan="2">Web Thickness, (mm)</td><td>8</td></tr>
<tr><td colspan="2">CFRP width, (mm)</td><td>50</td></tr>
<tr><td rowspan="3">End-Anchorage Plate Details</td><td>Length, (mm)</td><td>150</td></tr>
<tr><td>Width, (mm)</td><td>100</td></tr>
<tr><td>Thickness, (mm)</td><td>6</td></tr>
<tr><td rowspan="3">Stiffener Details</td><td>No. provided</td><td>6</td></tr>
<tr><td>Height, (mm)</td><td>90</td></tr>
<tr><td>Thickness, (mm)</td><td>6</td></tr>
<tr><td rowspan="3">Loading Plate Details</td><td>Length, (mm)</td><td>150</td></tr>
<tr><td>Width, (mm)</td><td>140</td></tr>
<tr><td>Thickness, (mm)</td><td>10</td></tr>
</table>

Table 6: Retrofit Scheme Details of Test Beams

Mark no.	Description	Condition prior to strengthening	CFRP Bonded	End anchorage provided	No. of bolts provided	Diameter of bolts provided (mm)	Exposure condition subsequent to strengthening.
Beam 1	UC 152X152X37	Corroded	NO	NO	-	-	No exposure
Beam 2	UC 152X152X37	Corroded	YES	NO	-	-	No exposure
Beam 3	UC 152X152X37	Corroded	YES	YES	4	10	No exposure
Beam 4	UC 152X152X37	Corroded	YES	NO	-	-	Saline water. 30 wetting-drying cycles.
Beam 5	UC 152X152X37	Corroded	YES	NO	-	-	Saline water. 30 wetting-drying cycles.
Beam 6	UC 152X152X37	Corroded	YES	NO	-	-	Saline water. 30 wetting-drying cycles.
Beam 7	UC 152X152X37	Corroded	YES	YES	4	10	Saline water. 30 wetting-drying cycles.
Beam 8	UC 152X152X37	Corroded	YES	YES	4	10	Saline water. 30 wetting-drying cycles.
Beam 9	UC 152X152X37	Corroded	YES	YES	4	10	Saline water. 30 wetting-drying cycles.

Table 6 gives the detail of the beams used in the project along with the condition of beams at different stages of the experiment i.e. the condition of the test beams prior to strengthening operation and the exposure condition of the beams subsequent to strengthening activity before conducting the test. In addition, the details of the retrofit scheme i.e. CFRP, end anchorages and details of bolted connection have also been provided. Further, for dimensional details of the beams and the components, Table 5 in conjunction with fig. 13, 14, 15 and 16 can be referred.

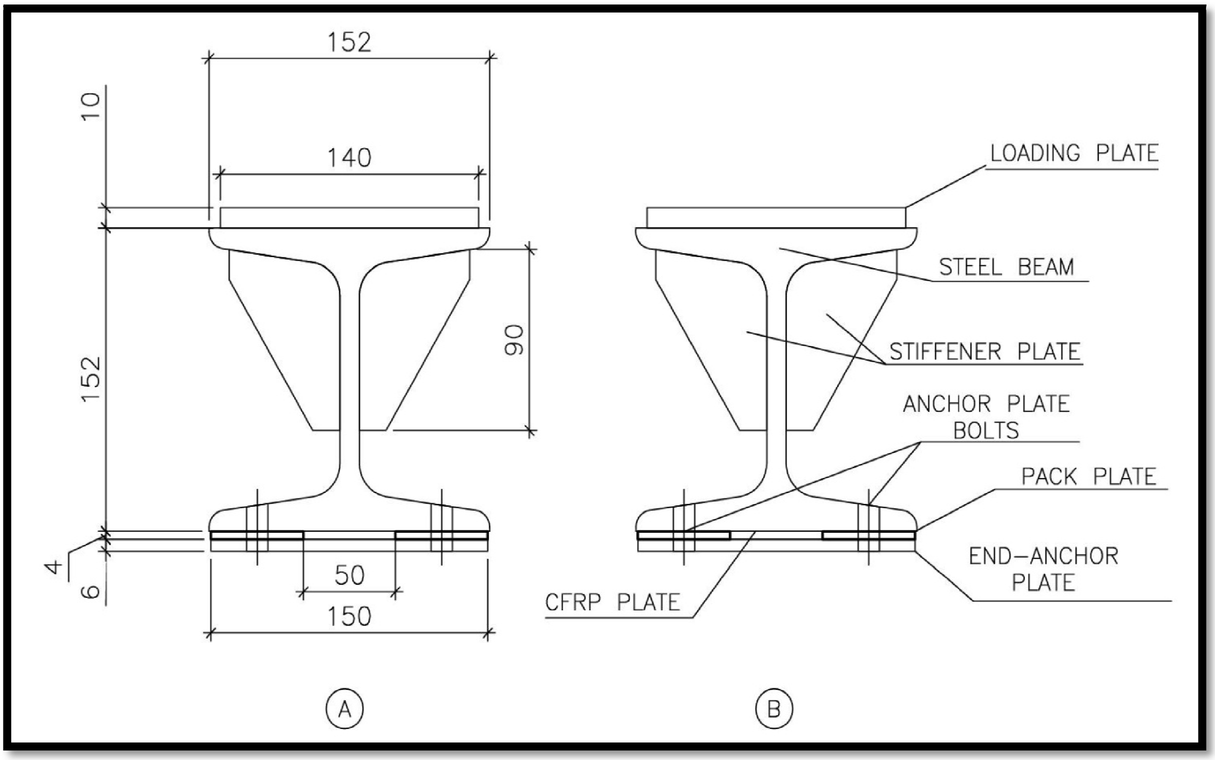

Figure 13: A: Dimensioning Details of Strengthened Test Beams with End Anchorage.
B. Component Details of Strengthened Test Beams with End Anchorage.
(Beam 3, Beam 7, Beam 8 and Beam 9)

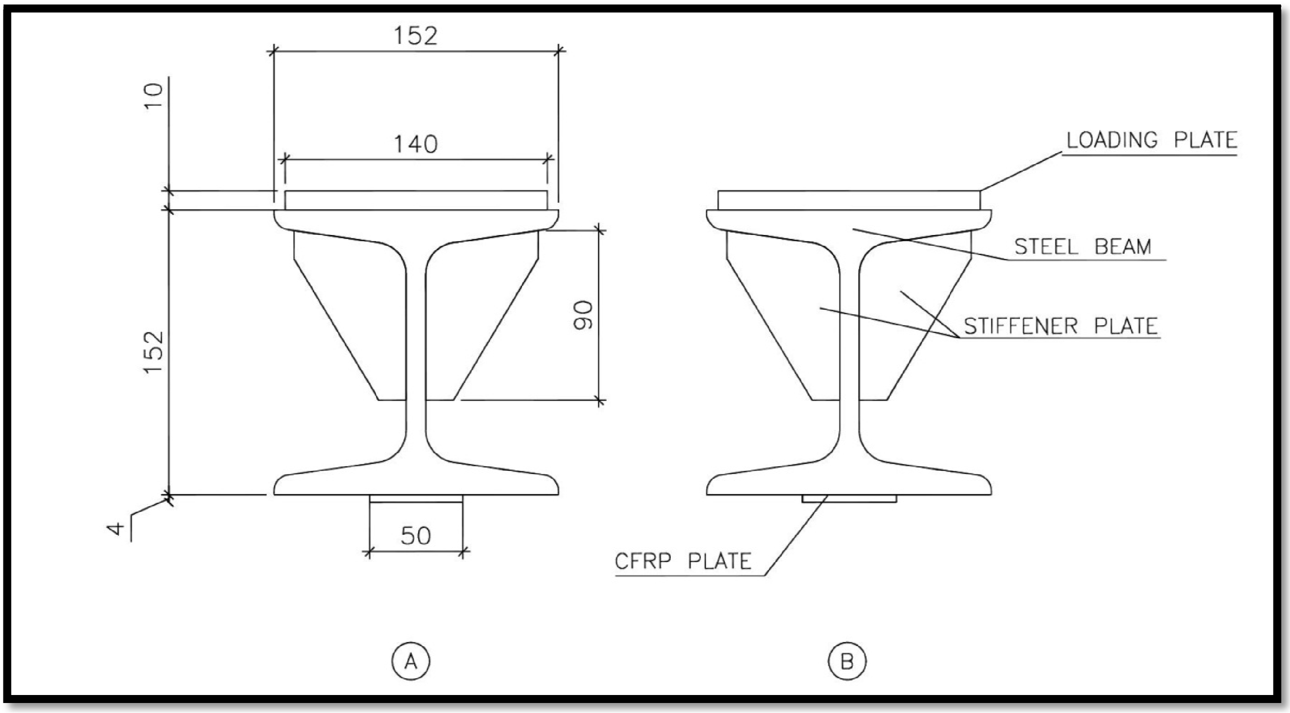

Figure 14: A: Dimensioning Details of Strengthened Test Beams without End Anchorage.
B. Component Details of Strengthened Test Beams without End Anchorage.
(Beam 2, Beam 4, Beam 5 and Beam 6).

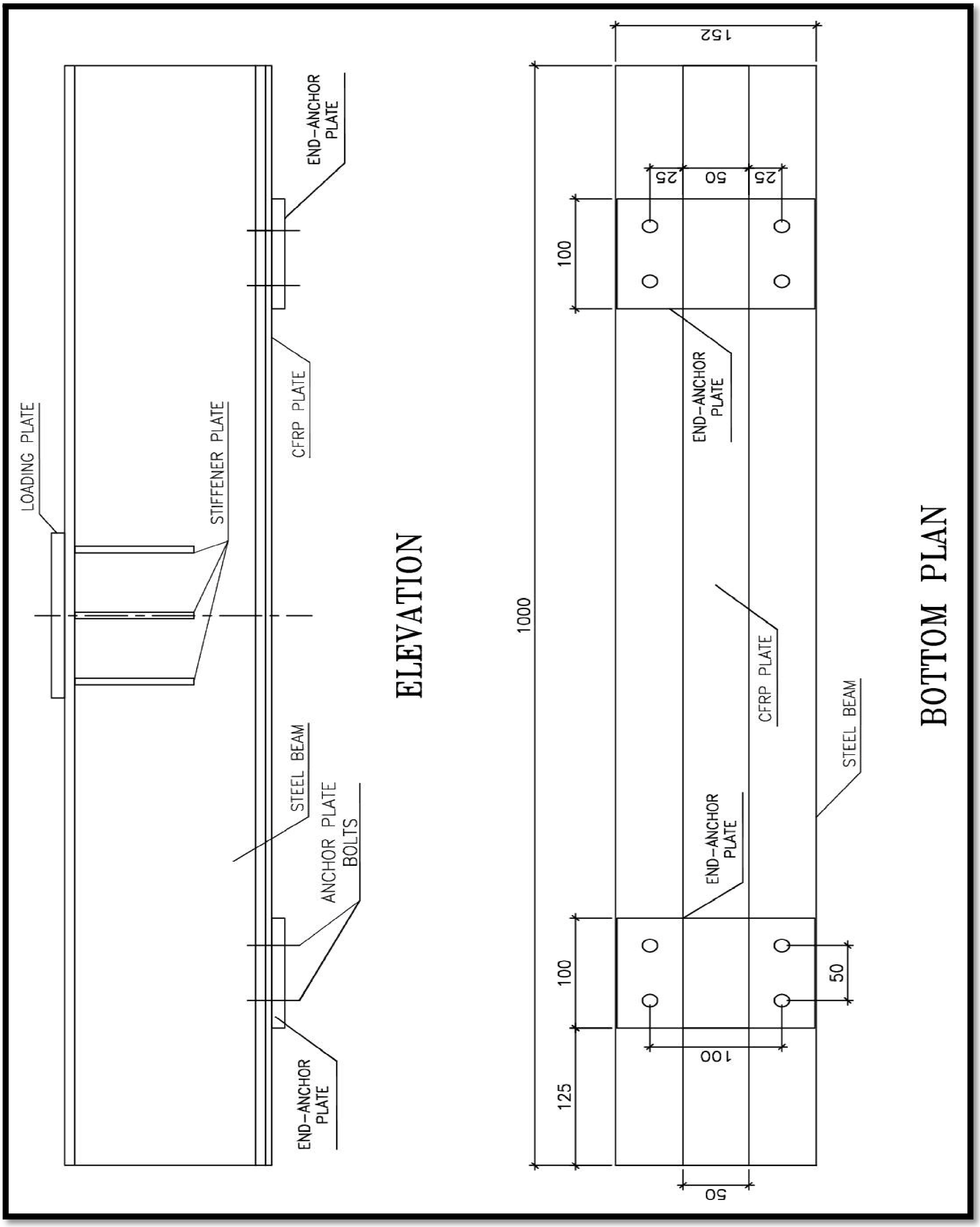

Figure 15: Details of Strengthened Test Beams with End Anchorage. (Beam 3, Beam 7, Beam 8 and Beam 9)

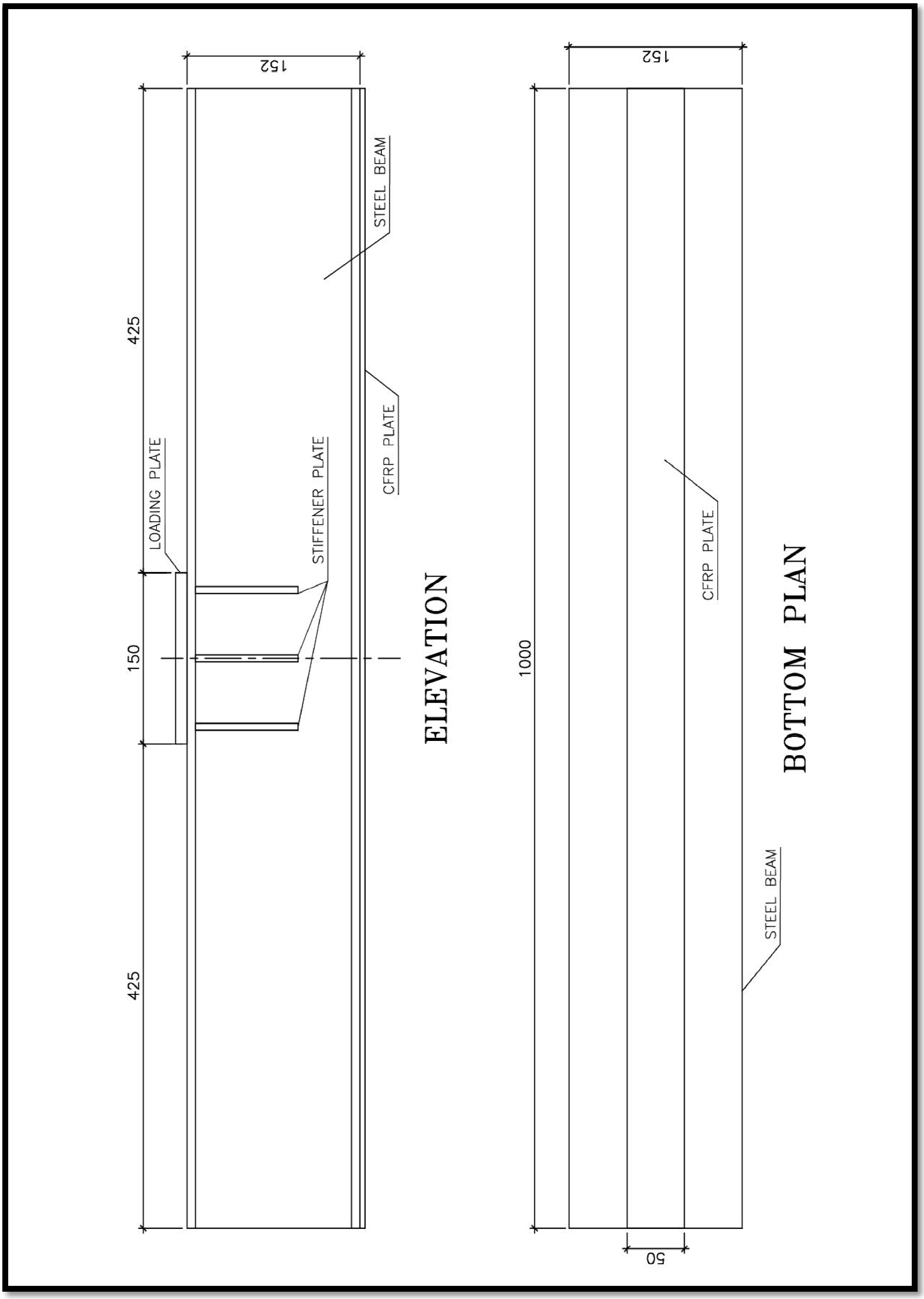

Figure 16: Details of Strengthened Test Beams without End Anchorage. (Beam 2, Beam 4, Beam 5 and Beam 6)

4.3 ADOPTED PROCEDURE

Preparation of Test Beams

In the process for preparation of the test beams, first the steel anchor plates and the bottom flange of the section of the steel I-beams were punched in the region of the end of CFRP plates as per detail shown in Fig 16. Also, the loading plate (refer section 4.2: loading plate) and the top flanges of the section of the steel I-beams were punched in the mid-span as per detail shown in Fig 15 and 16. Further, the stiffener plates were fabricated and welded at the desired location.

Corrosion Process

The experiment focuses on retrofit of girders that have been affected by corrosion therefore it is necessary for the beams to be corroded in such a way that there is significant loss in section and in strength. Thus, the beams were artificially corroded by exposing them to concentrated HCL, concentrated HNO_3 and Aqua Regia. Aqua regia, or nitro-hydrochloric acid is an exceedingly corrosive mixture of acids, a incensed yellow or red solution. The mixture is formed by freshly mixing concentrated nitric acid and hydrochloric acid, optimally in a volume ratio of 1:3. Fig 17 shows image of typical beam after corrosion. The flange thickness of the bottom flange was reduced from 11.5 mm initially to 4mm at the mid span by ponding concentrated acid at the desired location. A total of 9 beams were corroded for this project as per Table 6.

Figure 17: Typical beam after corrosion

Test 1 - Testing of control beam

After corrosion, one of the nine beams (Beam 1) was tested without any kind of retrofit, surface preparation or repair to act as control beam to compare the results of the strengthening. The loading plate was attached to the top flange using location bolts of 10mm diameter. The beam was tested on an setup based on three-point bending test using the Universal Testing Machine (UTM), with the end supports 800mm apart and a point load applied at the mid span of the beam as shown in Fig. 18. The two roller supports carried the reactions and load was applied at one point, therefore the loading state was three incremental bending point loads. The beam has been tested to failure in order to determine the ultimate load capacity of the beam.

Figure 18: Image of Beam 1 before test and during the test.

Preparation of the strengthened beams

The remaining eight beams have been retrofitted prior to testing. Thus, according to manufacturer's construction method statement [52], the surface of the steel beam was prepared by first wire brushing in order to remove surface rust and then blast cleaned such that the surface was free from grease, oil, rust and any other contaminants which could prevent adhesion. After preparation the surface was cleaned using ethanol to ensure there is absolutely no trace of contamination. The maximum wait time after blast cleaning before attaching the CFRP plates is 48 hrs. But as the epoxy chosen does not require any priming, thus in order to avoid reoccurrence of corrosive layer the bonding process was started immediately after drying of the steel surface.

Further, the CFRP plates were cut to length using a rotary disc cutter by putting tape on the location of cuts in order to prevent excessive dust generator. The CFRP strips were supported on both sides during cutting to avoid splintering of the ends and to cut perpendicular to the fibres. After cutting the surface of the plates were cleaned using a white cloth and ethanol until there are no trace of black dust appearing on the cloth. The plate was left to dry before application of the adhesive.

Adhesive Preparation

Component A and B of the two part epoxy system (refer chapter 2, section 2.2) were measured in the required quantity and component B was added to component A in the correct proportion in a mixing container. The two were then mixed slowly so to avoid entrapping of air using a spatula – like tool for about 3 minutes till a homogenous mix with uniform grey colour and appearance was achieved. The adhesive pot life begins when the resin and hardener are mixed. It is shorter at high temperatures and longer at low temperatures. Additionally, the greater the quantity/volume of material mixed together at one time, the shorter the pot life. Thus the mix obtained was immediately used.

The sequence of operations was planned to ensure that the adhesive can be applied, the plates bonded and installation completed within one hour of mixing the adhesive, or within 80% of the pot life.

Dependent on the substrate surface plane, profile and roughness, together with any plate crossings and the degree of loss or wastage, the actual consumption of adhesive can be higher.

Application

The thoroughly mixed epoxy adhesive was then applied carefully to the prepared, dust free steel surface with a spatula, scrapped to a very thin layer of order of 1-2 mms along the centreline on the top surface of the steel bottom flange at a pre-marked location. Immediately after, CFRP plate was placed onto the adhesive and pressed using a roller until material is forced out on both sides of the plate as shown in figure 19. The freshly bonded system should not be disturbed for at least 24 hours and any vibrations should normally be kept at a minimum during the curing period of the adhesive. The full design strengths of epoxy adhesives are reached after approximately 7 days at 20 °C. Thus, without any further disturbance the beam were left to be cured for an period of 10 days to ensure proper curing.

Figure 19: Image Showing Typical CFRP Strengthened Beam without End-Anchor Plates during Application of CFRP Sheets.

Air pocket check

To check the installed plates for air pockets / voids within the adhesive layer, or at the bond interfaces, they can be tapped with a metal bar (there are distinctly different sounds for fully bonded plate areas and any plate areas with air pockets / voids).

End anchorage

After the installation of CFRP plates, the end anchorage plates were installed on four beams namely, Beam 3, Beam 7, Beam 8 and Beam 9 (as per Table 6) using bolts (as per Table 5) and at the location as shown in Fig 13 and 15. Nuts were tightened appropriately using a spanner and check washers were provided on both sides (Refer Figure 20).

<u>Test 2</u> - Testing of retrofitted beams without any exposure

Out of the eight beams, 2 beams i.e. Beam 2 (without end-anchorage plates) and Beam 3 (provided with end-anchorage plates) were tested in the same way after attaching the loading plate just as on the control beam using universal testing machine with supports 800mm apart and the direct point load coming onto the mid-span. The beams were tested to failure in order to determine the ultimate load capacity of the beam and also percentage increment in load (if any) due to the strengthening procedure in both end-anchored and without end anchor CFRP strengthened beams. Figure 21 and 22 show images of Beam 2 and Beam 3 placed on UTM for testing.

Figure 20: Typical Beam strengthened with CFRP and end-anchor plates.

Figure 21: Beam 2 before test in position for testing on Universal Testing Machine.

Figure 22: Beam 3 before test in position for testing on Universal Testing Machine (UTM)

Extreme Exposure Conditions

In order to verify the effectiveness of the retrofitted members for resisting further corrosion in extreme exposures the remaining beams (Beam 4-9) were immersed in saline water for 30 days (Fig. 23). In order to subject the beam to the worst case scenario the beams were kept immersed during the night and were taken out daily in the day so as to maintain a daily wetting-drying cycle. Figures 24 a), b) and c) illustrate the Beams 4-9, after exposure to saline water for 30 wetting-drying cycles during 30 days.

Figure 23: Beams immersed in saline water tank

In figure 24 (a) it can be seen that there is a difference in the color of CFRP plate, it can be attributed to the presence of end anchor plates. As the plates were not protected against corrosion it may lead to degradation of CFRP. Thus, it is very important to provide a protective coating on the steel plates in order to prevent further corrosion.

Figure 24 a) : Beam 4 and Beam 7 after exposure in saline water tank for 30 days and giving daily one wetting-drying cycle.

Figure 24 b) : Beam 5 and Beam 8 after exposure in saline water tank for 30 days and giving daily one wetting-drying cycle.

Similar to figure 24 (a) in figure 24 (b) and (c) also the difference in the condition of CFRP can be seen between beam 5 and 8 and beam 6 and 9. This layer of corrosive film present on the CFRP is however superficial but may lead to severe degradation and galvanic corrosion may also come into play on longer duration of exposures. Thus, either the steel beam and the steel plates and bolts be covered with corrosion protective coating or some non-mechanical method of end anchorage be proposed.

Figure 24 c) : Beam 6 and Beam 9 after exposure in saline water tank for 30 days and giving daily one wetting-drying cycle.

<u>Test 3</u> - Testing of retrofitted beams after exposure to extreme conditions (for 30 days):

The beams (Beam 4-9) after exposure to extreme conditions of immersion in saline water for 30 days and daily drying wetting cycles were visually inspected for any nature of degradation due to exposure or absorption of water, any visible swelling or softening of adhesive or galvanic corrosion being initiated between CFRP and steel, and also further appearance of corrosion in steel. After the visual inspection, the loading plate was attached to each beam and the beams were tested on the Universal Testing Machine (UTM) similar to the other beams as described earlier. The beams were tested to failure in order to determine the ultimate load capacity of the beam. And the percentage change in load capacity compared to both the control beam and the beams tested without exposure.

Chapter 5: Results and Discussions

As proposed in the experimental program (chapter 4), testing of the Carbon Fiber Reinforced Polymer (CFRP) strengthened steel beams were carried out in three different stages:

- Test 1: Testing of the Control beam (Beam 1).
- Test 2: Testing of the retrofitted beam without any exposure (Beam 2 and Beam 3).
- Test 3: Testing of the retrofitted beam after exposure to extreme conditions for 30 days (Beam 4-9).

The results of the tests have been tabulated in Table 7 which shows the ultimate load carrying capacity of the beams and also the mode of failure and the increment of load capacity due to strengthening technique. For beam specifications and details refer Table 5 and 6.

Table 7: Results of Test 1, Test 2 and Test 3 showing failure modes and load capacity of CFRP strengthened beams.

Testing Stage	Beam No.	CFRP Plate End-Anchor	Dominant Failure Mode	Ultimate Load Carrying Capacity (KN)	Increment Of Load Capacity w.r.t Test 1	Change In Load Carrying Capacity w.r.t Test 2
Test 1	Beam 1	N/A	Yielding of steel	270	0	N/A
Test 2	Beam 2	N/A	Complete steel and adhesive interface debonding failure	305	12.96 %	0
	Beam 3	Provided	Intermediate steel and adhesive debonding failure	352	30.37 %	+15.4%
Test 3	Beam 4	N/A	CFRP tensile rupture	325	20.37 %	+6.56 %
	Beam 5	N/A	Complete steel and adhesive interface debonding failure	315	16.67 %	+3.27 %
	Beam 6	N/A	Complete steel and adhesive interface debonding failure	285	5.56 %	-6.56 %
	Beam 7	Provided	CFRP tensile rupture	367	35.92 %	+4.26 %
	Beam 8	Provided	CFRP tensile rupture	365	35.18 %	+3.69 %
	Beam 9	Provided	CFRP tensile rupture failure along with end anchorage failure	335	24.07 %	-5.07 %

One of the most important parameter in strengthening of any structural member is the increment of load capacity achieved in the strengthened beam compared to the non-strengthened member. Table 7 shows that the strength of a strengthened beam without end anchor plates was found to be improved by 5 to 20% and of that of a beam with end anchor plates was found to be improved by 24 to 36%. However, there has been seen variation in test results between all samples which may be due to varying adhesive thickness or voids left in adhesive layer or due to temp difference on the days of testing. All these factors can affect the ultimate load capacity of the members and may be the reason behind variations in results and may also govern the failure modes.

However, it is evident from the results (refer table 7) that the maximum increment in the load capacity was in case of end-anchored CFRP strengthened beams, which clearly indicates the efficiency of the technique used. Further, the modes of failure have also been summarized in Table 7 which can be co-related to the images in Figure 25 a) to h) showing failure modes of beams 2 to 9 after test.

In figure 25 (a) it can be clearly seen how the adhesive has debonded from the steel surface completely it may also be noted that the steel surface under CFRP has not been affected with corrosion further. However, rest of the surface of the tension flange after surface preparation due to the presence of a chemically active service has corroded even faster. It is therefore important to make sure that the steel surface uncovered by CFRP laminates must be protected by a coat of primer during the retrofit and subsequently a corrosion protection coating must be provided so as to avoid further corrosion of the steel.

Figure 25 a): Beam 2 after test showing mode of failure complete steel and adhesive interface debonding.

Figure 25 b): Enlarged views of Beam 3 after test showing mode of failure intermediate steel and adhesive debonding.

In figure 25 b) the mode of failure of the beam changed from complete debonding of the adhesive to intermediate debonding due to the introduction of end anchor plates. And thus an improvement of load carrying capacity by 15.4% from the beam without end anchor plates could be seen and an overall increase of 30.37% was indicated by the results.

Figure 25 c): Beam 4 after test showing mode of failure i.e. CFRP tensile rupture

In figure 25 c), the CFRP ruptured under stress and the splitting of CFRP can be clearly seen (as pointed by the arrow) beyond which the steel-adhesive interface failure occurred leading to the complete failure of the beam. Beam 4 was tested after 30 days of exposure to saline water and alternative dry-wetting cycles. However, no effect of corrosion could be seen in the adhesive such as swelling or moisture ingress etc. but the surface of CFRP on the exposed side was observed to be turned a little rough and may be on a longer period of exposure the clear initiation of corrosion could be seen on the surface of CFRP which had already started showing some signs of degradation. However, there was no loss of strength noted in the load capacity of this beam compared to Beam 2 testing under no exposure condition. However, a slight increase in the capacity can be seen, which may be due to various environmental effects or may also be attributed to longer curing period of the beams tested under test 3.

Figure 25 d): Beam 5 after test showing mode of failure complete steel and adhesive interface debonding failure.

Figure 25 e): Enlarged views of Beam 6 after test showing mode of failure complete steel and adhesive interface debonding failure.

In figure 25 (d) and 25 (e) the beam was found to fail by debonding of steel-adhesive interface followed by yielding of steel. However, the beams failed in the exact same manner there was a difference in the load carrying capacity of the two beams. Beam 5 showed an increase of 3.27 %

load capacity compared to Beam 2 of test 2 as opposed to Beam 6 that showed a decrease in load carrying capacity by about 6 % as compared to Beam 2 of test 2. This behavior may be due to a fault in or void infested in the adhesive layer during application which may have made the bond more susceptible to the extreme conditions causing the adhesive layer to become brittle under salt water action and thus causing premature debonding under a smaller load.

Figure 25 f): Beam 7 after test showing mode of failure CFRP tensile rupture.

Figure 25 g): Enlarged views of Beam 8 after test showing mode of failure CFRP tensile rupture.

The figures 25 (f) and (g) show the failure of Beams 7 and 8 by CFRP rupture and the splitting of the CFRP can be clearly seen in the images. The two beams failed in the exact same manner and showed similar load capacity with an improvement of 35.92 and 35.18 percent respectively.

However, the Beam 9 as can be seen in figure 25 (h) behaved differently, at first one of the CFRP end anchor plate failed due to failure of bolts in shear and subsequently the CFRP rupture was seen. The failure of the bolts may be due to degradation of strength due to excessive corrosion or may be due to additonal reaction on one side of the beam due to improper placement at the time of testing or excessive pulling force in the CFRP laminate.

Figure 25 h): Beam 9 after test showing mode of failure CFRP tensile rupture along with End Anchorage Failure

Figure 26: Image showing the failed bolt of end anchor arrangement of beam 9

In this chapter, the results of the experimental program were summarized and discussions of the results documented including possible reasons for the behavior of the strengthened beams in the experimental program. The conclusions on the experimental program and the study as a whole have been presented in the next chapter.

Chapter 6: Conclusions and Scope of Further Study

Interest of researchers in the field of retrofit of steel structures using CFRP bonding systems is gradually increasing and the topic is being focused upon worldwide. It can be seen that many researchers have carried out works studying different aspects of the behavior of the strengthened beams using this technique. An experimental program has been thus performed to study the efficiency of using CFRP bonding systems to corrosion-affected steel members.

The results of the experimental program clearly indicate that the method of retrofit of corrosion affected steel beams using CFRP laminates is remarkably effective and can be a viable technique used in the future to retrofit degraded steel bridge members. Both techniques of CFRP retrofit i.e. with and without end anchor plates showed significant increase in load capacity (as shown in Table 7), it can be said that the technique of strengthening the beams using CFRP with end-anchor plates is a much superior method.

The mode of failure changed from complete debonding, in most cases, in beams without end-anchorages to CFRP rupture and intermediate debonding in beams with end anchorages, which points out to a better bond behavior. Also, in this method of retrofit, using CFRP plates with end anchor plates, the behavior of the bond is very controlled and no sudden premature debonding has been noticed, which makes it a more stable and reliable technique of retrofit. It was observed that once the failure of the bond by debonding failure or tensile rupture of CFRP was seen the steel beam failed through yielding. However, it is still a very early stage to implement it in the field. As much work needs to be done in the study of behavior of this retrofit system (discussed in detail in the next chapter).

It is however seen that the effect of exposure to saline water and wetting-drying cycle for 30 days on the behavior of the strengthened beams was very insignificant in the short-term with drying-wetting cycles being very limited. But some visible degradation of the CFRP could be noticed although very insignificant and indistinct. The behavior of the strengthened beams in longer duration of exposure is of major concern and thus a more detailed research needs to be done. Also, after strengthening using CFRP it is very important that the exposed surfaces of the steel be protected using a corrosion resistant coating or paint. It is suggested that the CFRP be also protected using the same coating which would protect it from any ill-effects of exposure and also galvanic corrosion could be avoided and it will lead to a much more durable bond.

Due to promising performance and other advantages of bonding CFRP laminates to steel, the technique seems to be of importance to the retrofit of steel structures. However there is still a long way to go in understanding the behavior of this system properly. Further scope of study related to CFRP strengthening systems are as under:

- Further research is needed to develop low-cost carbon fibers /resin system with superior strength and stiffness characteristics as compared to conventional CFRP available.

- The durability of the bond on exposure to UV radiations and at different temperatures needs a more careful study with much longer durations of exposure. Further, the study using different adhesives in different environmental conditions need to be done and depending upon the behavior suitable adhesive for different range of temperature, exposure conditions etc. should be proposed.

- Guidelines for design and analysis of strengthening steel members using CFRP plates need to be proposed based upon sufficient research.

- The technique of retrofitting using CFRP plates with end-anchor steel plates and bolts need further study under fatigue loads. To understand the behavior better.

- Further research on the bonding techniques needs to be carried out in order to increase the capacity of retrofitted members by full-utilization of the strength of CFRP material.

- Development of non-metallic solutions for end-anchorage of CFRP plates.

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