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Retrofitting of Corrosion Effected Steel Bridge Members using Steel – CFRP Bonding Systems

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Abstract: Most steel bridges need strengthening either due to prolonged exposure to severe corrosive environments or increase of live load in due course of time or to improve fatigue performance. The current techniques of retrofit of steel structures need heavy equipment and skilled manpower for installation/implementation of them. The use of fibre reinforced polymer sheets for repair and strengthening of RCC structures is well recognised but the relevance of FRP composites to steel structures has been limited. The use of FRP materials for the repair and rehabilitation of steel members has abundant benefits over traditional method of retrofit by bolting or welding of steel plates. Carbon Fibre Reinforced Polymers (CFRPs) have been preferred over other FRP materials for strengthening of steel structures due to higher stiffness of the former. The emergence of high modulus CFRP plates, which have elastic modulus higher than or comparable to that of steel is helpful in considerable load transfer from steel beams, prior to the yielding of steel. In CFRP strengthened structure, the behaviour of bonding joints between CFRP and steel plays a very important role. In CFRP-steel bonding, the weak link is epoxy adhesive.

In the present study, feasibility and effectiveness of adhesively bonded Carbon Fibre Reinforced Polymer (CFRP) sheets in retrofitting of steel member, with and without end anchor plates, affected by corrosion to be used in bridges, has been determined along with short term effect of further corrosion in extreme exposures.

Keywords: Control beams, CFRP strengthened beams, ultimate load, extreme exposure, wetting-drying cycles and epoxy adhesive.

INTRODUCTION

Bridges play a significant role in railway infrastructure throughout the world. With the introduction of trains of progressively heavier axle load and high horse power locomotives, bridges are being subjected to, much greater loads than their original design loads considered. Indian Railways now have the herculean task of investigating and retrofitting of aged bridges across the country. It is great challenge considering Indian Railways for retrofitting of more than 36000 bridges that are over 100 years old [1].

Hence, the main challenge for bridge engineers of the date is to assess the strength and capability of the existing bridges and retrofit them to enable them to suffice for enhanced loading. The Railways has spent nearly Rs. 6000 Crore on the repair of bridges in the past 10 years [1] that amount would have to be multiplied many 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 their replacement. In addition, retrofitting comparably takes less implementation time and greatly reduces service disruption time.

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

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

In the present study, feasibility and effectiveness of Carbon fibre reinforced polymer (CFRP) bonding on corrosion affected steel members (a technique of retrofitting) has been focussed. In this regard, various aspects like structural behaviour of strengthened beams, bond and force transfer mechanism between steel and CFRP, modes of failures of CFRP-steel joints and durability of retrofitted systems particularly providing solution to eliminate galvanic corrosion have been considered.

Various Other Methods of Retrofit

1. *Retrofitting of steel bridges for fatigue considerations*

The cause of fatigue in steel bridges may be categorised as:

- Built-in welding defects incorporated at the time of fabrication,
- Assumed incorrect structural details of low fatigue strength,
- Unexpected deformations and stresses occurred at member joints and
- Structure behaved in an unexpected manner due to vibration.

Repair methods include: (a) crack removal, (b) re-welding, (c) surface treatment such as TIG dressing, (d) post welding surface treatment in addition to re-welding, (e) provision of bolted splice, (f) F-shape improving methods, (g) G-stop hole,(h) connection detail modification and (i) strengthening by steel place splicing.

2. *Concrete covering*

This method is the same as shotcreting. Also concrete covering helps to avoid buckling of the member.

3. *Steel stitching*

Steel stitching is a common technique for retrofitting of steel bridge girders. Some advantages of this method are listed here: (1) feasible at any location, (2) minimal traffic interruption and (3) it is economical and cost effective technique.

4. *Seismic retrofitting technique*

This repair method includes: (a) provision of shear links: built up shear links fabricated using plates of varied grades of steel, (b) provision of unbounded brace for controlled rocking approach to seismic resistance in steel truss bridge piers, (c) seismic isolation methods, (d) 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 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.

Advantages of using CFRP Bonding System over Conventional Methods

Most of existing methods of retrofit either use welding or bolting of steel plates or sections to the existing members. However, constructability and durability disadvantages are related with these methods. Welding of additional plate has 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 reduction in cross section area due to drilling, which in turn leads to requirement of additional strengthening material. The corrosion performance of the bolted retrofits is also affected due to accumulation of debris and moisture around edges of bolted connections and plates. Constructability disadvantages are requirement of heavy lifting machinery for steel plates, addition of considerable more dead weight to the structure, future maintenance cost, time consuming and cost effective.

Thus, the need for adopting a 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 fibre reinforced polymers (FRP). The intrinsic high strength and stiffness of steel makes it more challenging material to strengthen compared to concrete. When steel is to be retrofitted using a material having lower Young's modulus, load/ transfer sharing of strengthening material will only be substantial after the steel has completely yielded. Therefore, material like glass fibre reinforced polymer (GFRP) that have relatively low inherent tensile modulus, are rendered less desirable for retrofitting of 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 of labour cost, 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 cost of the strengthening project can be greatly optimized [2].

The use of CFRP and GFRP bonding systems for retrofit of concrete structures has evidently been quiet successful. Its effectiveness has also been verified for a number of retrofitted mechanisms and is becoming more widely accepted practically. These are used in the form of plates or sheet bonded to the concrete surface for flexure 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 researchers' attention and is being studied, but yet many aspects need consideration.

LITERATURE REVIEW

A lot of retrofitting work using CFRP has been carried out on strengthening of reinforced concrete (RC) structures but retrofitting of steel structures using CFRP has also attracted many researchers. A brief review of their work is given in this section.

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 aluminium and steel aircraft structures and also in marine applications of large ships and submarines. [3-7]. 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.

Teng et. al.[8] 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 possible failure modes are (Fig. 1 and 2): (1) in-plane bending failure, (2) lateral buckling, (3) rupture of laminate at mid span when maximum axial stress in the laminate reaches its ultimate strength, (4) plate end de-bonding due to high localised interfacial stresses and peeling stresses in the vicinity of plate ends or due to maximum shear in the bond-line at the end of the plate [9], (5) intermediate de-bonding due to local cracking or yielding at a distance from the end of the plates somewhere in the middle, (6) mid splitting of CFRP under point load, (7) inter-laminar shear failure at the end of laminate [9], (8) local buckling of compression flange and (9) local buckling of web.

De-bonding failures are the most challenging issue in the flexural strengthening of steel beams as the adhesive is the weak link, most de-bonding failure depend upon the adhesive properties. Buyukozturk et. al. [10] defined de-bonding failure as the significant reduction in member capacity of a retrofitted system due to instigation or transmission of de-bonding. De-bonding in FRP strengthened systems occurs in the regions of high stress concentrations (i.e. due to material discontinuity or presence of cracks) depending upon elastic and strength characteristics of retrofit systems, the constituent materials and their interface properties. They reviewed the progress of understanding of de-bonding problems in RC and steel members retrofitted using FRP.

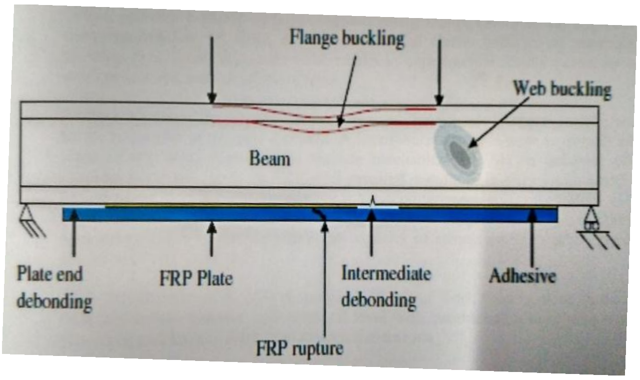


Fig. 1 different failure modes of strengthened steel beams

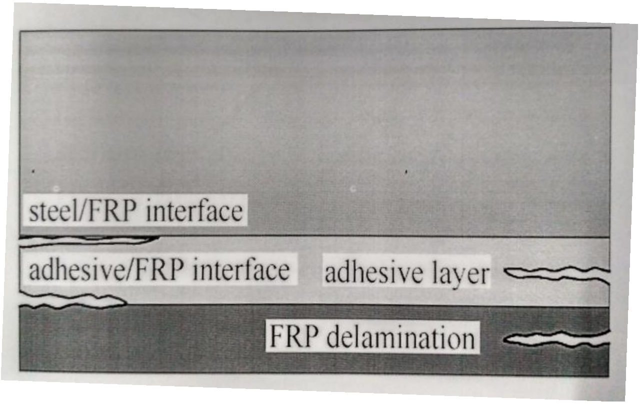
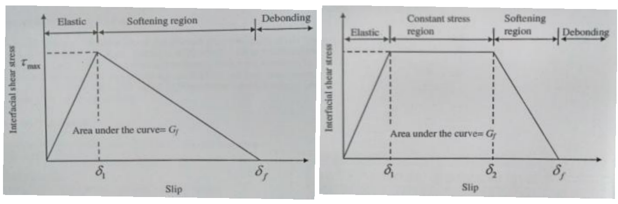


Fig. 2 de-bonding failure planes

Modelling of de-bonding problems using strength approach [11, 12 and 10] involves characterisation based on elastic properties of materials. In this approach, prediction of de-bonding failure is made by calculating interfacial or bond stress distribution in FRP strengthened systems and further comparing them with ultimate strength of the materials based on which the probable mechanism and load level of de-bonding failure is suggested. Many researchers have studied different methods of predicting de-bonding failure based on this approach with varied assumptions of shear, normal stresses and elastic behaviour of materials. The solution provided close results to experimental results except for a very small zone at the ends of adhesive layer.

A precise bond-slip model for FRP to steel interface is of elemental importance to properly understand, characterise and model the true behaviour of FRP strengthened steel members. As bond slip illustrates the relationship between local interfacial shear stress and relative slip between the two adherents which can be obtained experimentally though tests carried on bonded joints. [8]. Single lap pull test has been found to be the most suitable method for studying full range behaviour of FRP-steel bonded joints.

Fernando [13] conducted a series of single-lap pull tests on FRP-steel bonded joints using various types of adhesives and concluded that a two branch slip model shown in Fig. 3(a) is only suitable to predict behaviour 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 shown in Fig. 3(b), based on experimental test results.



(a) linear adhesive (b) non-linear adhesive

Fig. 3 generalised bond - slip curves

The study [10] shows that the performance of strengthened structures under cyclic loading may fall below that under monotonically increased loading in RCC retrofits due to brittle de-bonding failures, based on strengthening parameters and anchorage conditions. However, in steel members retrofitted using FRP composites not much effect has been noticed under low amplitude fatigue loading but results under high amplitude cyclic loading are not available and need careful consideration.

De-bonding in FRP strengthened materials can take place (Fig. 2) [10] at: (1) steel/ CFRP interface, (2) between adhesive layers (also referred as cohesive failure), (3) adhesive/CFRP interface and (4) CFRP de-lamination.

Further, Emrani et. al. [14] suggested two types of failure modes occurring at the end of CFRP plates in a strengthened system, namely inter laminar shear failure (de-lamination) at the end of laminates and de-bonding failure due to maximum shear in the bond line at the laminate ends.

Xia and Teng [15] 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, de-bonding happens by plate de-lamination due to brittle failure mode instead of de-bonding between adhesive layers as in case of thin adhesive layer. Also, adhesive 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 bevelling the ends as shown in Fig. 4, CFRP plats decrease the peeling stress notable [16]. Further, adding steel plate anchors or clamp at the end of CFRP plates improves the behaviour of the bond [9]. Also, good surface preparation and elimination of any kind of contaminants from the surface of both steel and CFRP plays an important role in improving the bond behaviour of the strengthened system [16].

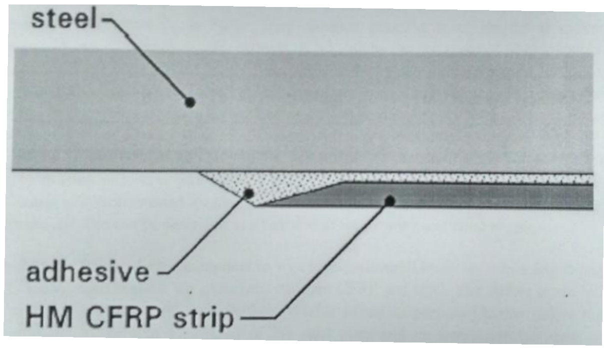


Fig. 4 configuration of reverse tapered end

Sen et. al. [17] also designed steel clamps as end anchors 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. [18] recommended wrapping of GFRP sheets around the bottom (tension) flange and a part of the web perpendicularly to the longitudinal CFRP sheets bonded for strengthening of the member. These sheets were proposed to be attached along the whole length of the girder to avoid de-lamination of CFRP sheets.

Bocciarelli et. al. [19] conducted numerical as well as experimental study on de-bonding 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 adhesive produced higher failure loads but, significantly lower than yield load of the steel plate and no interaction between steel plasticity and interface be-bonding 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.

Hart-Smith [20] developed a theoretical model to predict the strength of steel-CFRP double strap joints at room temperature by adopting a 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 not 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.

Zubaidy et. al. [21] 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 is least influenced by the impact load. The failure mode also changed from CFRP-adhesive interface failure to CFRP de-lamination failure under impact lading due to shear strength enhancement of epoxy adhesive in the joint.

Gillespi et. al. [22] tested two strengthened girders under fatigue loading for 10 million cycles at the estimated stress range in the field. No de-bonding of any kind was found during periodical monitoring and inspection carried out throughout the 10 million cycles.

Deng and Lee [23] 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, S-N curve was developed from the test results for further use in prediction of behaviour and design of retrofitting of different sizes with same adhesive.

EXPERIMENTAL INVESTIGATION

Materials

Mild steel

Steel I-section, plate sections used as stiffeners, end anchor plates, pack plates and loading plates are of Grade BR – E250 (Fe 410W) as per IS: 2062-2011 [24] and IS: 800-2007 [25] has been used. Properties of the material are shown in Table 1.

Table 1 Physical/ mechanical properties of steel

grade designation	yield strength (MPa)	ultimate strength (MPa)	limiting elongation (%)	elastic modulus (MPa)
E250 (Fe410W)	250	410	23	$2x10^5$

Carbon fibre reinforced polymer (CFRP)

Carbon fibre reinforced polymer is a composite material made of two components i.e. carbon fibres and resins. Continuous fibres are set in polymeric resin matrix such that the resultant material has distinct non-reactive materials bonded together and possesses the combined properties of both the materials. Fibres are harder and stronger and the resin matrix works as shielding layer that hold reinforcement collectively and helps in transfer of forces between them. Moreover, the mechanical and material properties of the composite are superior to the constituent materials. Carbon fibres are strong and have stable bond at atomic level, higher rigidity, high strength, resistance to many chemical aggressive environments, low density and good availability. However, Carbon material is brittle.

In this work, Sika CarboDur S: pultruded carbon fibre plates (Fig. 5) have been used for strengthening. These plates are having tensile strength, ultimate strain and elastic modulus of 2800 MPa, 1.7% and 165 GPa respectively. CFRP sheets are available in 50 mm width and 1.4 mm thickness.

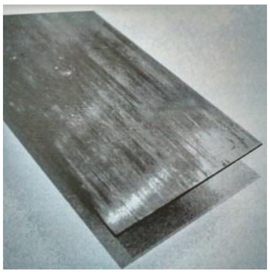


Fig. 5 CFRP Laminate

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 characteristics of high strength, low shrinkage and excellent adhesion to various structural materials, effective electrical insulation, chemical and solvent resistant, low cost and low toxicity. They are chemically compatible with most structural materials and get wet readily, thus suitable for composite applications. When choosing an epoxy resin, consideration is usually given to tensile strength, elastic modulus and strain.

In this project, Sikadur-30LP (IN), adhesive for bonding has been used to bond CFRP to steel surface. This epoxy is a thixotropic, structural two parts adhesive, based on a combination of epoxy resins. Tensile strength, compressive strength, shear strength, flexural strength, tensile bond strength and elastic modulus of the epoxy is 15-18 MPa, more than 90 MPa, 10 MPa, more than 25 MPa, 18 MPa and 10 GPa respectively.

Experimental Program

In this study, all nine beams were of same cross-section and all were artificially corroded only on bottom flange i.e. tension flange and then retrofitted using CFRP bonding systems with and without end-anchors and the beams thus prepared, were tested under three point load test arrangement. 10 mm diameter bolts were used for end anchors. The study was planned to verify the effectiveness of the method of retrofit for corrosion affected beams using CFRP and to compare the results of those retrofit with CFRP alone and CFRP with steel plate end anchors. Furthermore, some of the strengthened members were exposed to extreme conditions of exposure i.e. wetting - drying cycles of saline water to study the durability of the technique.

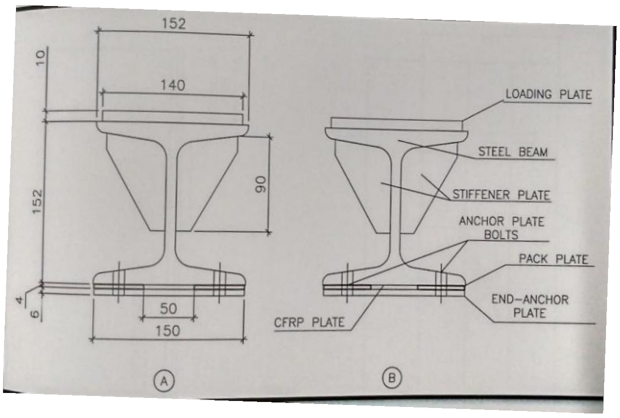


Fig. 6(A) dimensioning details of strengthened beams with end anchorage
6(B) component details of strengthened beams with end anchorage

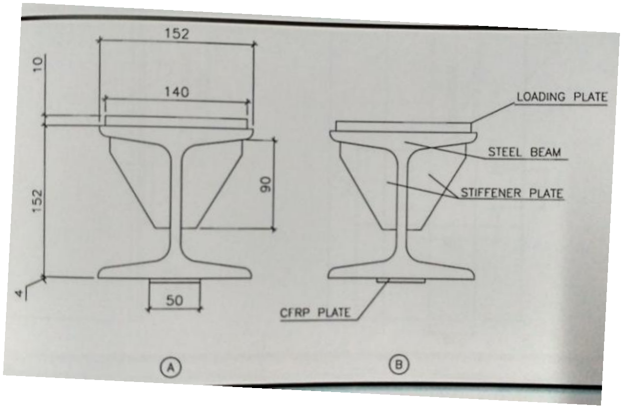


Fig. 7(A) dimensioning details of strengthened beams without end anchorage
7(B) component details of strengthened beams without end anchorage

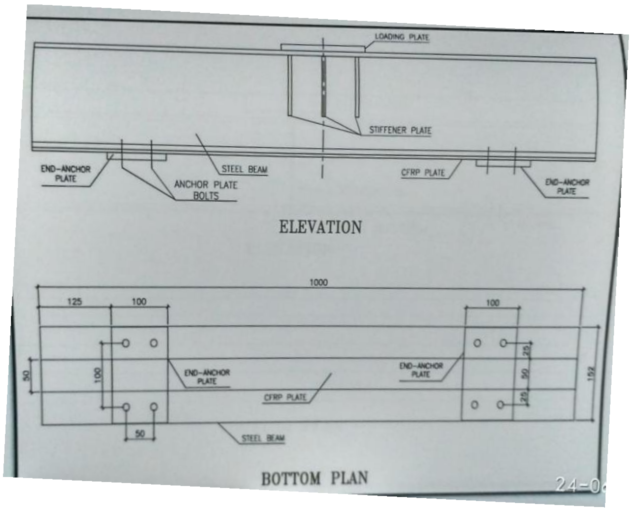


Fig. 8 details of strengthened beams with end anchorage

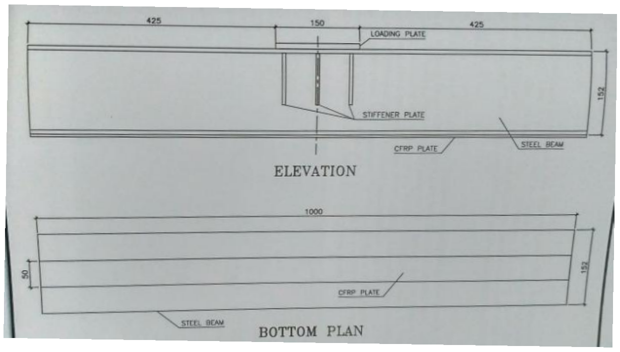


Fig. 9 details of strengthened beams without end anchorage

Details of test beams

Cross section details of I-section and its accessories are given in Table 2 while details of retrofit scheme adopted on various beams is given in Table 3. Fig. 6 and 7 show sectional view of a test beam along with accessories details of beams with and without end anchor respectively. Fig. 8 and 9 give elevation and plan of a strengthened beam showing the arrangement of retrofit and location of end anchor and CFRP laminates.

Table 2 Details of I-section and its accessories

Description of I-section		UC 152 x 152 x 37
Flange width (mm)		152
Overall height (mm)		152
Flange thickness (mm)		11.5
Web thickness (mm)		8
CFRP width (mm)		50
End anchor plate details	Length (mm)	150
	Width (mm)	100
	Thickness (mm)	6
Stiffener details	Nos. Provided	6
	Height (mm)	90
	Thickness (mm)	6
Loading plate details	Length (mm)	150
	Width (mm)	140
	Thickness (mm)	10

Table 3 Details of retrofit scheme

Mark no.	CFRP bonded	End anchor used	No. of bolts used	Exposure condition after strengthening
Beam 1	No	No	-	No exposure
Beam 2	Yes	No	-	No exposure
Beam 3	Yes	Yes	4	No exposure
Beam 4	Yes	No	-	30 wetting-drying cycle with saline water
Beam 5	Yes	No	-	30 wetting-drying cycle with saline water
Beam 6	Yes	No	-	30 wetting-drying cycle with saline water
Beam 7	Yes	Yes	4	30 wetting-drying cycle with saline water
Beam 8	Yes	Yes	4	30 wetting-drying cycle with saline water
Beam 9	Yes	Yes	4	30 wetting-drying cycle with saline water

Steel end anchor plates

Providing end anchors and clamps is an effective way to avoid premature de-bonding. Thus, steel plates have been provided to act as an anchor at both ends of the CFRP laminates as shown in Fig. 6 and 8. Dimensions of the plates are shown in Table 2. The plates have been attached to the bottom flange with the help of bolts. The width of 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 deduction of sectional area and also the carbon fibres are thus not exposed directly to the steel due to punching or drilling. Also, pack plates (Fig. 6) have been provided between end anchor plate and bottom flange plate outside 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.

Bolts

Bolts were provided to hold the end anchor steel plate to the bottom flange plates and also help to increase the bonding strength between the adhesive, CFRP and the steel beams. Four bolts have been provided in each anchor plate. The location and arrangement scheme of bolt holes is given in Fig. 6 and 8.

Stiffener Plates

Stiffener plates were provided to render stiffness to the compression flange and web against local buckling failure (Fig. 1) 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 is given in Fig. 6, 7, 8 and 9.

Loading Plate

A plate of thickness comparable to that of flange thickness of the beam has been provided at the mid span i.e. at application of point load in order to ensure distribution of load to a broader area and to avoid local failure of web plate.

Table 3 gives the details of the beams used in the project along with the condition of beams at different stages of the experiment i.e. condition of test prior to strengthening operation and the exposure condition of the beams subsequent to strengthening activity before conducting the test. Additionally, the details of retrofit scheme i.e. CFRP, end anchors and details of bolted connections have also been provided.

Preparation of test beams

The holes were made first in the anchor plates and bottom flange of the I-sections in the end region of CFRP sheets. The holes were also made in loading plate and top flanges of I-sections in mid span region. Stiffener plates were fabricated and welded at desired locations.

Corrosion process

Before adoption of strengthening, corrosion of bottom flange of all beams was necessary so that there could be significant loss in the section and its strength. The beams were artificially corroded by exposing them to the mixture which was formed by freshly mixing concentrate nitric acid and hydrochloric acid optimally in a volume ratio of 1:3. The thickness of bottom flange of the section was reduced from 11.5 mm to 4 mm at mid span by ponding of concentrate acid at the desired location. All 9 beams of the project were corroded.

Preparation of the strengthened beams

Eight beams were strengthened prior to their load test. The surface of steel beams was prepared by first brushing in order to remove rust and then blast cleaned so that surface could be free from grease, oil, rust and any other contaminations which would prevent adhesion. Thereafter, surface was cleaned using ethanol to ensure it free from any contamination. The maximum wait after blast cleaning and before attaching the CFRP sheet was 48 hours. But, the chosen epoxy does not require any priming. So to avoid reoccurrence of corrosive layer, bonding process was started immediately after drying of steel surface.

CFRP sheets were cut to length using a rotary disc cutter by putting tape on the location of cuts in order to prevent excessive dust from generation. 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 sheets were cleaned using a white cloth and ethanol until there were no trace of black dust appearing on the cloth. The sheets were left dry before application of adhesive.

Components A and B of the two part epoxy system 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 to avoid entrapping of air using a spatula like tool for about 3 minutes till a homogenous mix with uniform grey colour in appearance was achieved. The pot life of an adhesive begins when resin and hardener are mixed. It is shorter at higher temperature and longer at lower temperature. Larger the quantity of material mixed together at one time, the shorter the pot life. Thus, the mix obtained was immediately used.

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 about 1 – 2 mm along the centreline on the top surface of the bottom flange of the steel I-section at pre marked location. Immediately after, CFRP sheet was placed onto the adhesive and pressed using a roller unit, material is forced out on both sides of the sheet as shown in Fig. 10. The freshly bonded system should not be disturbed for at least 24 hours and also any vibration should normally be kept at a minimum during the curing period of adhesive. The full design strength of epoxy adhesive was reached at 7days at 20^0C. Thus without any further disturbance, the beams were left to be cured for a period of 10 days to ensure proper curing.

Fig. 10 application of CFRP sheets on steel sections without end anchorage

The installed sheets were checked for air pockets/ voids within the adhesive layer or at bond interface by tapping of a metal bar.

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

After the installation of CFRP sheets, the end anchor plates were installed on 4 beams namely Beam 3, Beam 7, Beam 8 and Beam 9 using bolts and at the location shown in Fig. 6 and 8. Nuts were tightened appropriately using a spanner and washers were used under the nuts.

Extreme exposure condition

In order to verify the effectiveness of the retrofitted beams for resisting further corrosion in extreme exposures, the remaining beams (i.e. Beams 4 – 9) were subjected to wetting-drying cycle in saline water for 30 days (Fig. 11). To make this effect most aggressive, the beams were kept immersed during night and were taken out daily during the day so as to maintain a daily wetting-drying cycle.

Fig. 11 beams immersed in saline water tank

Test Results

Control Beam

After corrosion, one of the 9 beams (Beam 1) was tested without any kind of retrofit, surface preparation or repair to act as control beam for comparing the results with strengthened beams. The loading plate was attached to the top flange using location bolts. The beam was tested on a setup based on three point bending test using UTM, with the end supports at 800mm apart and a point load applied at mid span. One roller and other hinged supports carried reactions and load was applied at middle point, therefore loading state was three incremental bending point loads. The beam was tested up to failure in order to determine the ultimate load carrying capacity of the beam.

Testing of strengthened beams without any exposure to saline water

Out of 8 strengthened beams, 2 beams i.e. Beam 2 and Beam 3 (one with and other without end-anchor plates) were tested in the same way after attaching the loading plate just as on control beam using UTM. The beams were tested up to failure in order to determine their ultimate load carrying capacity of the beam. The effectiveness of the strengthening systems (i.e. with and without end-anchor plates) was also computed for addition load in terms of percent of failure load of control beam.

Testing of retrofitted beams after exposure to extreme condition

Beams 4-9 after exposing to extreme condition were visually inspected for any degradation like swelling or softening of adhesive or galvanic corrosion initiated between CFRP and steel and further appearance of corrosion in steel due to exposure. After inspection, loading plate was attached to each beam and thereafter they were load tested on UTM in similar manner to other beams as already explained. The beams were load tested up to failure in order to determine the ultimate load carrying capacity of the beam. The effectiveness of the strengthening systems (i.e. with and without end-anchor plates) after exposing them to extreme condition was also computed for addition load in terms of percent of failure load of control beam.

Results and Discussion

As per experimental program, load testing of CFRP strengthened beams along with control beam were carried out in three different stages:

1. load test of control beam (i.e. Beam 1),
2. load test of retrofitted beams without any extreme exposure (i.e. Beams 2 and 3) and
3. load test of retrofitted beams after extreme exposure (i.e. Beams 4 - 9).

Test results of all 9 beams mentioning their ultimate failure load and modes of failure are given in Table 4.

Table 4 Load test results of the beams

beam no.	CFRP sheet	end anchor	ultimate failure load (kN)	failure mode
Beam 1	No	No	270	yielding of steel
Beam 2	Yes	No	305	complete de-bonding between steel and adhesive
Beam 3	Yes	Yes	352	Intermediate de-bonding between steel and adhesive
Beam 4	Yes	No	325	CFRP tensile rupture
Beam 5	Yes	No	315	complete de-bonding between steel and adhesive
Beam 6	Yes	No	285	complete de-bonding between steel and adhesive
Beam 7	Yes	Yes	367	CFRP tensile rupture
Beam 8	Yes	Yes	365	CFRP tensile rupture
Beam 9	Yes	Yes	335	CFRP tensile rupture along with end anchorage failure

One of the most parameter in strengthening is the enhancement in load carrying capacity of strengthened beam with respect to control beam. Table 4 shows that the strength of strengthened beam without end anchor was found to improve from 5 to 20% and that of with end anchor was found to improve by 24 to 36%. However, the variation in test results has been seen between all specimens, which may be due to varying adhesive thickness or voids left in adhesive layer or due to temperature difference on the days of testing. All these factors affect the load carrying capacity of the strengthened member and may be causing variation in test results and failure modes.

However, it is clear from the results that maximum enhancement of load carrying capacity is in end anchored CFRP strengthened beams, which indicates more efficient technique for strengthening.
In Fig. 12, it can be clearly seen how adhesive has de-bonded from steel surface completely, but steel surface under CFRP has not been affected with corrosion further. However rest of the surface of tension flange after surface preparation due to the presence of a chemically active service has corroded even faster. This reveals that steel surface covered by CFRP sheet has been protected by CFRP sheet till the de-bonding has taken place and to avoid further corrosion, corrosion protection coating must be applied.

Fig. 12 complete steel and adhesive interface de-bonding

In Beam 3, mode of failure of the strengthened beam has been changed from complete de-bonding in Beam 2 to intermediate de-bonding due to the introduction of end anchor plates. And thus an enhancement of load carrying capacity of 15.4 % has been noted in the beam with end anchor plates with respect to the beam without end anchor plates and an overall enhancement of 30.37% in load carrying capacity of the Beam 3.

In Beam 4, CFRP ruptured under stress and splitting of CFRP was clearly noticed, beyond which steel-adhesive interface failure occurred, leading to complete failure of the beam. Beam 4 was tested after subjecting it to 30 alternate wetting-drying cycles under saline water. However, no effect of corrosion (like swelling or moisture ingress etc.) could be seen in the adhesive, but the surface of CFRP on the exposed side was observed a little rough (might be a sign of degradation), which may give clear initiation of corrosion on CFRP sheet in longer period of exposure. As there was no loss of strength noted in the load carrying capacity of this beam compared to Beam 2 tested under no exposure condition. However, a slight increase in load carrying capacity can be seen due to sample variation.

Beams 5 and 6 were failed by de-bonding of steel adhesive interface followed by yielding of steel. These beams were failed exactly in the same manner and there was a difference in their load carrying capacity of the two beams. Beam 5 showed an increase of 3.3% of load carrying capacity compared to Beam 2 while Beam 6 showed a reduction in load carrying capacity by about 6% compared to Beam 2. This variation in load carrying capacity of Beam 5 and Beam 6 is also sample variation.

The failure of Beam 7 and Beam 8 was noticed by CFRP rupture and splitting of CFRP. The two beams failed in exactly the same manner and showed almost similar load carrying capacity, with an improvement of 35.92% and 35.18% respectively. However Beam 9 behaved differently, at first one of the CFRP end anchor plate failed due to failure of bolts in shear and subsequently CFRP rupture was seen. The failure of bolts may be due to degradation of strength caused by excessive corrosion.

CONCLUSIONS

1. Retrofit of corrosion affected steel beams using CFRP laminates is remarkably effective and can be a viable technique in future to retrofit degraded steel members.
2. Both techniques of CFRP retrofit with and without end anchor plates demonstrated significant increase in load carrying capacity but CFRP with end anchor plates is much superior method.
3. De-bonding was notice as mode of failure for beams without end anchor while the beams with end anchor plates were failed by CFRP rupture and intermediate de-bonding (a better bond behaviour) in beams. In this type, behaviour of bond was very controlled and no sudden premature de-bonding was noticed, due to which latter may be considered a more stable and reliable technique of retrofit.
4. Failure of the CFRP - steel bond or tensile rupture of CFRP was succeeding to failure of steel beam by yielding.

5. Short term effect of saline water under wetting – drying cycles for 30 days has some visible degradation (insignificant) on CFRP strengthened beams.

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