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RETROFITTING AND REHABILITATION OF RAILWAY BRIDGE USING CFRP

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Abstract. *This paper aims at the topic of strengthening of existing old deteriorated bridges with the help of case study of retrofitting of half century old Indian railway bridge. The structure has monitored before and after strengthening. The two response parameter has considered for monitoring i.e. deflection and frequency, because deterioration and cracks affect the stiffness of structure and hence these responses. The result shows significant change in response after strengthening. The two type of strengthening i.e. flexural and shear strengthening of bridge's girder with the help of CFRP System has discussed.*

1 INTRODUCTION

Deterioration of ageing bridges has been well noted worldwide. Retrofit of deteriorated infrastructure has become a major challenge for governments in developed and under developing countries in the last decades. Currently there exists a multitude of options for viable methods to repair structurally deficient reinforced concrete and prestressed concrete bridge components. The use of externally bonded carbon fibre reinforcing polymers (CFRP) to repair bridge girders has proven to have numerous advantages in comparison to traditional methods. CFRP has a high strength to weight ratio, is resistant to chemicals, and the repair methods are usually inexpensively and rapidly applicable in the field with little to no disturbance to traffic; the repairs also maintain the over height clearance and original configuration of the structure. In response, there has been an escalating world-wide tendency to select Fibre Reinforced Polymer (FRP) composite retrofit systems as an alternative to traditional bridge rehabilitation schemes. Yet, in spite of their benefits, the use of externally bonded FRP systems is hampered by the lack of nationally accepted design specifications for their use in the repair and strengthening of concrete bridge elements. The current national specifications for designing externally bonded CFRP laminates is the ACI 440.2R-08[1]. This document provides a large array of guidelines for strengthening structural members. However, it does indicate some limitations in its contents and refers to durability and de-bonding behaviours as "areas that still require research". It continues to state specifically that "more accurate methods of predicting de-bonding are still needed" (ACI Committee 440-2008). Similarly, this document also does not provide deflection provisions specific for FRP-strengthened RC beams but instead refers the designer to ACI 318-99 which does not address post yielding deflections for strengthened beams. Accordingly, several design codes were developed to standardize the bridge strengthening process using FRP systems. This paper covers the effectiveness of external FRP strengthening of bridge elements such as I-girders with help of case study.

2 DISCRIPTION OF CASE STUDY

Western Railway has proposed rehabilitation of PSC I girders on Bridge of Godhra-Ratlam section of Ratlam Division of Western Railway, India. The Bridge has been observed to require immediate strengthening. The bridges were constructed during 1958-60 and as such no detailed design & drawings are available. The bridges have composite PSC I girders supported on neoprene bearing. The main reason for retrofitting of the PSC I girders on the bridges was due to development of cracks on the girders. As the cracks are propagating with time strengthening of these girders are required immediately to arrest further deterioration.

Sr. No.	Details	
1	Type of superstructure	PSC-I girders
2	Span	1 x 18.29 m
3	Clear span	18.15 m
4	Effective span	19.17 m
5	Overall length of girder	19.67 m
6	Overall length of deck slab	20.20 m
7	Diaphragm details	6 (4 Intermediate and 2 End) size-1880x1450x275 mm
8	Name of river	Nakdi river
9	Year of casting and launching of Bridge	1958-60
10	Total weight of girder (per span)	145 T (Girders +Deck slab+Diaphragms)
11	Ballast cushion	300 mm
12	Details of track	60 Kg, Sleeper-PSC
13	Bank height in approaches	10.5 m
14	Dimensional details of girder cross section	Overall depth-2130 mm Width of Top and bottom bulb-620,Thickness of web-310 mm, Depth of web-1435 mm
15	Thickness and width of deck slab	Thickness-150 mm,width-4300 mm
16	Substructure type and material type	Stone masonry in cement mortar, gravity type substructure

Table 1: Dimension detail of PSC I girder bridge.

3 PRELIMINARY STRUCTURAL ASESSMENT

The overall evaluation included a thorough field inspection, measurement of different load response parameter such as deflection and frequency for static and dynamic load and a structural capacity analysis. Existing construction and operational documents for the bridges were reviewed, including the design drawings, project specifications, as-built information, and past repair documentation. Assuming that the PSC girder beams behave as simple reinforced girder beam with no prestress. As the existing reinforcement details are not known, an analysis is done on the girder to find the reinforcement required for the section. Based on this reinforcement and section details the capacity is found out.

3.1 Visual inspection

Discoloration due to deterioration with time had seen in the overall structure. Spalling/ delamination of concrete had been observed in diaphragm bottom. Cracks at the girder bottom at bearing locations had been observed. Termite attack had observed in the girders. Surface deterioration had been observed throughout the structure. Severe spalling and corroded reinforcement exposure had been observed at the bottom slab. This may be attributed to the carbonation taken place in the structure. There were vertical cracks at multiple location in PSC I girder. As these cracks were structural cracks, speed restriction brought up to 45 kmph. Again during close and regular monitoring of these cracks it has been reveal that these cracks were active and progressing further, the speed limit reduced to 20 kmph (Agarwal S. N., et al 2017)[2]. Moreover, the immediate repair and strengthening of girder till they replaced has been suggested by RSDO. It is expected that such a distress will be reflected through a change of stiffness of the girder. As the stiffness of the girder changes, the natural frequency will also change. Hence, it may be possible to monitor the health of a bridge girder in a non-destructive in-situ manner by measurements of the natural frequency at regular intervals. It has decided to monitored natural frequency and deflection of the bridge before and after strengthening.

Figure 1. Cover delamination and corroded reinforcement exposure observed in deck slab bottom

Figure 2. Severe cracks seen in girder

Figure 3. Structural health monitoring of Ratlam railway bridge with help of sensor

3.2 Monitoring for deflection and frequency

The bridge girder has monitored for deflection and frequency with the help of LVDT and accelerometer respectively. The frequency has measured for dynamic forces and deflection has measured for static as well as dynamic forces. The un-cracked and undamaged girder has also monitored for reference. WAG-7 Loco was used for static as well as dynamic loading purpose. The vibration has measured just after passing the loco when girders were vibrating freely.

4 FRP STRENGTHENING OF PSC I GIRDER OF RAILWAY BRIDGE

The strengthening design for the PSC I girders was proposed with the help of Prestressed carbon laminates and carbon fiber wrapping. According to ACI 440.2R-08 the following assumptions are made in calculating the flexural resistance of a section strengthened with an externally applied FRP system.

4.1 Assumptions.

- Design calculations are based on the dimensions, internal reinforcing steel arrangement, and material properties of the existing member being strengthened;
- The strains in the steel reinforcement and concrete are directly proportional to the distance from the neutral axis. That is, a plane section before loading remains plane after loading;
- There is no relative slip between external FRP reinforcement and the concrete;
- The shear deformation within the adhesive layer is neglected because the adhesive layer is very thin with slight variations in its thickness;
- The maximum usable compressive strain in the concrete is 0.003;
- The tensile strength of concrete is neglected; and
- The FRP reinforcement has a linear elastic stress-strain relationship to failure.
- In addition to the basic assumptions for concrete the following assumptions are made in calculating the flexural resistance of a prestressed section strengthened with an externally applied FRP system.
- Strain compatibility can be used to determine strain in the externally bonded FRP, strain in the non-prestressed steel reinforcement, and the strain or strain change in the Prestressing steel.
- Additional flexural failure mode controlled by Prestressing steel rupture should be investigated.

- For cases where the Prestressing steel is draped, several sections along the span of the member should be evaluated to verify strength requirements; and
- The initial strain level of the concrete substrate ε_{bi} should be calculated and excluded from the effective strain in the FRP. The initial strain can be determined from an elastic analysis of the existing member, considering all loads that will be on the member at the time of FRP installation. Analysis should be based on the actual condition of the member (cracked or un-cracked section) to determine the substrate initial strain level.

4.2 Failure mode.

The flexural strength of a section depends on the controlling failure mode. The following flexural failure modes should be investigated for an FRP-strengthened section.

- Crushing of the concrete in compression before yielding of the reinforcing steel;
- Yielding of the steel in tension followed by rupture of the FRP laminate;
- Yielding of the steel in tension followed by concrete crushing;
- Shear/tension delamination of the concrete cover (cover delamination); and
- De-bonding of the FRP from the concrete substrate (FRP de-bonding).

Concrete crushing is assumed to occur if the compressive strain in the concrete reaches its maximum usable strain ($\varepsilon_c = \varepsilon_{cu} = 0.003$). Rupture of the externally bonded FRP is assumed to occur if the strain in the FRP reaches its design rupture strain ($\varepsilon_f = \varepsilon_{fu}$) before the concrete reaches its maximum usable strain.

4.3 Material properties.

Grade of Concrete of RCC deck Slab	M20
Grade of Concrete of PSC Girders	M35
No. of cables in each girder	6
No. of Strands in a cable	8
Diameter of one strand	8mm
Tensile Strength of Pre-stressing Steel	1600 MPa

Table 2: Properties of concrete prestress wire.

Width of CFRP laminate	100 mm
Thickness of CFRP laminate	2.4 mm
Modulus of Elasticity of CFRP laminate, E	165 GPa
Tensile Strength of CFRP laminate	1800 MPa
Rupture Strain of CFRP laminate	0.012
Pre-stressing Force Applied on one Laminate	80 kN

Table 3: Properties of CFRP system.

Figure 4. Actual load has applied to the for monitoring before strengthening

Figure 5. Actual load has applied to the for monitoring after strengthening

4.4 Flexural strengthening.

In order to increase the flexural capacity and stiffness, reduce the distribution and width of the flexural cracks and improve the performance of the RC members under the service load conditions, the FRP material can be epoxy-bonded to the areas under tension while the fibers are oriented parallel to the principal stress direction. This type of strengthening can increase the ultimate flexural strength of the strengthened members from 10% to 160% (Dr. Khaled Galal 2016)[3]. For the railway bridge 3 numbers of prestressed laminate has provided at bottom of I girder and 2 number of non-prestressed laminate has provided at side face of bottom flange of I girder (as shown in figure) to increase the stiffness and the flexural capacity of the girder. The numbers of prestressed and un-prestressed laminate has calculated through retrofitting design calculation.

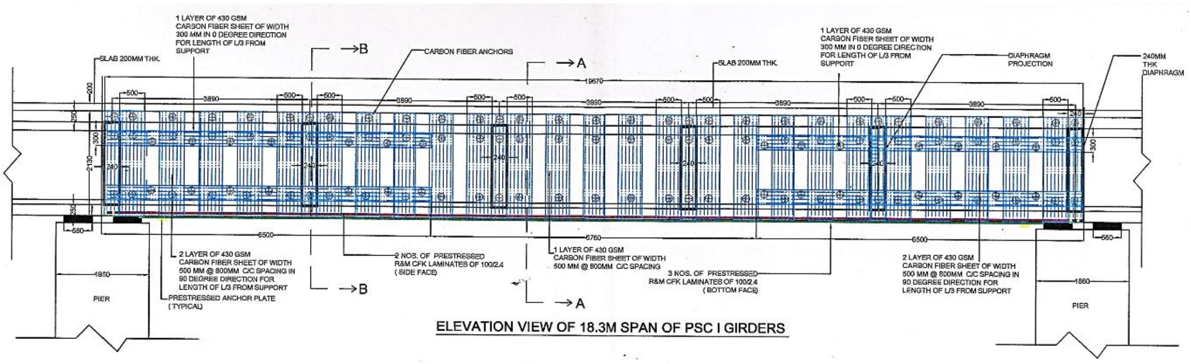

Figure 6. Elevation view of 18.3m span of Ratlam Railway Bridge PSC I girders.

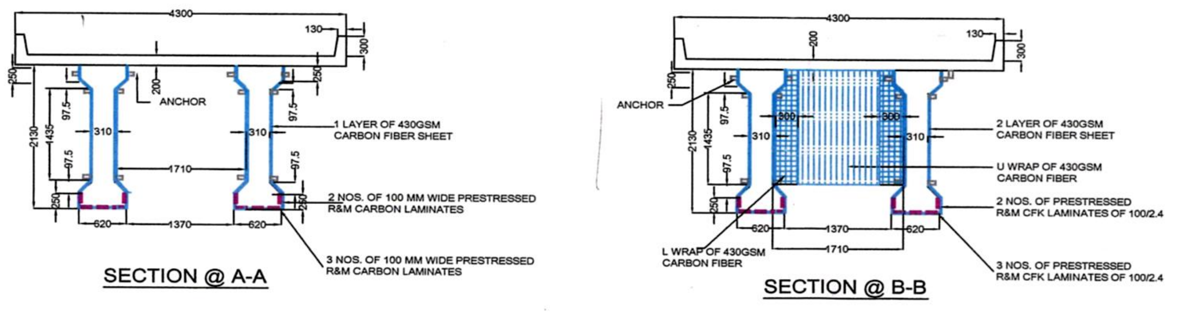

Figure 7. Sectional elevation of Ratlam Railway Bridge PSC I girders

4.4.1 Available data:

Grade of concrete of slab = M20

Equivalent cylindrical strength f_c'= 0.8 x 20 = 16 N/mm^2

Modulus of Elasticity $E_c = 4700\sqrt{f_c'} = 4700\sqrt{20}$ =21019 N/mm^2

Grade of concrete of PSC girder = M35

Equivalent cylindrical strength f_c'= 0.8 x 35 = 28 N/mm^2

Modulus of Elasticity $Ec = 4700\sqrt{f_c'} = 4700\sqrt{35}$ = 27806 N/mm^2

Area of one CFRP laminate, $A_f = n \times t_f \times W_f$ = 10 x 2.4 x 100 = 2400 mm^2

Cross section area $A_g = 2.55 \times 10^6$ mm^2

Gross Moment of inertia, $I_g = 1.45 \times 10^{12}$ mm^2

Distance from top fiber of sectional centroid, y_t = 941.46 mm

Effective force in cables, P_e = 4069 kN

Area of Prestressing wire, A_p = 7235 mm^2

Modulus of Elasticity of Prestressing cables, E_p = 19500 N/mm^2

Effective Prestressing Stress, $f_{pe} = P_e / A_p$ = 562.4 N/mm^2

Effective Prestressing Strain, $\varepsilon_{pe} = f_{pe} / E_p$ = 0.00288

Eccentricity of Prestressing force, e = 855 mm

Rupture strain of CFRP laminate, ε_{fu}^* = 0.012

The repair of bridge is directly exposed to external environment. Therefore, as per table 9.1 of ACI 440 a reduction factor CE of 0.85 is to be taken.

Hence, ε_{fu} = CE x ε_{fu}* = 0.85 x 0.012 = 0.0102

Ultimate strength of CFRP laminate, f_u = 1800 N/mm^2

Similar to designing RC member the nominal moment multiplied by the phi value must be greater than the ultimate moment of the beam as seen in equation.

$$\phi M_n > M_u \tag{1}$$

4.4.1.1 Load calculation:

BM due to Dead Load + Super Imposed Dead Load, M_{DL} = 5101.21 kN-m.
BM due to Live Load for 25t loading-2008, M_{LL} = 7166.03 kN-m.
Factored Moment, Mu = 1.2 M_{DL} + 1.6 M_{LL} = 17587.10 kN-m.

4.4.1.2 Design calculation:

Theoretically the de-bonding strain can be calculated as

$$\varepsilon_{fd} = 0.083\sqrt{\frac{f_c^{'}}{nE_f t_f}} \leq 0.9\varepsilon_{fu} \tag{2}$$

But the de-bonding strain has provided by manufacturer of FRP sheet was 0.008.

$$\varepsilon_{fd} = 0.008$$

The effective strain level in CFRP reinforcement at the ultimate limit state controlled by concrete crushing can be calculated by

$$\varepsilon_{fe} = \varepsilon_{cu}\left(\frac{d_f - c}{c}\right) - \varepsilon_{bi} < \varepsilon_{fd} \tag{3}$$

Where ε_{bi} is the initial strain and should be excluded from strain in the FRP system. Unless all loads on the member, including self-weight are removed before installation, initial strain will exist. In this case it was assumed that the stresses in the bottom fibers of the bottom flange of PSC girders are just in compression due to dead load(DL) and super imposed dead load(SIDL).
Hence strain at the beam soffit ε_{bi} = 0$\varepsilon_{bi} = 0$.
Depth of Neutral axis, C = 400.25 mm (adjusted after checking equilibrium)

Therefore $$\varepsilon_{fe} = 0.0145 > 0.008$$

The effective strain in the CFRP corresponding to 0.003 strain of concrete in compression is more than the de-bonding strain. Hence CFRP de-bonding will be failure mode.

$$f_{fe} = E_f x \varepsilon_{fe} \tag{4}$$

The above equation (4) provides the effective maximum stress level in the CFRP that can be developed before failure of section; assuming perfectly elastic behavior. Then, based on the strain level in the CFRP reinforcement, the strain level in the Prestressing steel can be calculated as

$$\varepsilon_{ps} = \varepsilon_{pe} + \frac{P_e}{A_c x E_c}\left(1 + \frac{e^2}{r^2}\right) + \varepsilon_{pnet} \tag{5}$$

Where, the net tensile strain ε_{pnet} can be calculated using following equation

$$\varepsilon_{pnet} = (\varepsilon_{fe} + \varepsilon_{bi})(\frac{d_p - c}{d_f - c}) \tag{6}$$

$$= 0.00713$$

$$\varepsilon_{ps} = 0.01001$$

Therefore, the stress level in the Prestressing steel is

$$f_{ps} = 1600 - \frac{0.276}{\varepsilon_{ps} - 0.0064} \tag{7}$$

$$= 1523.63 N / mm^2$$

and the stress level in the CFRP laminate is

$$f_{fe} = E_f x \varepsilon_{fe} \tag{8}$$

$$= 1360 N / mm^2$$

Since de-bonding of CFRP is the failure mode, the maximum compressive strain in concrete at failure from strain compatibility is as under

$$\varepsilon_c = (\varepsilon_{fe} + \varepsilon_{bi})(\frac{c}{d_f - c}) \tag{9}$$

$$\varepsilon_c = 0.00166$$

$$\varepsilon_c^{'} = \frac{1.7 f_c^{'}}{E_c} \tag{10}$$

$$\varepsilon_c^{'} = 0.00191$$

$$\beta_1 = \frac{4\varepsilon_c^{'} - \varepsilon_c}{6\varepsilon_c^{'} - 2\varepsilon_c} \tag{11}$$

$$\alpha_1 = \frac{3\varepsilon_c^{'}\varepsilon_c - \varepsilon_c^2}{3\beta_1 \varepsilon_c^{'2}} \tag{12}$$

$$\beta_1 = 0.734 \; and \; \alpha_1 = 0.839$$

For checking internal force equilibrium,

$$C = \frac{A_p f_{ps} + A_f f_{fe} - (\alpha_1 f_c^{'} (b_s - b_g) d_s)}{\alpha_1 f_c^{'} \beta_1 b_g} \tag{13}$$

Where, $\mathbf{b_s}$ = 2457 mm and $\mathbf{b_g}$ = 1240 mm

Therefore, C = 400.25 mm…(equal to assumed depth of neutral axis)

Hence the centroid of compression force from top, C_c = 128.16 mm

Now, Prestressing steel contribution to bending

$$M_{np} = A_p f_{ps} (d_p - C_c) \tag{14}$$

$$M_{np} = 21955.6 \; kN.m$$

$$M_{nf} = A_f f_{fe} (d_f - C_c) \tag{15}$$

$$M_{nf} = 7186.80\,kN.m$$

As εps is 0.001, a strength reduction factor k = 0.65 should be used and additional reduction factor ψf = 0.85 is used for CFRP contribution to normal capacity. The flexure strength of the section will be

$$\phi M_n = \phi(M_{np} + \psi_f M_{nf}) \tag{16}$$

$$\phi M_n = 18242\,kN.m > (M_u = 17587.10\,kN.m)$$

Hence the flexure strength of the girder has been increased to more than the flexure strength required with the help of CFRP laminate.

4.5 Shear strengthening.

The Shear Strength of a RC beam calculationis attributable to aggregate interlock, compressive zone concrete, dowel action, and transverse steel reinforcement, and can be increased significantly by bonding the FRP composites externally to the RC member, having the fibers crossing the shear cracks and parallel to principal tension stresses. Thereby, the beam will fail in flexure and the brittle shear failure can be avoided. The FRP composite is bonded to the beam covering either only the two sides of the beam (side bonding), or the two sides together with the tension face (U-jacketing). It is noteworthy that, covering the whole cross-section (closed wrapping) is possible only in bridge columns (and not the girder), because of girder's being integral with the slab. For the railway bridge girder FRP ply has applied as U wrap (as shown in figure 4 and 5) as it is most efficient.

4.5.1 Load calculation (DGC Engineering Report): [4]

The uniformly distributed load acting on girder= 174.4 kN/m.

The maximum shear force on girder = 0.5wl = 0.5* 174.4*19.17= 1671.624 kN

Factored shear force = 1.3* 1671.624 = 2173.111 kN

4.5.2 Load calculation (DGC Engineering Report):

Evaluation of shear capacity

Data Available

Total depth of beam D = 2330 mm

Effective depth of beam d = 2290 mm (Assuming a clear cover of 40 mm)

Breadth of beam b = 620 mm

Effective cover for Compression reinforcement d' = 0 mm

Area of Steel in tension Ast = 16855 mm2

Area of Steel in Compression Asc = 0 mm2

percentage of reinforcement p = 100As/bd = 1.19 %

Shear strength of concrete τc = 0.72 N/mm2 (Table 19, IS 456:2000)

Shear contribution of Concrete Vuc = τcbd = 1022.26 kN

Area of Shear reinforcement Asv = 157.00 mm2

Shear contribution of shear reinforcement Vus = 601.52 kN

Total shear strength of the section Vu,all = Vus + Vuc = 1623.78 kN < 2173.111 (required)

Hence, the section is unsafe so strengthen is required

The shear contribution of FRP shear reinforcement is given by

$$V_f = A_{fv} x f_{fe} (S\text{in}\alpha - \text{Cos}\alpha) d_f / S_f \tag{17}$$

$$A_{fv} = 2nt_f W_f \tag{18}$$

$$f_{fe} = \varepsilon_{fe} E_f \tag{19}$$

$$\varepsilon_{fe} = \kappa_v \varepsilon_{fu} \leq 0.004 \tag{20}$$

$$\kappa_v = \frac{k_1 k_2 L_e}{(11900\,\varepsilon_{fu})} \leq 0.75 \tag{21}$$

$$L_e = \frac{23300}{(nt_f E_f)^{0.58}}$$

$$k_1 = (\frac{f_c'}{27})^{2/3} \tag{22}$$

$$k_2 = (\frac{d_f - 2L_e}{d_f}) \tag{23}$$

For our case (two sides wrap on full side face)

d_f = 2180 mm (depth of beam –thickness of slab

tf = 1 mm thickness of ply

E_f = 220 Gpa

Assuming n=1 no. of ply

f_c'= 28.00 N/mm^2 (0.8f_{ck})

ε_{fu} = 0.0155

α = 90^o

Thus, L_e = 18.567 mm

k_1 = 1.0245

k_2 = 0.9915

Hence $\kappa_v = 0.1023 \leq 0.75$ (Okay)

Thus, $\varepsilon_{fe} = 0.0016 \leq 0.004$ (Okay)

Which gives,

$$f_{fe} = 348.09\, N / mm^2 < 1250\, N / mm^2$$

For continuous side face wrap

$$W_f = S_f \tag{24}$$

Hence

$$V_f = A_{fv} x f_{fe} (\text{Sin}\alpha + \text{Cos}\alpha) \tag{25}$$

Thus,

$$V_f = 1521.5\, kN$$

Over this nominal strength a strength reduction factor, Ø =0.85, as recommended by ACI 318 and an additional strength reduction factor, ψ_f= 0.85, as recommended by ACI 440.2R should be applied to get the allowable shear strength due to fiber wrap.

Therefore, allowable shear strength due to wrap is

$$V_{uf} = \phi \Psi_f V_f = 1099.287\, kN > 200\, kN$$

Therefore, shear strength of the section after wrapping,

$$V_{uc} + V_{us} + V_{uf} = 2723.06\, kN > 2173.11 kN \quad \text{(required)}$$

The shear capacity of girder has increases considerably and it is greater than required shear force capacity. Hence the section is safe for taking shear force acting on it.

Figure 8. U-wrap FRP applied to PSC I girder

Figure 9. FRP ply and Non-Prestressed laminate applied to inner face side of PSC I girder

5 PRE-STRENGTHENING AND POST STRENGTHENING MONITORING RESULT OF GIRDER

Sr. No.	Girder	Loading Condition	Natural Frequency in Un-Cracked Girder (Hz)	Natural Frequency in Cracked Girder (Hz)	Natural Frequency in post strengthened Girder at 20 kmph (Hz)	% Increase in Natural Frequency as compared to the Un-cracked girder
1	Span 2	Dynamic	9.574	8.6435	9.2885	97.02%

Table 4. Comparison of pre-strengthening result with post strengthening result for natural frequency

Sr. No.	Girder	Loading Condition	Deflection in Un-Cracked Girder (mm)	Deflection in Cracked Girder (mm)	Deflection in post strengthened Girder (mm)	% Reduction to the excessive deflection compared to the Un-cracked girder
1	Span 2	Static	2.728	3.9656	3.08	71.48%
2	Span 2	Dynamic	2.644	3.9055	3.01	70.49%
Average % Reduction to the excessive deflection of cracked span No. 2 girder as compared to the sample un-cracked span No. 1 girder						70.99%

Table 5. Comparison of pre-strengthening result with post strengthening result for deflection

6. OBSERVATIONS

- The deflection in the un-cracked girder is less than the deflection observed in the cracked girder indicating that stiffness of un-cracked girder is more than cracked girder.
- Natural Frequency of un-cracked girder is more than the natural frequency of cracked girder indicating that un-cracked girder has enhanced stiffness compared to cracked girder.
- After strengthening there is an average 70.99 percentage reductions in excessive deflection in the cracked girder of span 2 as compared to the un-cracked girder of span No.1. Also, the reduction in excessive deflection is within 25% of the un-cracked girder.
- The natural frequency of the cracked girder is within 25% of the natural frequency of un-cracked girder.
- After strengthening the average percentage of the natural frequency of the cracked span girders is increased to 97.02 percentage of the un-cracked girder of span No.1.The natural frequency of cracked girder has been improved and brought closer to that of un-cracked girder.
- With the strengthening measures the stiffness of the cracked girder has been improved as indicated by the reduction in deflection and improvement in the natural frequency.

7 CONCLUSIONS

- Retrofitting of the bridge with prestressed and non prestressed laminate has increased the flexural strength of the girder.
- U wrapped CFRP sheet not only increases the shear strength but also increases de-bonding strength of CFRP laminate.

- CFRP retrofitting reduced the excessive deflection in cracked girder about 70% compared to the deflection in un-cracked girder.
- The cracked girder got stiffened after retrofitting and hence its natural frequency has increased almost nearly to un-cracked girder i.e. about 97% of un-cracked girder.
- The retrofitting with FRP not only improves the performance of bridge reduced by crack in girder but also cease the further propagation of active crack up to considerable extent as deflection check result showing constant deflection value for throughout monitoring for passing of one year after retrofitting.
- The CFRP retrofitting system proves effective and successful.

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