CHAPTER 3: RENOVATION TECHNIQUE FOR THE PROMOTION OF LOADING CLASS OF AN OLD STEEL TRUSS BRIDGE

This chapter presents an expeditious renovation technique for the promotion of the loading class of the through-type historic steel truss bridges. To develop this concept for real-time use, an existing bridge functional for more than 50 years was selected for the assessment. The bridge was modeled in the STAAD Pro to analyze for its existing state and the state after the incorporation of the strengthening measures in it.

The main feature of this renovation technique was the proper utilization of the open space available in the sections of the main members so as to overcome the deficiencies of the existing bridge and make it comparable to the new steel-concrete composite bridges or the double-composite bridges. It was a handy and cost-effective renovation technique where no member was replaced. All the members were found to be safe in design and serviceability (*upgraded from lower than class-B loading to maximum of 70R loading*). Along with properly strengthening the bridge, its historic appearance was also preserved.

3.1. CASE STUDY OF THE BRIDGE

3.1.1 History, location, importance and structure

A simply supported Baltimore-type steel truss bridge constructed by Hindustan Construction Company (India) in the year 1968 over Bhagirati river at Joshiyada town, in Uttarkashi district of the Uttarakhand state of India was selected for assessment. It is a single lane bridge having a span of 68.4 m (shown in Figure 3.1). The members of the truss were majorly assembled using riveting and were connected with each other by riveted connections using gusset plates.

Figure 3.1 Picture of the selected steel truss bridge situated in Uttarkashi, India

3.1.2 Major issues encountered with the existing bridge

The bridge was having issue of noticeable vibration on the movement of vehicles. The bridge was designed for class-B loading and the signage on the bridge showed the maximum weight of vehicles as 16.2 Ton (*less than class-B loading*) but plying of heavy trucks was very common, so the barricades were installed to restrict the movement of heavy vehicles as shown in Figure 3.1, Even-though they needed that passage but the heavy vehicles were restricted from moving through the bridge because of the incapability of the bridge to carry even the class-B loads.

3.1.3 Current status of the bridge

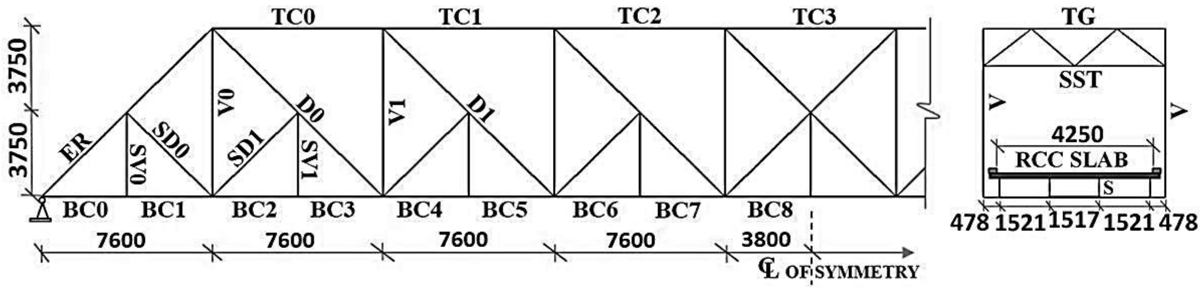
The initial work of retrofitting, which included the replacement of existing slab has been completed. After the replacement of slab (as shown in Figure 3.2), problem of vibration was resolved but the restriction over plying of heavy vehicles was still not lifted. As per the authorities, the whole bridge needed to be renovated to carry the loads higher than the class-B loads. This slab was designed in such a way that it would be capable of carrying the class-A loads also, as if the whole bridge was upgraded for class-A loading. The plying of heavy vehicles in the past shows the importance and the requirement of that bridge for movement of heavy vehicles, which has been restricted as of now.

a) Reinforcement in slab b) Finished slab construction

Figure 3.2 Retrofitting of slab to control the vibration

3.1.4 Requirement and deficiencies

As far as the selected bridge was concerned, through-type, single span bridges can neither take advantage of conventional steel-concrete composite action as the slab is situated in the tensile zone of through type bridges; nor they require concreting in bottom chords as there is no intermediate support (*which require resistance to compression*) like in double composite bridges. The line diagram of the truss has been shown in Figure 3.3.



a) b)

Figure 3.3 a) Side elevation and b) front elevation view of the bridge

3.2. METHODOLOGY FOR RENOVATION

The existing bridge was modeled and analyzed on STAAD Pro software (as shown in Fig. 3.3), initially for the designated Class-B loading and then for the higher class of loading, Class-A loading (*with 500 Kg/m^2 load on remaining carriage width*). The analyses of the models of the existing state and the strengthened states of the bridge were not just the axial load analyses but the combined axial and bending stress analyses. The bridge was found to be fully safe under Class-B loading in design and serviceability both. It was found to be capable of moving much heavier vehicle after renovation, as only three members were found to be critical under Class-A loading.

Open sections of the compression chords (*TC and ER*) were utilized to overcome the excess compressive stress by filling them with high strength no-shrink micro-concrete which could be easily pumped from one end to another for the ease of concreting. For the formwork in *TC* and *ER* members, base plates of 3 mm were considered to be welded and to be left attached as nonstructural member to support concreting and to provide better encasing of the concrete. Concreting wasn't enough to allow heavy vehicles (*40T bogie load, class-70R loading*) in case of emergency because of lack of strength in tensile zone of the truss. To overcome this deficiency, along with the concrete filled compression chords, the bottom chords were post-tension prestressed using '12t13' prestressing cables and were filled with *M40* concrete. The pictorial representation of the conceptualization of the concrete-filled and prestressed (*CFP*) truss bridge for the renovation of old steel truss bridges has been shown here in Figure 3.4.

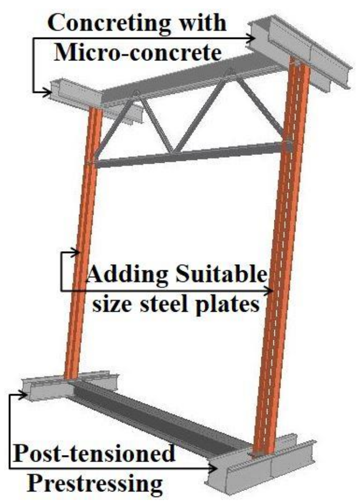


Figure 3.4 Conceptualization of concrete-filled prestressed (CFP) truss bridge

By incorporating the strengthening measures, the bridge was found to be capable of moving 40 Ton bogie load and 70R (*wheeled, W*) vehicle train load, with the need of strengthening of some intermediate members that were strengthened using extra steel plates at appropriate location of their section profiles. In the inspection of the bridge, damage and spalling of the bridge deck at different locations were observed. The replacement of the deteriorated slab by a 200 mm thick RCC slab with 75 mm thick wearing coat and the stirrups welded with stringer has been completed, which would also provide better stiffness against vibration.

3.3. ANALYSIS AND DESIGN

Various loads, load combinations and the percentage of permissible stresses were considered based on IRC-6 2010, IRC-24 2010, IRC-6 2016 and referring expert literatures and textbooks.

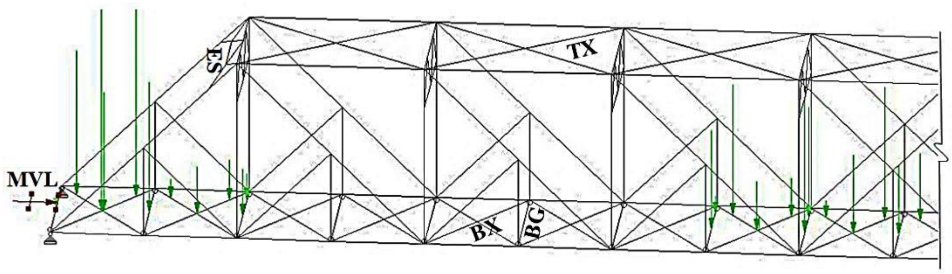


Figure 3.5 Moving vehicle load trains (*shown by arrows*) in STAAD model
Note: *BG, ES, BX, TX* and *MVL* represent bottom girder, end sway struts, bottom cross braces, top cross braces and moving vehicle load respectively.

3.3.1 Loading considerations and the analysis procedure

Class-A live load (with 500 kg/m2 load on remaining area): Live load was applied as shown in Figure 3.5, in the form of three consecutive moving trains of vehicles at a spacing of 20m from each other at an offset of 150 mm from kerb face. The axle load setup of the Class-A vehicle load train was as per the IRC-6 code.

70R(W) and 40T bogie live load: Spacing of three consecutive moving trains of vehicles was 30 m at an offset of 865 mm *(minimum limit was 300 mm for single-lane bridge as per Victor 2004 and IRC-6 2010*) from kerb face on both the sides of the bridge frame. The axle load setup of the 70R(W) vehicle load train was as per the IRC-6 code.

Impact load consideration: '*LL*' represents live load with added impact load, where the impact load is 15.4% of the live load for Class-A and 25% of the live load for 70R (W) and 40T bogie load.

Dead Load: '*SW*' represents the self-weight of members included in the analysis model and the additional dead loads like 500 kg/m^2 load on the area remaining after the application of Class-A load, load of concreting in *TC, ER* and *BC* (*used for prestressing*) and the load of the non-structural members not included in the analysis model.

Prestress: Considering load factor equal to 0.45 (*fatigue consideration*) and relaxation loss of 5% of the prestressing force, prestressing in each bottom chord using 16 prestressing cables of 12t13 type (in M40 concrete) was found to be appropriate for strengthening. It simulated in the STAAD model as the axial compressive loads of 1597 kN on both the roller ends and the weight of the M40 concrete was added separately.

Earthquake load: Earthquake load, '*EQ*' was applied for Zone-V (*which is the zone of the region in which the bridge is situated*) with 2% damping coefficient.

Wind Load: For Uttarakhand region, wind load, '*WL*' was calculated as per IRC-6 2016. In transverse direction, longitudinal direction and in vertical direction (*up/down*) the load per unit exposed area was taken as 2 kN/m^2, 1 kN/m^2 and 0.86 kN/m^2 respectively.

Additional considerations: The connections with *BG* member were provided close to the neutral axis, so in the analysis, *BG* members were considered as hinged; and for rest of the members, the analysis was done as rigid jointed framed structure. Continuous stringer

(*S*) and 200 mm thick continuous RCC slab were analyzed separately (*for all, self-weights were included in main analysis also*). Bottom cross brace arrangement was available in whole structure analysis also, but for its safety check, it was analyzed separately by including continuous concrete slab in the structure.

The percentage of permissible stress was taken as 100% for '*LL+SW*' load combination and was taken as 133% and 150% for the combinations of the loads involving '*WL*' and '*EQ*' loads respectively. Analysis was done for all the load combinations, available in IRC-6 2016 for the existed and strengthened states of the bridge. Even-though the carriage width of the bridge is best suited for the Class-A loading, bridge was found to be capable of carrying heavier vehicles after strengthening.

3.3.2 Details of strengthening

The section profiles of the members of the truss before strengthening and after strengthening (*only for the members that involved strengthening*) have been shown in Figure 3.6. The length and the open area of one *TC* segment were 7600 mm and 128800 mm^2 respectively; and that of the *ER* member were 10677.55 mm and 122360 mm^2.

After preliminary analysis, high strength micro-concrete of M75 grade was found to be appropriate for filling the open sections. Using the IS-456 code, the modulus of elasticity of the M75 concrete came out be equal to $5000\sqrt{75} = 43301.27$ N/mm^2, which resulted into modular ratio of 4.73 for the transformation of concrete into the equivalent steel. Areas of the equivalent steel in the *TC* and *ER* segments were 27230.44 mm^2 and 25868.92 mm^2 respectively. The difference of the weight of the filled concrete and the equivalent steel was applied separately in the analysis model. Considering the density of steel and concrete as 7833.4 kg/m^3 and 2400 kg/m^3 respectively, the extra UDL equal to 0.958 N/mm and 0.910 N/mm was applied on the *TC* and the *ER* respectively.

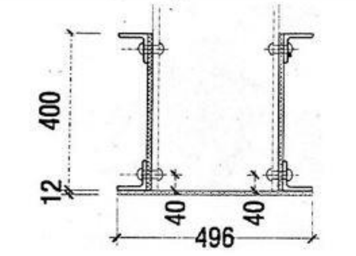
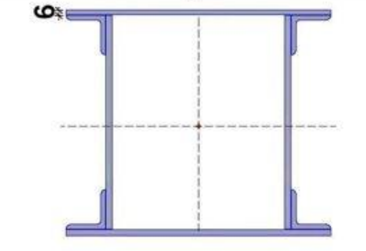
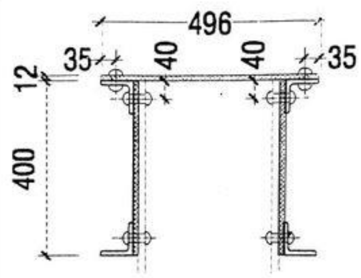
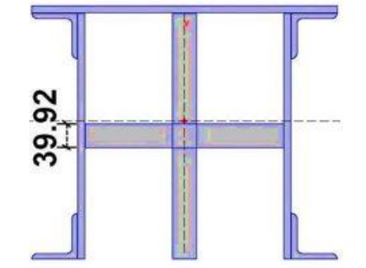
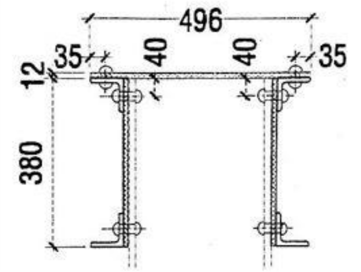
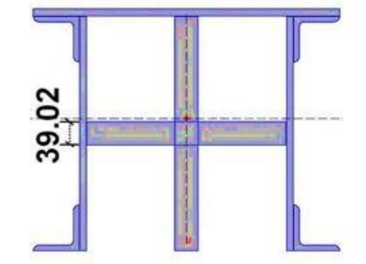
Member	Original profile (a)	Description	Modified profile (b)
BC8		a) 4-ISA 75X75X10 +12MM THK. WEB/ BOTTOM PL. + 10MM THK. TOP BATTEN. b) PRESTRSSING; 6 MM THK. PL. TOP	
TC		a) 4-ISA 75X75X10 +12MM THK. WEB/ TOP PL. + 10MM THK. BOTTOM LACING. b) MICRO-CONCRETE M75 FILLED IN SPACE	
ER		a) 4-ISA 75X75X10 +12MM THK. WEB/ TOP PL. + 10MM THK. BOTTOM LACING. b) MICRO-CONCRETE M75 FILLED IN SPACE	

Figure 3.6 Details of the main structural members

For the symmetrical distribution of equivalent steel, the width of splices was found be equal to 39.92 mm and 39.02 mm for *TC* and *ER* members respectively as shown in the Figure 3.6. In the next part of the analysis of the bridge model, post-tensioned prestressing of the bottom chord by 16 numbers of 12t13 prestressing cables (*in M40 concrete*) was found to be sufficient and appropriate, as addition of more prestressing had slight adverse effect on some of the intermediate and the connecting members.

Weight of M40 concrete was included in the analysis but it was not considered for strengthening in tension zone. Along with prestressing, in the BC8 segment alone, a 6 mm thick steel plate was considered to be welded on the top of the section to overcome excess moment. After the strengthening of the main members (by *not much affecting the other members*), the analysis for the strengthening of the other members was carried out.

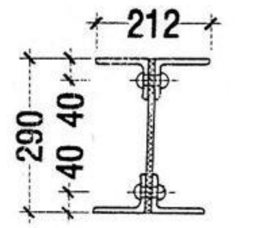
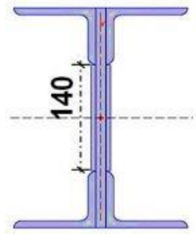
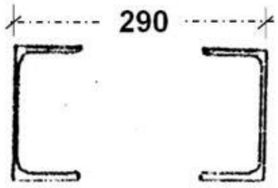
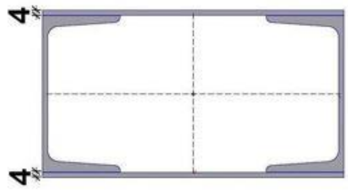
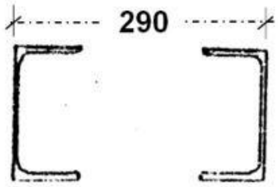
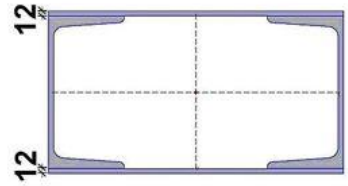
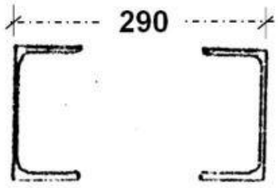
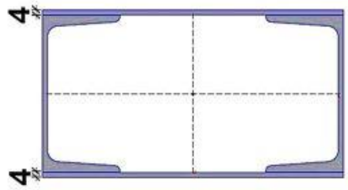
Member	Original profile (a)	Description	Modified profile (b)
D0		a) 4-ISA 100X75X10 + 12 MM THK. CENTRAL PL. b) 6MM THK. WEB PL. BOTH SIDES	
V0		a) 2-ISMC 150 + 60X10 MM. THK. LACING b) 4 MM THK. PL. BOTH SIDES	
V1		a) 2-ISMC 250 + 60X10 MM. THK. LACING b) 12 MM THK. PL. BOTH SIDES	
V2		a) 2-ISMC 200 + 60X10 MM. THK. LACING b) 4 MM THK. PL. BOTH SIDES	

Figure 3.7 Details of the intermediate members

Referring Figure 3.7, in diagonal, *D0* members 140 mm wide and 6 mm thick steel plates were connected on both the sides of the central section of the web. 4 mm thick steel plates were connected on the sides of the verticals, *V0* and *V2* members. 12 mm thick steel plates were connected on the sides of the *V1* members. Even-though, verticals *V1* and *V2* were found to experience axial compression, load of concreting in *V2* was nearly thrice the load of two 12 mm plates, whereas just 4 mm steel plates were found to do the same job.

3.4. RESULTS AND DISCUSSION

In this section, the analysis results have been presented through tables. In these tables, '*P*' represents calculated axial stress, '*P0*' represents permissible axial stress and '*IR*' represents the interaction ratio for the combination of the axial and bending stresses.

3.4.1. 70R (*W*) loading check

It can be inferred from Table 3.1 that the suggested strengthening measures improved the load capacity of the bridge such that even the Class 70R (*W*) vehicle trains could be moved in case of emergency. After strengthening, all the members were found to be safe both in the axial stress and in combined axial and bending stress analyses. In Table 3.1, data has been given only for the members that were found to fail before strengthening. After strengthening, axial compressive stresses obtained were nearly half of the stresses before strengthening for the members failing in 'buckling compression' (*TC and TX*).

Table 3.1 Axial and bending stress interaction ratio, IR Check for class 70R

Member	**Existing state Max. *P/P0***	**Existing state Max. *IR***	**Strengthened Max. *P/P0***	**Strengthened Max. *IR***
Vertical *V1*	0.85 (*SW+LL*)	1.31 (*SW+LL+WL*)	0.48 (*SW+LL*)	1.00 (*SW+LL+WL*)
End racker *ER*	0.85 (*SW+LL*)	1.32 (*SW+LL+WL*)	0.41 (*SW+LL*)	0.86 (*SW+LL+WL*)
Bot. Chord *BC8*	0.97 (*SW+LL*)	2.12 (*SW+LL*)	0.53 (*SW+LL+WL*)	0.94 (*SW+LL*)
Vertical *V0*	0.80 (*SW+LL*)	1.42 (*SW+LL+WL*)	0.54 (*SW+LL*)	0.88 (*SW+LL+WL*)
Vertical *V2*	0.86 (*SW+LL*)	1.06 (*SW+LL+WL*)	0.62 (*SW+LL*)	0.84 (*SW+LL+WL*)
Top cross bracing *TX*	1.09 (*SW+LL*)	1.41 (*SW+LL+WL*)	0.52 (*SW+LL*)	1.00 (*SW+LL+WL*)
Diagonal *D0*	0.98 (*SW+LL*)	1.06 (*SW+LL*)	0.90 (*SW+LL*)	0.99 (*SW+LL*)
Top Chord *TC*	1.17 (*SW+LL*)	1.28 (*SW+LL*)	0.56 (*SW+LL*)	0.63 (*SW+LL*)

Safe state of top braces (*TX*) members was attained as a result of strengthening of the upper compressive zone by concreting compression chords, without strengthening *TX* members individually. Strengthening of other connected members does affect the structural state of the *TX* members slightly. Some other noteworthy options tried for strengthening were the use of M40 concrete in place of high strength concrete which resulted into maximum '*IR*' ratio equal to 1.06 for *TX* members (*not high, but not safe* *also*). Addition of 10 mm plate for *V1* and 5 mm plate for *D0* resulted into their maximum '*IR*' ratio safe up-to single decimal value (*can be considered safe*). Using 20 prestressing cables instead of 16 resulted into much safer bottom chord but also had a minute adverse effect (*so proper analysis is must*) on the connecting members.

Table 3.2 Maximum deflection under class 70R loading

Location	Member	Existing state (mm)	Strengthened (mm)	Ratio
Central	Bottom girder *BG*	148 (*allowable 114*)	82	0.55

As per IRC-24 2010, the maximum allowable limit of deflection for this bridge is 114 mm. In the check of serviceability (*100% stress as permissible stress for all load combinations*), after strengthening, the maximum deflection of the bridge under 70R moving vehicle train reduced close to half of the deflection obtained for bridge without strengthening (as given in Table 3.2). The load of the deck slab was accounted but its stiffening effect was not accounted in the analysis (*further reduction in the deflection*).

3.4.2. 40T bogie load check

After analyzing for 70R vehicle train, the structure was analyzed for moving load of 40T bogies. As shown in Table 3.3, all the members (*keeping same strengthening as done for 70R loading*) were found to be safe under bogie load. Maximum deflection of 64 mm (*much safer than the allowable 114 mm*) was obtained at the central bottom girder.

Table 3.3 Axial and bending stress interaction ratio, IR Check for 40T bogie

Sl. No.	Member	Strengthened Max. P/P0	Strengthened Max. IR
1	Vertical *V1*	0.38 (*SW+LL*)	0.91 (*SW+LL+WL*)
2	End racker *ER*	0.31 (*SW+LL*)	0.79 (*SW+LL+WL*)
3	Bot. Chord *BC8*	0.42 (*SW+LL+WL*)	0.81 (*SW+LL*)
4	Vertical *V0*	0.40 (*SW+LL*)	0.77 (*SW+LL+WL*)
5	Vertical *V2*	0.44 (*SW+LL*)	0.71 (*SW+LL+WL*)
6	Top cross bracing *TX*	0.43 (*SW+LL+WL*)	0.92 (*SW+LL+WL*)
7	Diagonal *D0*	0.71 (*SW+LL*)	0.84 (*SW+LL+WL*)
8	Top Chord *TC*	0.45 (*SW+LL*)	0.50 (*SW+LL*)

3.4.3. Class A (with 500 kg/m^2 on remaining area, as per IRC-6) check

Table 3.4 shows the state of the strengthened members that were failing under class-A loading without strengthening. All the members were found to be safe in combined axial and bending stress analysis after strengthening. No member was found to fail in axial stress analysis (pin-jointed truss analysis).

Table 3.4 Axial and bending stress interaction ratio, IR Check for class-A

Sl. No.	Member	Existing state Max. *P/P0*	Existing state Max. *IR*	Strengthened Max. *P/P0*	Strengthened Max. *IR*
1	Bot. Chord *BC8*	0.69 (*SW+LL*)	1.40 (*SW+LL*)	0.42 (*SW+LL+WL*)	0.61 (*SW+LL+WL*)
2	Vertical *V0*	0.47 (*SW+LL*)	1.15 (*SW+LL+WL*)	0.34 (*SW+LL*)	0.71 (*SW+LL+WL*)
3	Top cross bracing *TX*	0.77 (*SW+LL*)	1.15 (*SW+LL+WL*)	0.42 (*SW+LL*)	0.90 (*SW+LL+WL*)

TX members were found to fail (*including buckling*) without strengthening, which would have needed the strengthening of individual *TX* member, but concreting of compression chords made *TX* members safe without strengthening *TX* members individually. Also, the prestressing in bottom chord and addition of 6 mm plate on to the top of the section of just *BC8* segments resulted into a very safe configuration.

Table 3.5 Maximum deflection under class-A loading

Location	Member	Existing state (mm)	Strengthened (mm)	Ratio
Central	Bottom girder *BG*	106	60	0.57

In the check of serviceability criteria, maximum deflection of the bridge under Class-A moving vehicle train was obtained at the central bottom girder. As shown in Table 3.5, the deflection was reduced close to half of the deflection obtained for bridge without strengthening. In the analysis, the load of the deck slab was accounted but its stiffening effect was not accounted, which would result in further reduction of the deflection.

3.5 ECONOMY

In bottom chord, for prestressing, 16 numbers of 12t13 prestressing cables and 846 bags (*in terms of 50 kg bags*) of M40 concrete were required. The overall requirement of steel for the strengthening of the intermediate members has been given below in Table 3.6.

Table 3.6 Details of Steel take-off

Existing state (kg)	Strengthened (kg)	Difference (kg)
126118	132002	5884

In top chords and end rackers combined, along with the requirement of the appropriate amount of 5-12 mm size aggregates, 909 bags (*in terms of 50 kg bags*) of M75 micro-concrete were required. Along with keeping the economy in the mind, the renovation strategies were introduced in such a way that without replacing any member of the bridge, the historic appearance of the bridge would be retained even after renovation.

3.6 CONCLUDING REMARKS

The old through-type steel truss bridge was converted into a concrete-filled prestressed (CFP) truss bridge just by the utilization of the spaces available in the sections of the main members. This configuration would overcome the deficiencies of the old through-type single span truss bridges. The old through-type simply supported bridges made-up of open built-up sections can be easily promoted to higher loading class by following the expeditious and handy procedure discussed in this article. Conclusions have been discussed elaborately in the last chapter, 'SUMMARY AND CONCLUSIONS'.