Low-Cost Rehabilitation with FRP Strips

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ABSTRACT:

Mechanically fastened, fiber reinforced polymer (MF-FRP) strips provided significant advantages for a low-cost, rapid construction, rehabilitation method that increased the structural load carrying capacity of a concrete flat slab bridge, while minimizing impacts to the traveling public. Faced with a difficult decision about how to spend limited financial resources on the bridge, the MF-FRP rehabilitation scheme economically upgraded the bridge capacity from a fourteen (14) Ton weight limit to legal loads (23 Tons).

INTRODUCTION:

In an age where budget woes and belt-tightening measures are in the forefront of everyone's minds, public transportation departments, from the Federal down to the Town level, are pressing to find any and all possible measures that will, at a minimum, preserve the status quo for public transportation structures. Small towns that have historically been strapped with lean budgets in trying to maintain their bridges are now even more hard-pressed to find low-cost rehabilitation alternatives. In addition the traveling public has become more and more demanding that construction efforts be conducted in a manner that minimizes any inconvenience to their schedules and normal daily routines. The rehabilitation method implemented for a small bridge in Gilford, New Hampshire provided an excellent low-cost alternative that was constructed with virtually no impact to the traveling public.

The use of FRP composites for repair/strengthening of deficient bridge structures has attracted increased interest during the last decade. FRP composites have several inherent advantages in that they are light in weight and corrosion resistant. FRP reinforcement has been used in numerous applications and has taken on diverse forms, including smooth and deformed bars and pre-cured and cured-in-place laminates. Techniques for employing the use of FRP surface reinforcement have the distinct advantage of being non-invasive and consequently they provide potentially economical solutions for repairing or strengthening bridges with minimal disruption to the traveling public.

ORIGINAL CONDITIONS

The Belknap Mountain Road over Gunstock Brook Bridge (locally known as the Glen Bridge, NHDOT Br. No. 184/098) is a 16'$\pm$ (clear span) structure (Figure 1). The concrete slab superstructure is supported on cast-in-place (CIP) concrete abutments founded on original ground. There were eight (8) small I-beams (8" deep) located directly beneath the concrete slab. These I-beams were located towards the interior of the bridge and were heavily rusted and provided little, to no, load carrying capacity that could be contributed to the overall structural capacity of the bridge. The reinforced concrete deck slab was 12" thick and it was reinforced with two mats of reinforcing steel. Width of the bridge was approximately 27'-6" out –out and 23'-5" curb-curb. The concrete substructure was in good to satisfactory condition other than the deteriorating toe walls that were cracked and partially undermined by the on-going scouring of the streambed. The structure is listed as originally built in 1907. The bridge underwent extensive reconstruction in 1973 with a new deck slab and abutment reconstruction, and there was further bridge rehabilitation work performed in 1994. This latest rehabilitation work included replacement of the bridge and bridge approach guardrail with new timber rail, removal and reconstruction of the concrete curbs, construction of toe walls and wing walls, and deck paving. The bridge was on NHDOT's Municipal "Red-List" of deficient bridges but in general the structure was in good condition. However, a significant concern with the bridge was that, structurally, it had a low load carrying capacity with a 14-ton weight limit. Fuel delivery trucks, fire trucks and fully loaded school buses, at an estimated weight of 20 tons, were thus restricted from legally crossing the bridge.

Figure 1 - Glen Bridge

THE CHARGE

The Town of Gilford requested that a solution be implemented to remove this bridge from the Red-List and upgrade its load carrying capacity to legal loads (approximately 23 tons for this bridge). The Town retained DuBois & King, (D&K) to evaluate the bridge and complete a bridge study to determine the best option for upgrading the structural capacity.

ALTERNATIVES

Four alternatives were considered for achieving the design legal load carrying capacity:

1. Total bridge replacement

2. Deck slab replacement/overlay

3. FRP using wet lay-up method

4. FRP Mechanically fastened method

The Town required that all the rehabilitation options (alternatives 2, 3 and 4) proposed for the bridge should provide a minimum of 20-25 years additional life for the structure, at which time a new structure would be considered. The evaluation of each of the four alternatives involved comparing the condition and functionality factors associated with the existing and proposed bridge, evaluating Town needs for the bridge, evaluating the costs associated with each alternative, and lastly, but most importantly, keeping a focus on the fundamental task of improving the load carrying capacity as soon as possible. Factors considered were:

- Substructure condition including on-going scour issues
- Hydraulic capacity of the existing bridge, i.e., is it adequate to pass the 50-year storm
- Superstructure condition – evaluated options for upgrading capacity
- Maintenance of traffic; minimize impacts to the traveling public, maintaining access to 40 residences (the bridge provides their only access)
- Existing roadway geometry is poor
- Attempt to avoid or minimize impacts to neighboring properties and environmental resources as much as practicable.
- Cost investment should be commensurate with the expected life span of the alternative; minimize costs to Town, if possible, while achieving the purpose of the project.

BRIDGE REPLACEMENT - Replacement of the bridge was given consideration but an important issue associated with any replacement option was to address the poor alignment of the existing approaches to the bridge. Alignment improvements quickly lead to significant ROW impacts (or possible road closure), substantial increases in cost and delays in the project schedule. The existing ROW was very narrow with constraints of a nature preserve on the upstream side of the bridge and an owner refusing access on any portion of his property on the downstream side of the bridge.

BRIDGE DECK REPLACEMENT/OVERLAY - A second alternative for accomplishing the goal of increasing the load carrying capacity of the structure to legal loads would be to replace the existing concrete slab with a new superstructure capable of carrying legal loads. The new slab could be constructed using either cast-in-place (CIP) concrete or precast concrete slabs. It was anticipated that it would be necessary to minimize the time period that the bridge was closed to one lane of traffic and consequently precast slabs would be preferred. Two (2) precast units would be required with the joint located approximately at the centerline of road. Existing timber bridge rail would be reconstructed and new membrane waterproofing and pavement installed.

A 4"to 6" thick reinforced concrete overlay was also considered as an option and costs investigated. The overlay would have necessitated reconstruction of the bridge curbs, bridge rail, bridge approach rail and shimming of the bridge approaches, in addition to the cost of the overlay itself plus new membrane and pavement.

FRP TO STRENGTHEN SUPERSTRUCTURE- A thorough search was performed for case studies and research papers, investigating the use of FRP strengthening systems for concrete bridge decks, or similar applications. The investigation focused especially on those applications documenting the use of FRP in a manner similar to that proposed for the project in Gilford.

After this search, the focus was narrowed to two externally bonded FRP options: 1) a wet lay-up application of carbon FRP laminates (sheeting) to be applied to the entire bottom of the concrete slab. 2) FRP strips (preformed) mechanically attached to the bottom of the concrete slab.

<u>FRP Wet Lay-Up</u> Several potential issues were discussed in the literature as being of importance to the long-term performance of this FRP System:

1. Proper preparation of the concrete sub-base (bottom of deck). The surface needs to be ground free of any protrusions/ridges that would prevent the FRP from laying flat and/or not having complete contact with the concrete deck. While this issue is not as critical for the mechanically attached FRP it is very important to the wet lay-up systems. The requirements for the different methods/systems could be addressed in the specifications, requiring the on-site presence and/or approval of the FRP supplier that the surface has been properly prepared. Preparation of the bottom of the concrete slab would also involve removal of the existing eight (8) steel beams. These beams would be cut off flush with the ends of the deck at both abutments and ground completely flush with the face of the abutments. Our field assessment determined that the top flanges of the beams were only minimally attached to the concrete deck and they could be removed without further cutting or concrete deck removal being necessary.

2. Debonding of the ends of the FRP, particularly if the ends of the material are in a tension zone, is a concern with the wet lay-up systems. Though the ends of the FRP may be in a very low tensile region, or possibly even in compression, it was advised that an angle or plate be bolted over the entire length of the ends of the FRP sheet in order to confine the ends and discourage delamination.

3. Again, with the wet lay-up systems moisture has the potential to degrade the bond between the FRP and the bottom of the concrete deck. To address this concern, the asphalt would need to be removed from the deck and a waterproof membrane installed on the top of the concrete deck; the bridge would then be repaved. In addition 1'-0" wide strips, at both curb lines and at the centerline of the bridge, could be left uncovered by the FRP on the bottom of the deck. These openings would allow the existing deck to breathe and provide an avenue of escape for any trapped moisture or chlorides.

<u>Mechanically Fastened FRP System (MF-FRP)</u> This proposal consisted of 1/8" thick by 4" wide preformed FRP strips extending the length of the bridge spaced at 12" on center. The 16' long strips would be fastened to the deck using high strength expansion anchors (wedge bolts). This alternative had a number of positive points in its favor as well as eliminating most of the potential concerns with the wet lay-up FRP system. The most significant factor in using the MF-FRP was that all the work could be conducted below the bridge deck and reduced impacts to traffic and resultant lower costs could be achieved. Using this system provided the significant benefit that new membrane and pavement would not be needed to seal the top of the deck. The existing roughness of the bottom of the deck also meant that the required level of preparation of the bottom of the concrete deck slab would be less than with the wet lay-up system.
The biggest concern with using the MF-FRP was the relative lack of information and the few bridges

where it had been previously used. However, the apparent simplicity of the system was correspondingly a significant factor in alleviating any concerns. Materials and installation procedures for the FRP were straightforward and this occasioned few opportunities for problems to occur either in construction or in its subsequent performance. In addition, the end product provided the opportunity to view the MF-FRP installed in place and to readily inspect if there were any problems developing. With an eye towards the cost advantages, significantly less impacts to traffic, and the goal of achieving a limited 20-25 years of additional bridge life, this alternative was selected.

Estimated costs in US Dollars for the four alternatives are provided below in Table 1.

Table 1 - Cost Analysis

COMPONENT	*ENG.*	*CONSTR.*	*TOTAL*
Bridge Replace	85,000	335,000	420,000
Deck Repl.	51,400	92,600	144,000
FRP Wet Lay-Up	37,000	48,000	85,000
FRP Mech. Fasten	35,000	36,000	71,000

ANALYSIS AND DETAILS OF THE MF-FRP ALTERNATIVE

Analytical models developed by L.C. Bank, AJ Lamanna, D. Arora and others were utilized to estimate FRP requirements and subsequently determine the capacity of the strengthened slab.

Design assumptions for the analysis were as follows:

1. Plane sections remain plane

2. The concrete has no tensile capacity

3. FRP contributes only to the tensile capacity (shear capacity of the section is based on the concrete section alone).

4. FRP has a linear elastic behavior.

5. The tensile capacity in the FRP is first transmitted via friction but ultimately via bearing on the high strength wedged anchor bolts. Slip will begin to occur with subsequent engagement of the FRP in a progressive fashion, more and more bolts becoming engaged at higher stress levels.

6. Capacity of the section is determined by the lesser of the bearing capacity of the bolts on the FRP versus the tensile capacity of the FRP net section.

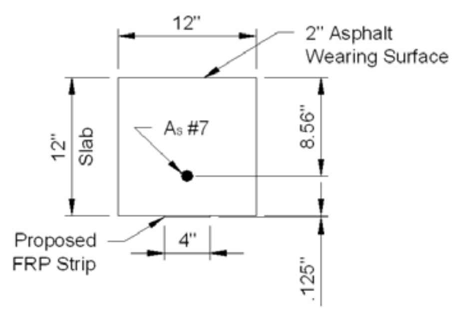
7. Cores taken from the deck (testing conducted in 1989) revealed nominal amounts of reinforcing in the top of the concrete slab as well as confirming the main reinforcement in the bottom of the slab. The top slab reinforcing was neglected in the analysis due to, for among other reasons, lingering uncertainties associated with this reinforcing (size, location, condition).

8. As stated previously, there were eight existing steel support beams underneath the deck. These beams were deteriorating and, since they were under only a portion of the deck, it was determined that any contribution to the structural capacity of the bridge from these stringers should be disregarded.

Material properties, rebar size and location, required for the deck analysis were obtained from the exploratory holes performed in 1989, plus additional concrete cores taken in 2010 to determine concrete compressive strength. A typical deck section is shown in Figure 2. The following bridge data was used in the analysis:

- Grade 40 reinforcement
- A_s: #7 @ 6.75"o.c.= 1.07 in2/Ft
- f'c: Original plans f'c=3,000 psi; concrete cores f'c approx. 6,000 psi. Used 4,000 psi
- Span: 17.33' (clear span 16.33')
- FRP: Design f_T= 92.9 ksi
- From ACI 440.2: Use Ce= 0.85 for exterior exposure. F_f= 0.85(92.9)=79.0
- A_{frp}= 0.50 in2

Figure 2 - Typical 1'-0" Wide Deck Section



Design for the reinforced structure was undertaken using proprietary software, as well as using hand calculations to confirm the recommendations for the required amount of FRP. After the software had provided the required amount of FRP, calculations, as shown below, were performed to verify that the subsequent section was capable of carrying legal loads. Per Bank and Arora, the nominal moment capacity, Mn, of the strengthened section can be calculated as follows:

$$M_n = A_s fy \left(d_s - \frac{a}{2}\right) + A_{frp} fr \left(d_{frp} - \frac{a}{2}\right)$$

A_s = Area steel reinforcement
A_{frp} = Area FRP strips
fy = steel yield stress
f_f = FRP stress
d_{frp} = distance to centroid of FRP
d_s = distance to centroid of steel

Then the bridge capacity was calculated as follows:

Determine "a" (depth of compression stress block):

$$F_s + F_f = F_c$$

$$40(1.07) + 0.125(4 - .438)(79.0) = 0.85(4)(12)a \qquad \textbf{Eq. (1)}$$

$$a = 1.91$$

$$M_n = (1.07)40\left(8.56 - \frac{1.91}{2}\right) + .445(79.0)\left(12.06 - \frac{1.91}{2}\right)$$

$$M_n = 325.5 + 390.4$$

$$M_n = 715.9^{"k}$$

$$M_n = 59.7^{'k}$$

$$M_{Dlnc} = 0.150\ (17.3)\ 2/8 = 5.6^{'k}$$

$$M_{Dlc} = 0.04\ (17.3)\ 2/8 = 1.5^{'k}$$

$$M_{allowable} = 59.7\ (.9) - 1.25\ (5.6) - 1.5\ (1.5) = 44.5^{'k}$$

Inventory Rating - Distribution of LL:

$$E = 4 + 0.06(17.3) = 5.04 \qquad \textbf{Eq. (2)}$$

$$1.75\left(\frac{Pl}{4} \times 1.3 \times \frac{1}{5.04}\right) = 44.5$$

$$P = 22.8\ k$$

Inv. Rating = HS28.5
Oper. Rating = HS 29.6

Therefore the strengthened capacity of the bridge exceeded legal load requirements (HS23).

CONSTRUCTION DETAILS & INSTALLATION

One of the significant benefits of the MF-FRP is the simplicity of construction. Contractors familiar with very basic construction practices should be able to install the strips satisfactorily. Therefore, the bid documents did not require that the contractor be prequalified for bridge construction. The low bid for the project was received from D&V Landscaping, a landscape contractor, though they had some previous concrete experience on a smaller scale.

Minimal preparation was required for the bottom of the slab in order to assure that the strips were seated against the concrete. Some patching and grinding of the bottom of slab was required (Figure 3), especially in those areas where the existing steel beams were removed. There were other areas with concrete ridges or sudden changes in smoothness from where the bridge had been widened on both sides and the concrete at the junctures between new and old did not evenly match. Grinding was approximately a week-long process including various interruptions to perform a scan of the bottom of the deck to determine rebar locations. Spalls and holes were filled and remaining areas of unevenness were transitioned using epoxy concrete filler.

Figure 3 - Underside of Concrete Slab

Prior to installing the FRP strips, the location of the main reinforcing mat in the bottom of the slab was mapped using an R-meter. Mapping the precise location of all primary reinforcing allowed for fine-tuning of the location of the wedge anchors so as to avoid damage to the reinforcing during drilling of the holes for the anchors.

During installation, traffic was routed away from portions of the deck where FRP installation was to take place. This allowed FRP strips to be installed and tensioned under dead loads only, and allowed the strips to fully engage when tension occurred at the bottom of the slab due to live load bending.

The 4" wide FRP strips (Figure 4) were obtained from a U.S. manufacturer. The FRP strips came predrilled with the prescribed bolt hole pattern. Holes were 7/16" in diameter to accommodate the 3/8" HS galvanized steel wedge bolts. The manufacturer specified the depth of holes as 2.25". One problem that was discovered late in construction was that, due to misunderstandings, the specified depth of installation for a number of the bolts was not achieved. The actual installed average embedment depth was conservatively estimated to be 1.5"; calculations confirmed that the installation was adequate and the tensile capacity of the strip was still the controlling design parameter.

Figure 4 - Installed FRP Strips

LOAD TESTS - Whenever a newer technology is used there are often unanswered questions, or perhaps only partially answered, by the first installations of the material. Some of these questions go to the heart of whether the technology accomplishes its fundamental purpose and therefore they are very significant. D&K sought to address some of these questions head on and approached the University of New Hampshire during the design process to ask if they would partner with us in conducting some research. The University was immediately interested in the project and together a plan was developed for performing several load tests on the bridge; the first test to be conducted in the pre-strengthened state (this was conducted on the original bridge but with the steel stringers removed) and the second test post-strengthened, i.e.; the FRP strips in place.

The initial pre-strengthened load tests were conducted utilizing an unloaded six-wheel dump truck weighing 23,540 lbs (front axle 12.5k, rear axle 11.0k). A loaded vehicle would have been much preferred in order to obtain larger deflections but it was determined that the load test needed to be kept within the 14 Ton posted weight limit to avoid any potential damage to the bridge. Deflections were obtained at five points underneath the bridge using three linearly varying displacement transducers (LVDTs) placed along the centerline of the bridge (Figure 5) and three digital image correlation (DIC) targets placed at three locations along the mid-span of the bridge. DIC measurements were taken only during the initial load test phase to confirm the readings that were obtained from the LVDTs.

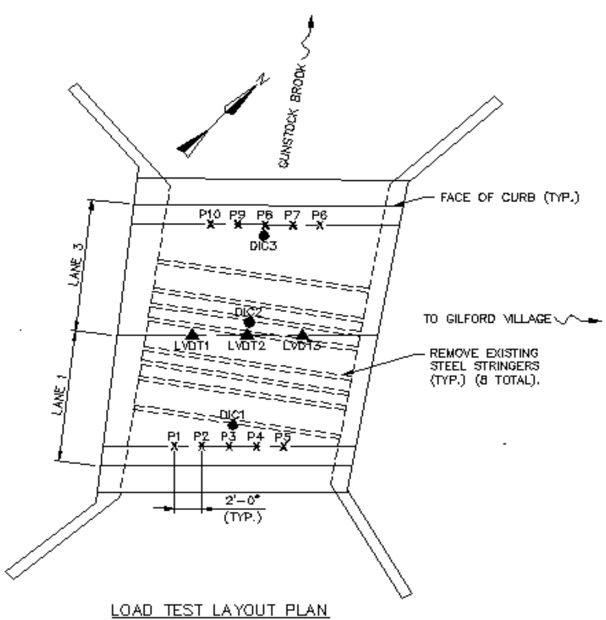
Figure 5 - Instrumentation Phase 1

Multiple passes of the test vehicle were made in both traffic lanes to provide redundancy to the deflection measurements. The vehicle travelled parallel to the curb-line(s) of the bridge at approximately a 3.0-foot offset from the face of curb to the edge of the rear tire (see Figure 6). The truck was stopped for approximately ten seconds with the line of the rear axle centered at each of five locations. Figure 7 shows the load test layout plan including instrumentation and truck placement.

Figure 6 - Truck Loading

The post-strengthened load tests were conducted approximately one month after the FRP strips had been installed and the bridge had been servicing traffic without any load restrictions. The tests conducted at this stage included both the same six wheel dump truck (identical vehicle) and then the same vehicle loaded with gravel; total weight of the loaded vehicle was 44,500 lbs (front axle 14.8k, rear axle 29.7k). Deflection measurements were obtained using the same identical grid pattern for both the unloaded and loaded truck. An additional load test was conducted with the loaded truck crawling across the bridge at approximately the centerline of the bridge.

Figure 7 - Load Test Layout Plan



Tables 2 and 3 below show bridge deflections as a function of time. The five major plateaus in the deflection diagram reflect the truck being stopped at the five longitudinal points (see Figure 7: P1-P5 for lane 1 and P6-P10 for lane 3), allowing sufficient time for the deflections to stabilize.

Table 2 - Deflection vs. Time, Original Br.

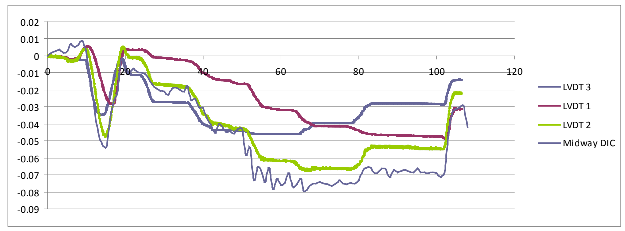
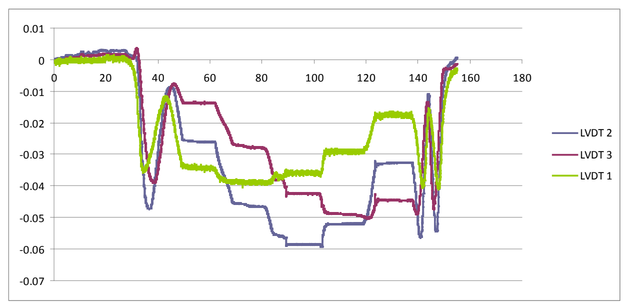


Table 3 - Deflection vs. Time, Reinforced Br.



Actual deflections were very small in magnitude but there was a small change witnessed between the pre and post-strengthened bridge; the maximum pre-strengthened deflection was .067 mm and the maximum post-strengthened deflection was .060 mm. A stiffness model of the bridge was developed to attempt to correlate the deflection readings with strength. Correlations were positive but not conclusive. Additional testing to evaluate strain in the FRP is currently in the planning stages.

CONCLUSIONS

The strengthening of the Glen Bridge was a success. The project was accomplished at minimal cost and minimal disruption to the travelling public. The posted weight limit has been removed and load tests confirmed that the addition of FRP strips has increased stiffness and the bridge capacity. An analysis performed on the bridge model also confirmed that under a fully loaded truck of 23 tons the bridge was operating well within acceptable stress levels.

New technologies such as MF-FRP are not one-size-fits-all solutions. This is just one more tool for the bridge engineer to employ, specifically where bridge capacity needs to be improved and a low-cost alternative is needed to extend the life of the bridge. The method employed on this bridge is particularly well-suited to concrete flat slab bridges, but it could be equally suited to concrete rigid frames, box culverts and other similar types of structures depending on the specific locations within the structure where strengthening is required.

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