Bridge Repair by External Prestress: The Gibe Crossing in Ethiopia

Marco Petrangeli
Professor
Pescara University,
Pescara, Italy
marco@integer.it

Gaetano Usai
Technical Director
Integra S.r.l.
Rome, Italy
usai@integer.it

Eugenio Zoppis
Director
Salini Costruttori
Rome, Italy
e.zoppis@salini.it

Summary

The Gibe bridge is located between Addis Ababa and Jima along the homonymous highway connecting the two Ethiopian cities. The bridge is made of a continuous, 4 span, reinforced concrete girder of 120m length and 9.6m width. The structure has 4 longitudinal beams, of variable height, connected by reinforced concrete diaphragms at piers, quarter spans and midspans.

The bridge, built in 1976-1977, suffered major damage from a terrorist bombing in the 80's. The bombing caused the collapse of a whole section of the deck that had to be re-cast in situ. The bridge has been used since then with load restriction with an alternative crossing available a few hundred metres downstream along the old road built by the Italians in '30. This latter crossing was an old steel truss girder built by the English in the '40 that collapsed in the fall of 2006 when a dozer crashed into the upper bracing of the main steel girders causing the spectacular collapse of the whole structure. Rehabilitation and strengthening of the concrete bridge became, all of a sudden, vital and urgent as it stands as the only connection between the agricultural South Western region and other parts of the country.

The Ethiopian Government contacted Salini Costruttori SpA, a contractor presently engaged in constructing hydropower projects in Ethiopia, with a request for technical assistance. Salini Costruttori SpA decided to perform the bridge strengthening as a contribution to the Country, and asked Integra to devise a fast, reliable and economic method to carry out the operations. The proposed solution makes use of external prestressing to increase the girder strength by closing the extended crack pattern and reducing bending and shear forces in the deck. The works, performed while the bridge was in service, were completed in three months and the bridge opened back to traffic without load restrictions in November 2007.

Keywords: concrete bridge retrofitting, external prestress, shear resistance.

The strengthening design

The severe damage suffered by the Gibe bridge deck would not allow for a simple retrofitting aimed at repairing cracks and spalled area. The lack of alternative crossing also called for a strengthening solution that could be implemented with the bridge open to traffic. Since the piers and foundations were in good shape, a screening of the different options was therefore carried out in order to identify the most simple and cost effective solution to strengthen the deck. The continuous deck configuration with variable height suggested the use of external prestressing as the most effective means to counteract bending and shear forces due to dead and live loading while introducing compressive stresses to seal the beam shear cracks. The deck depth and the diaphragms provided an optimal configuration for the cables to be deviated using steel saddles fixed to the concrete structure. Six cables with 12 (06”) strands could be easily spun under the deck, two on each side of the internal beams and one each on the inner side of the lateral beams. The cables are continuous over the whole length of the deck (120m circa) and anchored on the deck slab near the abutments. Cables are deviated using steel saddles and steel pipes directly glued onto the existing concrete sections. Only in few cases (mainly at the anchorages), additional cast in situ concrete blocks were required to allow for a safe transmission of the cable forces to the existing concrete sections.

Special care has been taken in the correct positioning of the deviation elements. Previous experiences suggested to check their correct positioning using a nylon rope before the saddles were glued and the strands were spun. Forces arising from strands jammed into the deviators can be as high as the tensioning force and therefore much higher than the local resistance of the concrete section. This was certainly the case for the Gibe bridge diaphragms that are insufficiently reinforced and therefore could only take the upward design deviation forces and would not bear any tangential ones. The 6 cables tensioned to 1000 MPa apply a total compression force of 1000 ton. The vertical component at the deviation points varies between 20 to 30 ton (upwards) for the 4 intermediate diaphragm along each span and from 70 to 110 ton (down-wards) at piers and abutment.

The combined action of bending moments and axial forces induced by the external prestressing suffice to counterbalance the permanent load effects. All sections along the deck are therefore fully compressed under dead and permanent load alone. Strengthening by post-tensioning is also particularly efficient with respect to shear because of the force vertical component of the inclined cables and the frictional contribution to the concrete shear resistance provided by the compression forces.

The principal advantage of the adopted solution is the extreme simplicity of its implementations. The steel saddles and deviation pipes were manufactured in shop and brought to site fully finished. The design of such pieces was optimized so that could be handled by two persons (max 80 kg). Installing of the saddles only required epoxy resin and small fasteners to hold them in place before the resin hardened and the prestressing forces pushed them against the concrete section. From the Gibe experience the following conclusion can be drawn:

- Concrete structures do often have large strength reserve, sometimes underestimated by the current codes. These strength reserves can be used when retrofitting damaged or deteriorated structures prolonging their service life.
- Strengthening of concrete structures by external prestress is generally very efficient. These structures, even when severely damaged, do benefit from a dramatic increase of strength when prestressed. The same beneficial effects are hardly obtained without prestressing as with standard FRP applications.
- Use of steel carpentry to couple the concrete structure to the external tendons is particularly efficient a sit is both robust and ductile. Installation is also easy, fast and cost efficient.

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Keywords: concrete bridge retrofitting, external prestress, shear resistance.

1. Introduction

The Gibe River Bridge is located 185 km from Addis along the National Road connecting the capital to the city of Jima, the largest city in the rich South-Western agricultural region of Ethiopia. The road was built by the Italians in the '30. The crossing of the Gibe river was provided with a masonry arch bridge later sabotaged during World War II and soon replaced with a steel truss girder simply supported on the existing arch abutments.

In the late '70 the crossing was displaced few hundred metres upstream on a more direct alignment that required the construction of a new concrete bridge (Fig.1) made of 4 spans for a total length of

120.5 m. Few years after its opening, during the civil war that preceded the fall of the military regime (early ’80), the bridge was severely damaged by a bomb that severed a full section of the deck. The missing deck section was recast in situ in the early ’90 thus allowing the bridge to be re-opened to traffic. The bomb damages were not fully repaired though, since the extensive cracking and spalling that took place on the adjacent spans could not be tackled at that time. Those damages caused a progressive deterioration of the structure during the subsequent years as the bridge is subjected to heavy traffic around the clock.

Fig. 1: The concrete bridge over the Gibe river

Recently, due to increased deck deterioration, load restrictions were enforced diverting lorries and other heavy freight to the downstream truss girder. Unfortunately, this structure collapsed in November 2006 (Fig. 2) when a dozer carried by a low loader exceeding the maximum clearance crashed into the girder upper bracing causing the whole structure to collapse. The traffic had to be diverted back onto the concrete girder with the same load restriction and a reduced carriageway width. The heavier freight had to be rerouted through Nekempt increasing the trip by 150 km along smaller and un-paved roads with severe damage to the country economy.

Fig. 2: The collapse of the old truss girder

The Ethiopian Roads Authority (ERA) had already assigned to a local contractor the works to build a new bridge a few dozen metres upstream of the concrete girder, but its completion will take a few years; meanwhile the situation was unbearable to the public and local administration that relied on that highway for transport of people and exchange of supplies. Therefore Salini Costruttori S.p.A. intervened assisting ERA by strengthening the existing concrete bridge at his own expenses.
The strengthening solution, as described in the following pages, has been outlined during the site visit of early 2007 by Prof. Petrangeli and Eng. Zoppis and thoroughly presented to ERA that promptly provided the go-ahead for the project. The works that lasted 3 months, with the bridge constantly opened to traffic, required 200,000 Euros of overseas procured technology and services and another 200,000 Euros of local cost (personnel, equipment and materials).

2. The Gibe Bridge

The bridge consists of a continuous reinforced concrete deck, 120.5 metre in length, with 4 spans, 35 m the central ones and 25.25 the lateral ones.The deck cross section is made of an 18cm thick slab and 4 rectangular beams, 40cm thick and with a variable depth between 3.15m over the pier to 1.7m at midspan. The beams are also connected by 18cm thick diaphragms, 4 intermediate ones each span plus one at each pier. Total deck width is 9.6m, with 8m carriageway and a pedestrian kerb on each side. The 3 piers, all with direct foundations on plinths, are made of 2 circular columns, 5 m apart, connected by a pier cap and a lower diaphragm.

2.1 Ante-Operam situation: damage assessment

From a simple visual inspection of the bridge it was clear the major damages were the very wide shear cracks (Fig. 4) at both side of the central pier (Pier 2). As said before, the bomb blast severed the deck at 5m from the lateral pier Jima side leaving the deck cantilevering out 25m from the central pier. This configuration, under dead load only, is by far the most severe the bridge could have possibly undergone during its whole history. With a deck self-weight of 14 ton/m the cantilever configuration increased the shear on Pier 2 of 110 ton and the bending moment of 3000 ton*m circa with respect to the pre-damage (continuous) configuration. The shear increased on both side of Pier 2 since bending increased the reaction on Pier 2 unloading Pier 1 (Addis side) thus increasing shear also on the span from Pier 1 to 2. The above mentioned values do not take into ac-count dynamic effects which may have increased the cantilever stresses (vertical vibrations) or added additional stresses via other dynamic mechanisms (axial impulse/vibration). The sharp increase of bending moment over Pier 2 was later confirmed by a closer inspection of the deck (Fig.5). The beams lower part had a large portion of concrete cover spalled and the reinforcing bars seemingly buckled.

Fig. 3: The Gibe River Bridge

Obviously the damage situation would have deteriorated because of rusting of the reinforcing bars exposed through the macro cracks and concrete cover spalled areas. Apart from a reduction of the net steel area, the resisting mechanisms which are very likely to deteriorate because of rusting and cyclic loading is the bond between rebars and concrete in the vicinity of the macro cracks. Once the cracks widen because of bond deterioration and/or rebar yielding, forces transmitted by aggregate interlock suddenly vanish potentially causing very brittle shear failure with associated collapse of the super-structure. The risk of a similar event in case of heavy loads crossing the bridge was therefore the prime source of concern.

Fig. 4 - Shear cracks in the deck beams

2.2 Strength assessment: ante-operam

An almost complete set of original drawings of the bridge was kindly provided by ERA, namely:

drawings of the original construction dating back to 1976/77 including rebar layout;

drawing of the repair works performed after the bomb blast (May 1993).

In these drawings, mechanical properties of concrete and steel used in the construction were set as follows: concrete characteristic strength fck=21 MPa, steel allowable stress бadm=140 MPa

Fig. 5 – Spalling of beam lower flange over Pier 2

Based on the above mentioned drawings and material properties section models have been set up and strength verifications performed.

As far as tests are concerned, it was decided not to postpone the strengthening intervention to allow for all the necessary equipment to be shipped from abroad since none of the basic instruments and equipment required for non-destructive test on bridges were available in Ethiopia at the time.

Using a finite element (F.E.) model of the bridge, maximum bending and shear forces due to permanent and live loading (AASHTO) have been calculated. Stress calculations for the deck based on the maximum bending moments are reported in the following table. These stresses are calculated using a unit load factor for both permanent and live loading.

The stress level, modest by today's standard, is unsafe for the structure under consideration given the poor quality of concrete and steel. The evaluation should also take into consideration that the above values have been calculated ignoring the damages to the concrete section and the corrosion to the rebars (reduction of resisting area) and assuming perfect bond (plane sections remain plane).

Table 1 – Section bending verification - Ante Opera

Section	Max. bending (kN*m)	σc (Mpa)	σs (Mpa)	Ultimate Bending resistance	Bending safety factor
Sagging shoulder span	8560	-4.7	156	13380	1.56
Sagging central span	9550	-5.0	155	15010	1.57
Hogging lateral pier	-23110	-6.8	143	-37620	1.63
Hogging central pier	-26000	-7.1	158	-43640	1.69

Nonetheless, as far as the flexural resistance is concerned, the bridge had some residual safety margin. This is also confirmed by the section Ultimate Limit State analysis and the corresponding safety factors summarized in Table 1. Once more, calculation ignored the existing damages, although, apart from some significant spalling, the modest corrosion and loss of bond would not have a significant impact on ultimate bending resistance of the section.

When it comes to shear verifications though, results are significantly different. Maximum shear force calculated with unit load factor is 1010 kN per beam. Shear resistance of each beam is only 60% of this value if the concrete contribution is neglected and 120% of it if concrete shear strength is accounted for. Given the macro crack pattern of at least 3 of the 4 beams, concrete contribution was very likely lower than what norms allow to consider. The numerical analyses therefore confirmed the shear as the most critical failure mechanism with a narrow safety margin. This safety margin was sufficient to keep the bridge standing only because traffic, although intense, never reached the maximum design values.

3. The strengthening design

The severe damage suffered by the deck would not allow for a simple retrofitting aimed at repairing cracks, spalled areas and corroded rebar. The lack of alternative crossing also called for a strengthening solution that would leave the bridge open to traffic during the works. At the time the repairs took place, the bridge was typically used by light weight trucks carrying all kind of goods and commuter busses. Since the piers and foundations were in good shape, it was decided to strengthen the deck. This solution is certainly competitive where alternative routes do not exists and where a quick bridge re-placement is not feasible. A screening of the different options was therefore carried out in order to identify the most simple and cost effective solution to strengthen the deck.

3.1 Strengthening with external prestressing

The continuous deck configuration with variable height suggested the use of external prestressing as the most effective means to reduce bending and shear forces and introduce compressive ones to seal the macro cracks. The deck depth and the diaphragms provided an optimal configuration for the cables to be deviated using steel saddles fixed to the concrete structure. Six cables with 12 (06") strands could be easily spun under the deck, two on each side of the internal beams and one each on the inner side of the lateral beams. The reason for not having cables on the outer side (of the lateral beams) is that these cables could not be deviated using the diaphragms and would be more exposed to weathering agents while spoiling the bridge aesthetic.

Figure 6 – Anchorage slot in the top slab

Longitudinal layout of the cables is optimized to counteract bending and shear due to self weight of the bridge. The cables are continuous over the whole length of the deck (120m circa) and anchored on the deck slab near the abutments. The cables follows two different layouts, one for the 4 inner cables (Type A) and another for the outer ones (Type B).

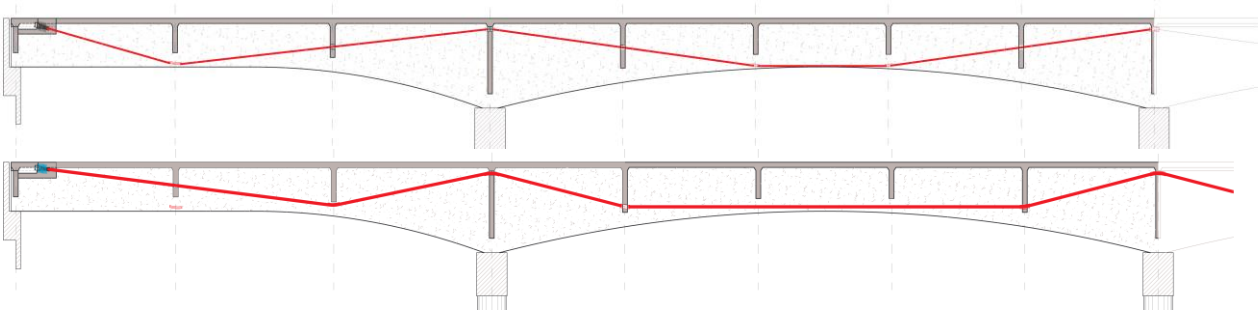

Fig. 7 – Cables Type A (above) and Type B (below)

The two layouts have been chosen so as to introduce the optimal deviation forces at each diaphragm.

Cables are deviated using steel saddles and steel pipes directly glued onto the existing concrete sections. Only in few cases, at the anchorages and at the deviations of Cable Type B over the piers, additional cast in situ concrete blocks were required to allow for a safe transmission of the cable forces to the existing concrete section. Special care has been taken in the correct positioning of the deviation elements. Previous experiences suggested to check their correct positioning using a nylon rope before the saddles were glued and the strands were spun.

As a matter of fact position and orientation are both very important because forces arising from strands jammed into the deviators can be as high as the tensioning force and therefore much higher than what the concrete sections can locally sustain. This was certainly the case for the Gibe bridge diaphragms that are insufficiently reinforced and therefore could only take the upward design deviation forces and would not bear any tangential ones. The 6 cables tensioned to 1000 MPa apply a total compression force of 1000 ton. The vertical component at the deviation points varies between 20 to 30 ton (upwards) for the 4 intermediate diaphragm along each span and from 70 to 110 ton (down-wards) at piers and abutment.

Figure 8 – Deviation saddle

3.2 Post Opera Verifications

The combined action of bending moment and axial forces induced by the external prestressing suffice to counterbalance the permanent load effect. All sections along the deck are therefore fully compressed under dead and permanent load alone. Under AASHTO live loading the section develops some tensile stresses but these are much smaller (Tab. 2) when compared to the Ante Opera situation (Tab. 1). Similar results are obtained for the safety coefficient at the SLU.

Table 2 – Section bending verification – Post Opera

Section	Max. bending (kN*m)	Axial Force (kN)	σc (Mpa)	σs (Mpa)	Ultimate Bending resistance	Bending safety factor
Sagging shoulder span	4250	9300	-3.0	0	19540	4.60
Sagging central span	7940	7500	-4.8	57	19930	2.51
Hogging lateral pier	-13900	8800	-5.3	39	-43710	3.14
Hogging central pier	-19920	6950	-6.4	79	-51720	2.76

Strengthening by post-tensioning is particularly efficient with respect to shear. Post tensioning does reduce shear forces because of the vertical component of the inclined cable but it also increase the section resistance as compression gives a frictional contribution to the concrete shear resistance. Ignoring the concrete contribution, shear safety coefficients are now up to 1.5, and taking into account the concrete contribution this value rises to 2.1.

3.3 Repair of damaged concrete

Before tensioning the cables, the major damages of the concrete deck have been repaired so as to benefit from the application of the prestressing forces. The wider cracks have been injected with epoxy resin, the large spalled portions near the supports and other major honeycombs have been repaired with highly fluid cement mortar. For the larger damaged areas, small diameter reinforcement has been added. The amount and extension of the repair works carried out on the concrete girder has been limited by budget restraint and cost-effectiveness evaluation. The authors have suggested ERA to proceed with a complete overhaul of the deck concrete surface in the near future. Luckily enough, Ethiopia benefits from a dry and warm weather that extends concrete structure service life well beyond European and North American standards.

4. Project implementation

The principal advantage of the adopted solution is the extreme simplicity of its implementations. All that is needed is a small by-bridge or otherwise suspended scaffolding that is placed at the deviation points as done for the Gibe bridge. The steel saddles and deviation pipes were manufactured in shop and brought to site fully finished. The design of such pieces was optimized so that could be handled by two persons (max 80 kg). Installing of the saddles only required epoxy resin and small fasteners to hold them in place before the resin hardened and the prestressing force pushed them against the concrete section. Deviation pipes were installed inside holes previously bored into the diaphragms with man held equipment.

Chemical anchoring of reinforcing bars in the existing concrete section and wooden form works to cast them using ready mix high strength concrete were required only for the cast in situ deviation blocks.

Fig. 9 – Suspended scaffolding at deviation points

The slot of the cable anchorages have been cut into the top slab and the additional reinforcing block cast from below the deck all in the same phase. Vehicles could run over temporary steel plates positioned over the anchorage slots.

With all the strands in position and all the anchorages ready, tensioning operations have been carried out with maximum symmetry with respect to the bridge axes of symmetry. The tensioning operation lasted 3 days: firstly cables have been tensioned to 50% of the design values (the authors were concerned about the deviation forces applied to the lightly reinforced diaphragms) . from both sides. At each side, tensioning started from the central cables and then moved to the outer ones. Finally the cables were tensioned to 100% of the design values from both extremities.

Strands have been tensioned to 1050 MPa. Strand elongation has been in accordance with the calculated value of 60cm circa and uniform for the different cables confirming the correct positioning of the deviation elements and that friction forces were within the expected values.

5. Commissioning

The bridge has been successfully commissioned on 3rd November 2007. The tests have been designed to guarantee the achievement of at least 85% of the maximum bending forces under the operating live loads prescribed by AASHTO. Taking advantage of the bridge symmetric design, only the spans on the Jima side have been tested, actually those that were worse damaged by the bomb blast. Therefore 4 sections were tested: the maximum negative bending moments on Piers 2 and 3 and the maximum positive bending moments on the central and shoulder (Jima Side) spans.

Few months after the bridge commissioning, Integra has been asked to asses whether the bridge could be crossed by the ultra heavy loads of the Gibe II transformers. Some of these convoys weighted less than 150 ton gross (including the truck) but two of them reached 213 ton gross. Shear forces induced by these heavier convoys would have been very close to the bridge capacity if not exceeding it. Using major international codes such as EC2, the calculated forces using unit load factor for both dead and live load exceeded 90% of the bridge capacity. Taking into account that the bridge did suffer from shear crack at the time of bombing and material properties could not be extensively investigated, the situation was to close to call without further test loads. These new tests load were conducted so as to simulate the transformer crossing. The bridge was loaded with four 40 tons trucks located so as to maximise shear and hogging moments close to the central piers. The bridge did withstand this test loading although displacements clearly indicated a non linear response due to some sort crack activation or re-activation. Elastic recovery was satisfactory though and therefore crossing of the transformers was given the go ahead.

Fig. 10 – The transformer convoy (213 ton) cross the bridge

Should the crossing be forbidden the convoys would have had to be rerouted trough a few hundred kilometres diversion of unpaved roads where the probability of tilting over are very high as happened during a previous attempt. In this unlucky event the transformer is abandoned as it is impossible to send a crane there to put the convoy back on track.

6. Conclusion

From the Gibe experience the following conclusion can be drawn:

- Concrete structures do often have large strength reserve, sometimes underestimated by the current codes. These strength reserves can be used when retrofitting damaged or deteriorated structures prolonging their service life.
- Strengthening of concrete structures by external prestress is generally very efficient. These structures, even when severely damaged, do benefit from a dramatic increase of strength when prestressed. The same beneficial effects are hardly obtained without prestressing as with standard FRP applications.
- Use of steel carpentry to couple the concrete structure to the external tendons is particularly efficient a sit is both robust and ductile. Installation is also easy, fast and cost efficient.